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MODAL IDENTIFICATION AND DAMAGE DETECTION ON A CONCRETE HIGHWAY BRIDGE BY FREQUENCY DOMAIN DECOMPOSITION

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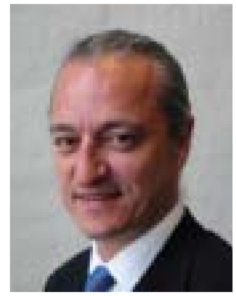


ABSTRACT:
As a part of a research project co-founded by the European Community, a series of 15 damage tests were performed on a prestressed concrete highway bridge in Switzerland. The ambient response of the bridge was recorded for each damage case. A dense array of instruments allowed the identification of a modal model with a total of 408 degrees of freedom. Six modes were identified in the frequency range from 0 to 16.7 Hz. The objective of this paper is to demonstrate the effectiveness of the Frequency Domain Decomposition (FDD) technique for modal identification of large structures. A second objective is to show the application of the FDD-method as an efficient way to perform health monitoring of civil engineering structures. The modal properties, frequencies, damping ratios and mode shapes for the different damage cases were compared with those for the undamaged bridge.

1. Introduction to Bridge Tests

The Z24-Bridge was an overpass of the national highway A1 between Bern and Zürich, Switzerland. It was a classical post-tensioned concrete box girder bridge with a main span of 30 m and 2 side-spans of 14 m (see Figure 1). Both abutments consisted of 3 concrete columns connected with concrete hinges to the girder. Both intermediate supports were concrete piers clamped into the girder. Although there were no known structural problems, the bridge dating from 1963 was demolished at the end of 1998. A new railway adjacent to the highway required a new bridge with one larger side-span. Before complete demolition, the bridge was subjected to progressive damage to study the influence of different realistic damage scenarios on its dynamic properties. A complete description of the fifteen progressive damage tests conducted can be found in Peeters [1], De Roeck [2] and Kramer, et al. [3]. During the night following on the application of a certain damage scenario, ambient tests and shaker tests were performed by EMPA [3].

The ambient vibration data was obtained in 9 data sets, 8 data sets with 33 channels and one with 27 channels (data set 5 being the data from the middle of the bridge). All sensors measured accelerations with a sensitivity of 5 V/g. The bridge was predominantly loaded by the air pressure waves produced by traffic passing underneath the bridge. Each data set consists of 10.9 minutes long time series sampled simultaneously at 100 Hz; the cut-off frequency of the anti-aliasing filter was 30 Hz. The response was measured on the bridge deck as well as on the piers. One 3D- and two 1D-sensors were kept as reference sensors at the same position in all data sets. The remaining sensors were roved along the bridge (one row at each side of the bridge and in the middle) and over the piers resulting in the above mentioned 9 data sets.

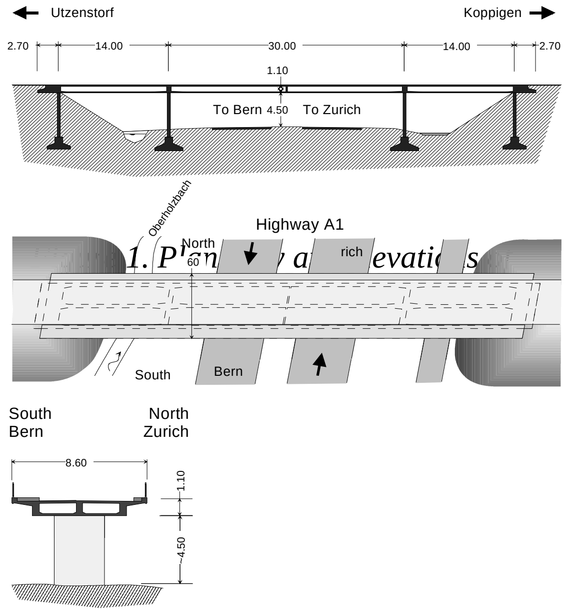


Figure 2. Plan view and elevations of Z24 bridge (after [1])

There were 120 measurement points on the bridge deck, 40 in each row, and 16 on the piers, 8 on each of the two piers. Thus, the total number of measurement points was 136. It is assumed that the transverse and longitudinal horizontal movement of the bridge deck was the same over the cross section of the bridge deck. Using this assumption, the 3D movements can be estimated in all of the 136 measurement points resulting in a modal model with 408 degrees of freedom. The damage cases of the 15 progressive damage tests (PDT's) can be summarized as follows:

PDT02: Reference measurement	PDT03:	Settlement of pier, 20 mm
PDT04: Settlement of pier, 40 mm	PDT05:	Settlement of pier, 80 mm
PDT06: Settlement of pier, 95 mm	PDT07:	Tilt of foundation
PDT08: Reference measurement	PDT09:	Spalling of concrete, 12 m2
PDT10: Spalling of concrete, 24 m2	PDT11:	Simulation of Landslide
PDT12: Formation of Concrete hinges	PDT13:	Failure of anchor heads
PDT14: Failure of anchor heads #2	PDT15:	Rupture of tendons #1
PDT16: Rupture of tendons #2	PDT17:	Rupture of tendons #3

2. Modal identification by Frequency Domain Decomposition

All data were analyzed using the enhanced frequency domain decomposition method (FDD) as described in Brincker et al. [4]. The enhanced FDD is based on the standard FDD technique (see Brincker et al. [5]), with the addition that mode shapes and natural frequencies are not only determined through a manual peak-picking procedure but using a robust estimation algorithm. This technique also allows a more accurate estimation of natural frequencies and estimation of damping ratios. All identifications were performed using the ARTeMIS *Extractor* software package [6], which permits the user to evaluate the data using different time-domain and frequency-domain techniques. These include the FDD and enhanced FDD procedures and the three classical versions of the Stochastic Subspace time-domain techniques: Principal Components, Un-weighted Principal Components, and Canonical Variate Analysis.

It has been advocated by many experienced analysts that the Stochastic Subspace Identification (SSI) algorithm is considered as the best choice for accurate identification of structures. The authors agree that the SSI techniques are among the strongest tools available today for output only identification. In this specific case, however, these techniques became too difficult to handle even for a user with ample experience. The SSI techniques were tested on the same data as the FDD technique, however, all of them failed to yield the modal parameters for all six modes in a way that could be considered as simple and user friendly. The problems encountered were related to the large amount of data and to the fact that all modes had to be clearly identifiable in all nine data sets. The authors believe, however, that if the monitoring of a structure is based on single data sets, and if the user choices of the SSI could be simplified somehow, the SSI techniques should still be considered a strong candidate for "user-friendly" modal identification and automated modal monitoring of structures.

The FDD technique can be explained in a simple way as follows. The response of any linear system can be expressed by

(1) $$\underline{y}(t) = \underline{\varphi}_1 q_1(t) + \underline{\varphi}_2 q_2(t) + \ldots = \underline{\underline{\Phi}}\,\underline{q}(t)$$

where $\underline{y}(t)$ is the vector of measured responses, $\underline{\varphi}_1, \underline{\varphi}_2, \ldots$ are the mode shapes, and $q_1(t), q_2(t), \ldots$ are the modal coordinates. $\underline{\underline{\Phi}}$ is the modal matrix and $\underline{q}(t)$ is the vector of modal coordinates. Now, the covariance matrix between the responses is

(2) $$\underline{\underline{C}}_y(\tau) = E\left\{\underline{y}(t+\tau)\underline{y}(t)^T\right\}$$

where $E\{\ \}$ is the expectation operator, and superscript T denotes transposition complex congugate. Now, if we then use the linear system equation (1) in equation (2), then we get the result

(3) $$\underline{\underline{C}}_y(\tau) = E\left\{\underline{\underline{\Phi}}\,\underline{q}(t+\tau)\underline{q}(t)^T \underline{\underline{\Phi}}^T\right\}$$

Or since the mode shape matrix is deterministic

(4) $$\underline{\underline{C}}_y(\tau) = \underline{\underline{\Phi}}\,\underline{\underline{C}}_q(\tau)\underline{\underline{\Phi}}^T$$

where $\underline{\underline{C}}_q(\tau)$ is the covariance matrix of the modal coordinates. If we take the Fourier transform then we obtain the final result

(5) $$\underline{\underline{S}}_y(f) = \underline{\underline{\Phi}}\,\underline{\underline{S}}_q(f)\underline{\underline{\Phi}}^T$$

where $\underline{\underline{S}}_y(f)$ is the power spectral density matrix of the responses, and $\underline{\underline{S}}_q(f)$ is the power spectral density matrix of the modal coordinates. Now, if the unknown inputs are un-correlated, then the covariance matrix $\underline{\underline{C}}_q(\tau)$ is diagonal, and so is the corresponding power spectral density matrix $\underline{\underline{S}}_q(f)$. Further, since auto spectral density functions are positive valued, the diagonal elements are always positive. In that case, if the mode shape matrix is unitary, then eq (5) is a singular value decomposition (SVD) of the response power spectral density matrix.

This is the idea of the Frequency Domain Decomposition (FDD) technique: by performing an SVD on the response spectral matrix, the singular values can be interpreted as the auto spectral densities of the modal coordinates, and the singular vectors as the modes shapes. The essential assumption is that the inputs on the modal coordinates are un-correlated, i.e. there must be independent sources loading the individual modes. The assumption of a unitary mode shape matrix is not essential. In case the mode shapes are not orthogonal, the technique will work nicely anyway following a procedure where a mode is determined only at the frequency line where it is dominating (where the corresponding singular value is maximum).

In the simplest peak-picking version of the technique, a mode is identified by the frequency line where it is dominating, and only information from that specific frequency line is used to obtain the modal information. Thus, the simple peak picking technique cannot be used for estimation of the damping. In the enhanced FDD technique, the auto spectral density function of the modal coordinate is identified and taken back to time domain by inverse FFT, and the damping and natural frequency is then determined from the corresponding single-degree-of-freedom auto correlation function.

3. Identification Results

The identification results given in this paper show a good picture of what is possible to do in engineering practice when it comes to modal identification of structures, and with the aim of using this information to monitor the health of the structure. The enhanced FDD is simple and reliable to use because the technique is intuitive and robust to user choices. For every mode the user wants to identify, the user just has to specify a frequency close to the natural frequency of that mode. Then based on a MAC criterion, the algorithm identifies the above mentioned single degree of system (SDOF) auto spectral density around the spectral peak that has been selected by the user. Mode shapes are obtained by averaging directly in the frequency domain using the identified part of the spectral function and the corresponding singular vectors from the singular value decomposition of the spectral matrix. Estimations of natural frequencies and damping ratios are then performed analyzing the free modal decay signals in the time domain checking for linearity at the same time.

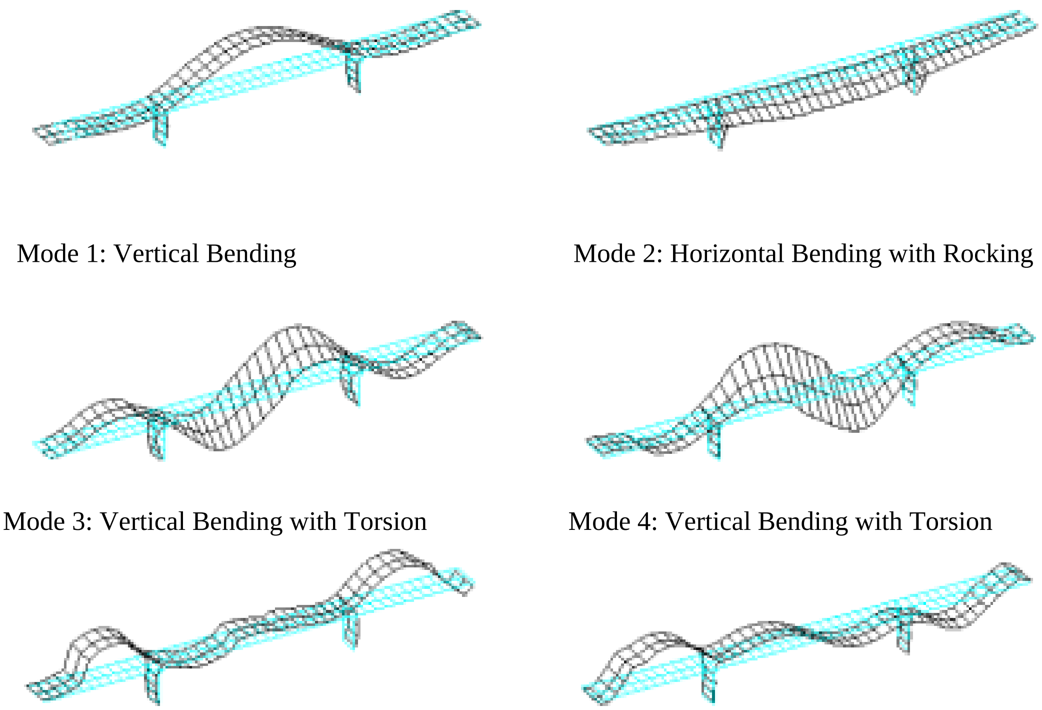
The identified natural frequencies for the first six modes are listed in Table 1 for all PDT's. The corresponding modal damping ratios are also given in Table 1. Typical mode shapes are shown in Figure 2.

Table 1: Identified modal frequencies and associated modal damping

PDT	Frequency (Hz)						Damping (%)					
	Mode No.						Mode No.					
	1	2	3	4	5	6	1	2	3	4	5	6
02	3.88	5.02	9.83	10.28	12.70	13.48	0.85	1.40	1.21	1.23	1.17	0.86
03	3.87	5.06	9.84	10.30	12.83	13.41	0.65	1.30	1.29	1.08	1.27	1.32
04	3.86	4.93	9.77	10.23	12.46	13.20	0.79	1.71	1.23	1.20	1.80	1.24
05	3.76	5.00	9.40	9.801	12.17	13.21	0.78	1.31	1.20	1.04	1.90	1.44
06	3.69	4.92	9.25	9.681	12.12	13.05	0.87	1.48	1.34	1.13	2.14	1.31
07	3.84	4.65	9.71	10.16	12.11	13.13	0.75	1.74	1.23	1.12	1.70	1.73
08	3.86	4.89	9.78	10.31	12.50	13.10	0.79	1.61	1.26	1.36	1.36	1.65
09	3.87	4.85	9.82	10.30	12.33	13.31	0.86	1.60	1.23	1.13	1.37	1.16
10	3.86	4.87	9.79	10.33	12.29	13.31	0.86	1.65	1.11	1.23	1.51	1.04
11	3.85	4.70	9.80	10.32	12.11	13.17	0.91	2.23	1.37	1.17	2.15	2.07
12	3.85	4.68	9.74	10.21	11.72	13.17	0.79	1.73	1.31	1.12	2.10	1.64
13	3.85	4.72	9.75	10.21	11.73	13.21	0.91	2.10	1.33	0.98	2.18	1.56
14	3.84	4.69	9.75	10.20	11.70	13.21	0.98	2.31	1.34	0.93	2.24	1.39
15	3.85	4.65	9.76	10.24	11.60	13.05	0.99	2.29	1.35	1.17	2.33	1.08
16	3.83	4.69	9.74	10.21	11.66	13.11	0.90	1.99	1.30	1.08	2.34	1.36
17	3.83	4.72	9.72	10.18	11.71	13.18	0.88	1.98	1.37	1.12	2.29	1.38

4. Health Monitoring

Monitoring of structural health using modal parameters allows detection of changes in structural stiffness. The stiffness changes can either be due to changes in the boundary conditions, due to local changes of stiffness such as localized cracking, or due to overall changes of material stiffness due to degradation of the material. In theory any change of stiffness can be detected from vibration data, since as a basic principle in structural dynamics, any stiffness change will cause a change of modal properties. However, modal properties are directly related to stiffness only and not to strength, and one must imagine that monitoring programs on real structures will be based on a combination of information from modal parameters and information from other sources. Those other sources of information might be a combination of traditional inspection techniques with data collection from static measurements of deformations, strain measurements on vital parts such as main tendons, etc. Further, if monitoring is to be done in a way where modal parameters play an important role,



Mode 1: Vertical Bending

Mode 2: Horizontal Bending with Rocking

Mode 3: Vertical Bending with Torsion

Mode 4: Vertical Bending with Torsion

Mode 5: Vertical Bending

Mode 6: Vertical Bending

Figure 2. *Typical mode shapes obtained from ambient vibration tests*

modal parameter changes due to damage must be distinguished from changes due to changes in the environment; i.e., from temperature effects. For the Z24, the temperature influence was investigated by keeping track of modal changes over a 9-month period of time. A discussion of the results of this investigation can be found in Peeters et al. [7]. For the Z24 PDT cases, all these damage scenarios were carried out over a relative short period of time during which only limited temperature changes took place. In this analysis it was assumed that temperature effects are marginal and negligible.

Health monitoring can be looked upon as a part of a more general problem that is usually called damage detection. It is normal in damage detection to think of the analysis in terms of different levels, often referred to as the "four levels" of damage detection: 1) detection, 2) localization, 3) quantification of a damage and 4) determination of the structure's remaining life. The results of the Z24 tests allowed the participants in the research project to successfully develop methods for a level 2 and 3 damage detection procedure. For instance, Maeck et al. describe in [8] the direct stiffness method, which allows level 2 and 3 damage detection using changes in mode shape curvature. This approach requires detailed knowledge of the mode shapes. For level-1 damage detection, the question asked is simple: "did any significant physical changes take place?" To answer this question a high modal spatial resolution may not be required.

Three variables were estimated for damage estimation: frequency deviation, damping deviation, and mode shape deviation. The frequency deviation was calculated as the relative frequency drop; i.e., the drop of natural frequency between the specific PDT and the reference

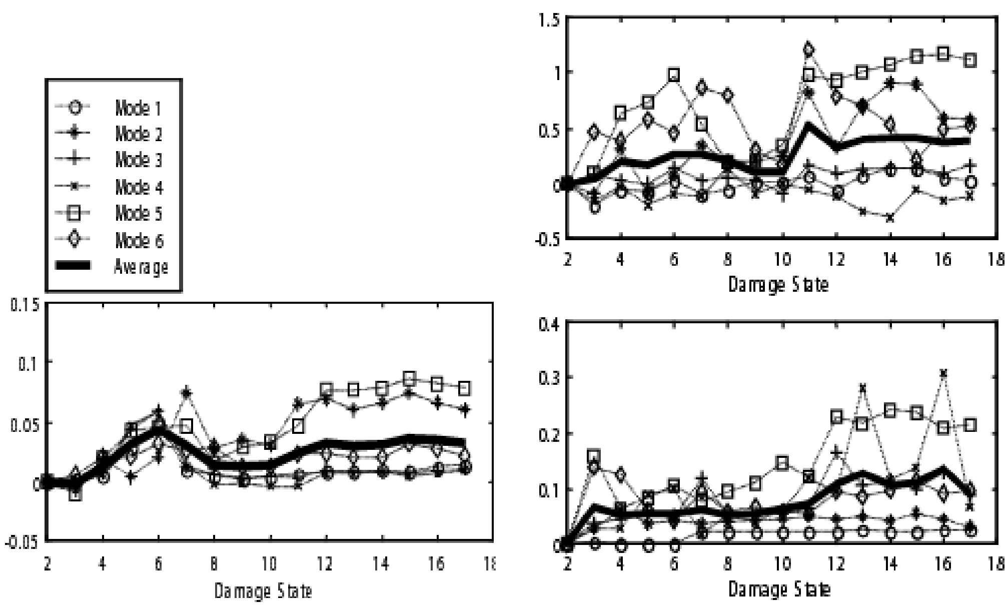


Figure 3. *Deviation of frequency (left), damping (top right) and MAC (bottom right)as a function of damage state*

test divided by the natural frequency of the reference test. The damping deviation was estimated in a similar way. The mode shape deviation was calculated as the deviation of MAC value from unity. The results are shown in Figure 3.As it appears from the results of the damage detection investigation, a clear frequency drop for all 6 modes can be detected for damage states 4-6. For later damage states a somewhat smaller but significant drop can be detected. For damage states 11-12 and later, a significant increase in the damping ratio can be detected. Some increase in mode shape deviation can be seen for all damage states. However, since the deviation per definition is one-sided, some of the deviation is due to random errors, and not necessarily due to damage. Cases 12-17 show a clear increase in the deviation indicating that damage has been introduced.

5. Conclusions

A major result of this investigation is the proof that today it is possible to perform reliable identification of even very large structures without any kind of artificial excitation. It is not only possible, it is actually quite easy, and the authors believe that practicing engineers with a solid knowledge in structural dynamics can readily do output only identification. Level-1 damage detection of the structure was performed using frequency, damping and mode shape deviations. All three deviations clearly indicated that damage had been introduced, however, the clearest indication is shown by frequency deviation. In practice, if no temperature compensation is used, mode shape information becomes more important since mode shapes are less sensitive to temperature changes than natural frequencies. If temperature changes are measured, and if a reliable database is established making it possible to filter out the influence of temperature shifts and other environmental changes on the natural frequencies, then one can argue that frequency deviation should be a key parameter for health monitoring.

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