by R. Brincker, P. Andersen and R. Cantieni

IDENTIFICATION AND LEVEL I DAMAGE DETECTION OF THE Z24 HIGHWAY BRIDGE

As a part of a research project co-founded by the European Community, a series of 15 progressive damage tests were performed on a prestressed concrete highway bridge in Switzerland. The ambient response of the bridge was recorded for each damage case with a relatively large number of sensors. This dense array of instruments allowed the identification of a modal model with a total of 408 degrees of freedom (DOF's). Six modes were identified in the frequency range from 0-16.7 Hz. Time-domain and frequency-domain techniques were used for the identification process. Because of the large amount of data and because of noise content in the signals it was difficult to apply parametric identification techniques like the Stochastic Subspace Identification algorithm. In contrast, it was relatively easy and straightforward to identify all modes in all the 15 tests using the enhanced Frequency Domain Decomposition (FDD) technique. The primary aim of this article is to demonstrate to engineers and technicians the effectiveness of this technique for modal identification of large structures. A second aim of this article is to show that the application of the enhanced FDD-method is an efficient way to perform level-1-health monitoring of civil engineering structures. A third aim is to illustrate that efficient health monitoring can be performed using only a rather limited number of sensors. The identifications were carried out for three different cases: the full 3D data set, a reduced data set in 2D (104 DOF's) and finally, a 1D set including 8 DOF's. The modal properties for the different damage cases were compared with those for the undamaged bridge. Changes in frequencies, damping ratios and MAC values were determined. An important observation from this investigation is that frequencies and mode shapes of the structure changed significantly during the damage cases.

DESCRIPTION OF TEST CASE

This paper describes the modal identification of the bridge Z24 overpassing the National Freeway A1 between Bern and Zurich in Switzerland. This prestressed concrete bridge with spans of 14-30-14 m was demolished in the autumn of 1998 due to construction of a new railway line parallel to Freeway A1. Before demolition, a series of 15 progressive damage tests (PDT's) were carried out. These tests were a part of the SIMCES project described in de Roeck [1]. The description of the PDT's can be found in Kramer et al. [2]. The damage cases 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 for further damage cases
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

The PDT tests start with PDT 02, a reference measurement of the undamaged bridge. Then follow PDT's 03 to 07, referred to as "reversible" PDT's. These PDT's were obtained by lowering of one of the piers. One pier was lowered in several steps (producing cracks in the bridge with a maximum width of about 2 mm) and put back to its original position after PDT 06. No reference measurement was made after that. One foundation was then tilted by 0.5 degrees, measurement PDT 07 made, and the foundation rotated back to its original position. After these PDT's what do we have now? A bridge that has suffered a good deal of cracking, but, because the elastic range of the reinforcement steel was never exceeded, the cracks closed again. However, to be able to check the effects of the remaining damage scenarios, it was decided to make a second reference measurement, PDT 08, on Z24. Hereafter came the so-called "irreversible" PDT's: chopping off concrete, digging out piers, cutting of anchor heads and tendons.

The vibration data were 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). Three reference sensors were used, two 1D vertical, and one 3D sensor. 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. However, in order to expedite the data analysis process, the time series were decimated by a factor of three hence decreasing the sampling rate to 33.3 Hz.

MODAL IDENTIFICATION

All data were analyzed using the enhanced frequency domain decomposition method (FDD) as described in Brincker et al. [3]. The enhanced FDD is based on the standard FDD

Editor's Note: Is it possible to detect structural damage using ambient vibration data?

In this interesting article, Prof. Brincker, Dr. Andersen and Dr. Cantieni address this question by showing first how can a proper identification of a structure can be done using modern techniques for analysis of ambient data, and then demonstrating how to do a level-1 damage detection with this information. To accomplish this they use the data obtained during a well-known research project on a bridge in Switzerland. This article is a welcome addition to the constantly increasing wealth of information on health monitoring of large Civil Engineering structures. Enjoy!

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technique (see Brincker et al [4]), with the addition that mode shapes and natural frequencies are not only determined through a manual peak-picking procedure but by using a more 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 [5], which permits the user to evaluate the data using different time-domain and frequency-domain techniques, including among others 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) algorithms should be 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 may become too difficult to handle for a user with limited experience. Firstly, since the number of channels and the measured data points are relatively large, the processing time becomes nearly unbearable. Secondly, since the high frequency part of the structural response in this case is contaminated by noise, the SSI techniques require a large number of noise modes that make it difficult to estimate the two modes in the high frequency region (mode 5 and 6) with confidence. It is simply too difficult to identify the structural modes among the large number of noise modes in this case. The third reason however might be the strongest argument against these techniques. The SSI techniques require intensive user interaction, the user must specify model orders, stabilization criteria, and finally sort out what information is related to the physical problem and what is not. Because of these reasons it is difficult to envision that these techniques could easily be used by engineers without significant specialized training and know-how. Actually, 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 enhanced FDD technique was used for the modal identification of the Z24 bridge simply because it is accurate, gives all six modes in all the damage cases, and is simple to use. Thus, the identification results given in this article show a pretty good picture of what is possible to do in engineering practice when it comes to modal identification of structures, with the aim of keeping an eye on 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 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. The corresponding SDOF free decays are obtained by transforming the identified spectral function back into the time domain by inverse FFT. Estimations of natural frequencies and damping ratios are then performed analyzing these free decay signals in the time domain. If the user specifies a frequency line that is a little off from the exact value, this only changes the identified part of the SDOF spectral function marginally, and thus, the modal parameters will be practically the same.

THE FULL 3D DATA CASE

Locations of the sensors for the first two data sets are shown in Figure 1. 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.

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 is 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 identified natural frequencies for the first six modes are indicated in Figure 2 and are listed in Table 1 for all PDT's. The corresponding damping ratios are given in Table 2.

Typical mode shapes for the first two modes are shown in Figure 3. The details of the bridge deformations are determined rather well. For instance, they clearly denote the clamped behavior between pier and bridge deck as shown in a close-up in Figure 4.

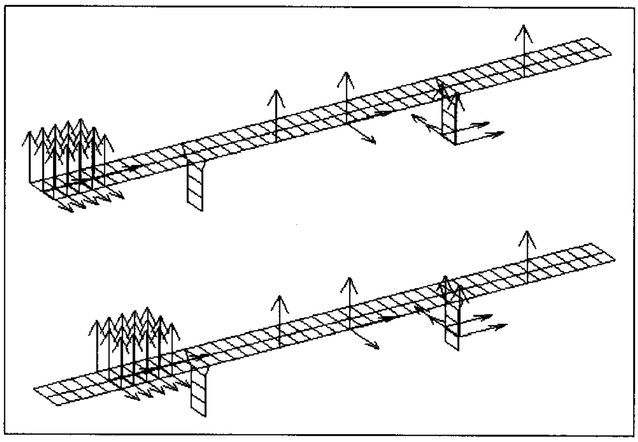

Fig. 1: Reference sensors and roving sensors in the first two out of the nine data sets for the Z24 bridge

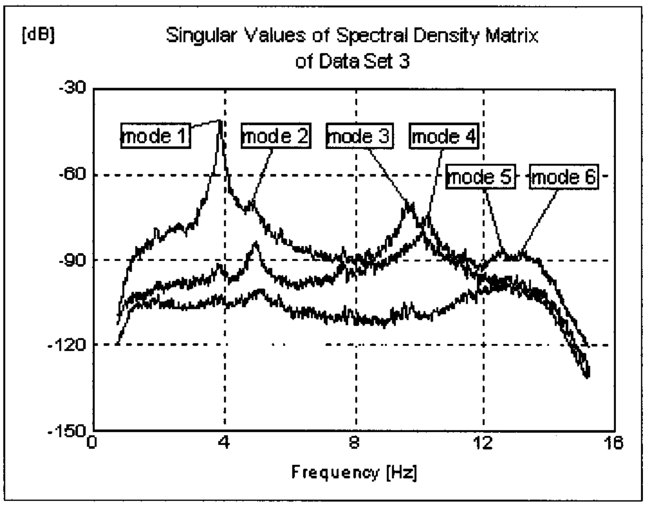

Fig. 2: Modes identified in all tests of the Z24 bridge

Table 1—Identified Natural Frequencies in Hz

PDT	MODE 1	MODE 2	MODE 3	MODE 4	MODE 5	MODE 6
02	3.876	5.021	9.828	10.28	12.70	13.48
03	3.871	5.059	9.836	10.30	12.83	13.41
04	3.858	4.926	9.770	10.23	12.46	13.20
05	3.763	5.003	9.397	9.801	12.17	13.21
06	3.686	4.917	9.253	9.681	12.12	13.05
07	3.842	4.648	9.705	10.16	12.11	13.13
08	3.856	4.886	9.783	10.31	12.50	13.10
09	3.869	4.853	9.819	10.30	12.33	13.31
10	3.860	4.871	9.789	10.33	12.29	13.31
11	3.853	4.696	9.799	10.32	12.11	13.17
12	3.846	4.678	9.735	10.21	11.72	13.17
13	3.847	4.715	9.747	10.21	11.73	13.21
14	3.842	4.689	9.754	10.20	11.70	13.21
15	3.846	4.648	9.764	10.24	11.60	13.05
16	3.830	4.689	9.739	10.21	11.66	13.11
17	3.825	4.720	9.720	10.18	11.71	13.18

THE 2D DATA CASE

Removing a large part of the measurement points and discarding all longitudinal sensors creates the 2 dimensional case. In this case only the vertical and the transverse horizontal movements of the bridge are determined.

This leads to a case with a total of 153 channels distributed in the 9 data sets, 36 measurement points on the bridge deck, one row at each side, and all the original measurement points on the piers. A total of 52 measurement points with 2 degrees of freedom each, thus, a case with a total of 104 degrees of freedom resulted. Typical mode shapes for the first two modes are shown in Figure 5.

Table 2—Identified Damping Ratios in %

PDT	MODE 1	MODE 2	MODE 3	MODE 4	MODE 5	MODE 6
02	0.85	1.40	1.21	1.23	1.17	0.86
03	0.65	1.30	1.29	1.08	1.27	1.32
04	0.79	1.71	1.23	1.20	1.80	1.24
05	0.78	1.31	1.20	1.04	1.90	1.44
06	0.87	1.48	1.34	1.13	2.14	1.31
07	0.75	1.74	1.23	1.12	1.70	1.73
08	0.79	1.61	1.26	1.36	1.36	1.65
09	0.86	1.60	1.23	1.13	1.37	1.16
10	0.86	1.65	1.11	1.23	1.51	1.04
11	0.91	2.23	1.37	1.17	2.15	2.07
12	0.79	1.73	1.31	1.12	2.10	1.64
13	0.91	2.10	1.33	0.98	2.18	1.56
14	0.98	2.31	1.34	0.93	2.24	1.39
15	0.99	2.29	1.35	1.17	2.33	1.08
16	0.90	1.99	1.30	1.08	2.34	1.36
17	0.88	1.98	1.37	1.12	2.29	1.38

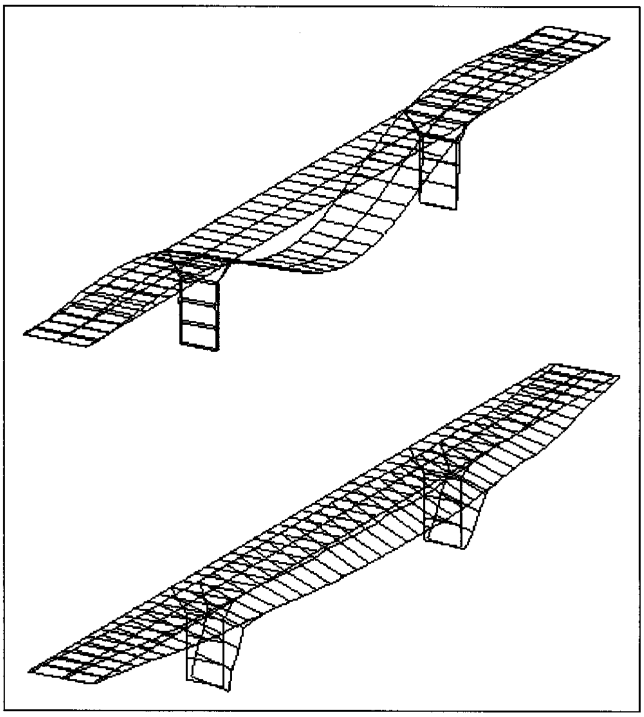

Fig. 3: Top: mode 1 vertical bending, bottom: mode 2, transversal rocking, horizontal bending

All natural frequencies and damping ratios determined in the 2D case were quite close to the estimated values for the full 3D case. The standard deviation of the differences between the natural frequencies for the two cases was determined to be 0.04 Hz, and the standard deviation of the

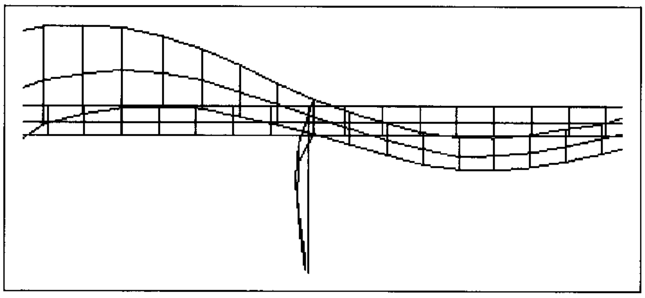

Fig. 4: Close-up of the shape for mode 3 showing the stiff connection between pier and bridge

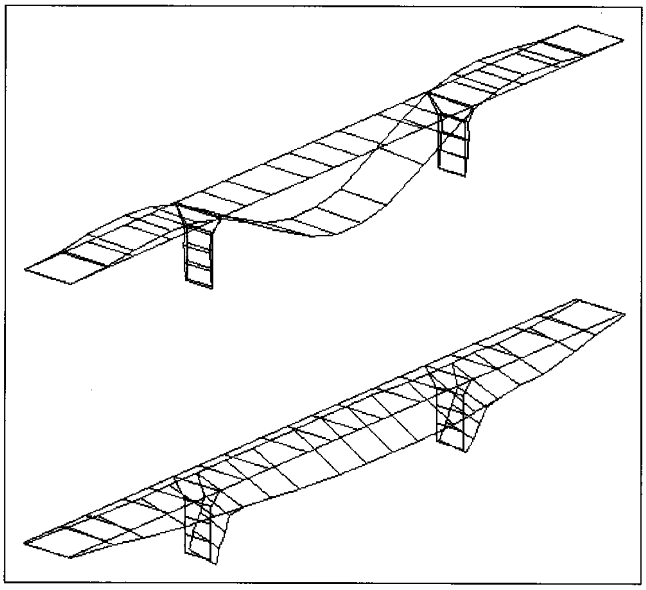

Fig. 5: Top: Mode 1 for the 2D case, bottom: mode 2 for the 2D case

difference between damping ratios for the two cases was determined to 0.20%.

THE 1D DATA CASE

Removing all horizontal sensors and keeping only two measurement points in the side spans and four in the mid span creates the 1D case.

All three reference sensors were kept for this analysis, but only the data sets 2, 4, 6 and 8 were applied. This leads to a case with a total of 20 channels distributed in four data sets, 8 measurement points each with one degree of freedom, thus a total of only 8 degrees of freedom.

All natural frequencies and damping ratios determined in the 1D case were quite close to the estimated values for the full 3D case. The standard deviation of the differences between the natural frequencies for the two cases was determined to be 0.06 Hz, and the standard deviation of the difference between damping ratios for the two cases was determined to be 0.33 %.

HEALTH MONITORING

Monitoring of structural health using modal parameters allows detection of changes in structural stiffness only. The stiffness change can either be due to changes in the boundary conditions, due to local changes of stiffness such as localized cracking, or due to over-all changes of material stiffness due to decomposition of the material. In principle any change of stiffness can be detected, since as a general principle in structural dynamics, any stiffness change will cause a change of modal properties. However, since modal properties are directly related to stiffness only and not to strength, in practice, 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 might be a combination of classical approaches such as manual inspection and modern approaches such as collecting data 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, modal parameter changes due to damage must be distinguished from changes due to changes in the environment, i.e. from temperature influence. In the case of Z24, the temperature influence was investigated by keeping track of modal changes over a 9 month period of time. A discussion of the results can be found in Peeters et al [6]. However, in the Z24 PDT case, all damage cases were carried out over a relative short period of time during which only limited temperature changes took place. Thus, in this analysis, a rather simplified approach is followed assuming that temperature influence is marginal and can be excluded from the analysis.

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 related research project to successfully develop methods for a level 2 and 3 damage detection procedure. Maeck et al describe in [7] the direct stiffness method which allows level 2 and 3 damage detection in the Z24 case using changes in mode shape curvature. However, this requires detailed knowledge of the mode shape, and thus, mode shapes estimated from the full 3D data set or at least the 2D data set.

Since damage detection on level 2 or 3 normally requires detailed information of the deviation of mode shapes between the undamaged and the damaged state, these kinds of analysis as a main rule require modal information with a high spatial resolution. Thus in these cases a large number of sensors should be used and/or the data should be taken using several data sets as it was actually done in this case. On the contrary, for level 1 damage detection, the question asked is simple: "did any significant physical changes take place?"

A well designed monitoring program for any structure is believed to be based on a strategy that allows for both a detailed comparison of modes for the undamaged and the damaged state and an economical monitoring instrumentation that allows for only level 1 damage detection. Thus, a monitoring program should include: a) a base-line test giving the full 3D modal properties of the structure, b) a long-term monitoring instrumentation giving enough information to perform level 1 damage detection and c), full 3D identification in the case of the level 1 damage detection program detecting any significant stiffness changes.

This article focuses on the economically important questions: How many degrees of freedom have to be measured to perform a reliable level 1 damage detection. Is it really necessary to keep track of the full 3D model? Does it suffice to use a 2D or even a 1D model?

LEVEL 1 DAMAGE DETECTION

To answer that question three simple damage detection variables were estimated for the 3D case, the 2D case and the 1D case: frequency deviation, damping deviation, and finally 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 test divided by the natural frequency of the reference test. The mode shape deviation was calculated as the deviation of MAC value from unity. The results are shown in Figures 6, 7 and 8.

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. Regarding frequency deviation no significant difference can be seen between the 3D, the 2D and the 1D case.

For damage states 11-12 and later, a significant increase in the damping ratio can be detected for the 3D and the 2D case, however, a similar increase cannot be seen for the 1D case.

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. Later cases 12-17 show a clear increase in the deviation indicating that damage has been introduced. The mode shape deviation of the later cases is stronger in the 2D and 1D case than it is for the 3D case. For the 1D case large and unstable deviations are seen for mode two. This is due to the fact that mode two was weakly excited by the present loads, and since in case 1D the horizontal movements are discarded, the influence on the remaining measured degrees of freedom simply becomes to weak for a reliable identification.

DISCUSSION

A major result of this investigation is 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 it is believed by the present authors, that output only identification is no longer only for the few specialists. Today, any engineer with a solid knowledge in structural dynamics could be able to perform this kind of analysis.

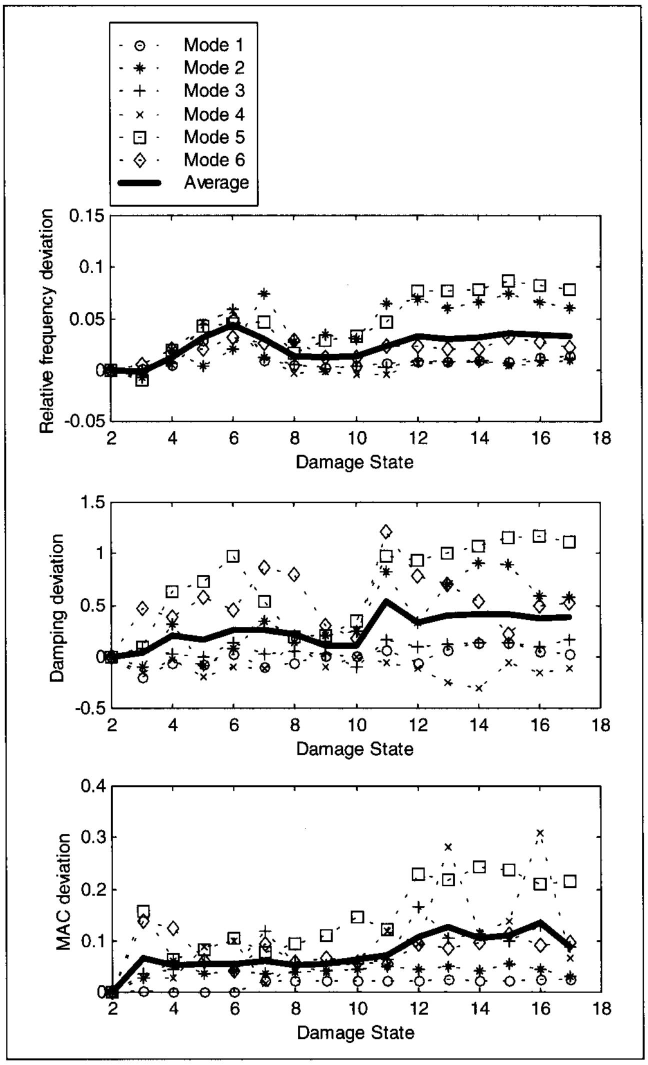

Fig. 6: Monitoring variables for the 3D case

It is believed that the FDD technique will come to play an important role in the future as a tool for practical identification of structures. Especially the enhanced FDD and other related techniques that are based on robust identification algorithms insensitive to user choices should have high priority for practical use. Even though the SSI failed to perform well because of practical problems with the large amount of data, the SSI techniques might still turn out to be a good choice in cases where it pays off to spend more time with the identification, or in cases with a limited number of data and channels.

With the frequency domain decomposition technique it was straightforward to identify all six bending/torsion modes for all the 15 damage cases in the frequency range 0-16.7 Hz for the full 3D case including 408 degrees of freedom, for the 2D case including 104 degrees of freedom and for the 1D case

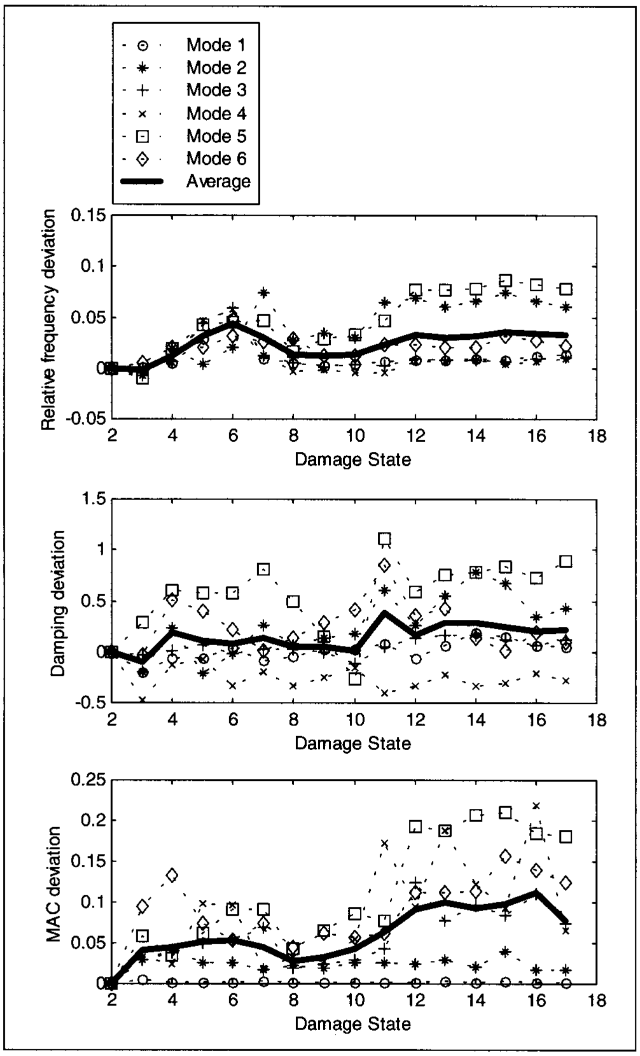

Fig. 7: **Monitoring variables for the 2D case**

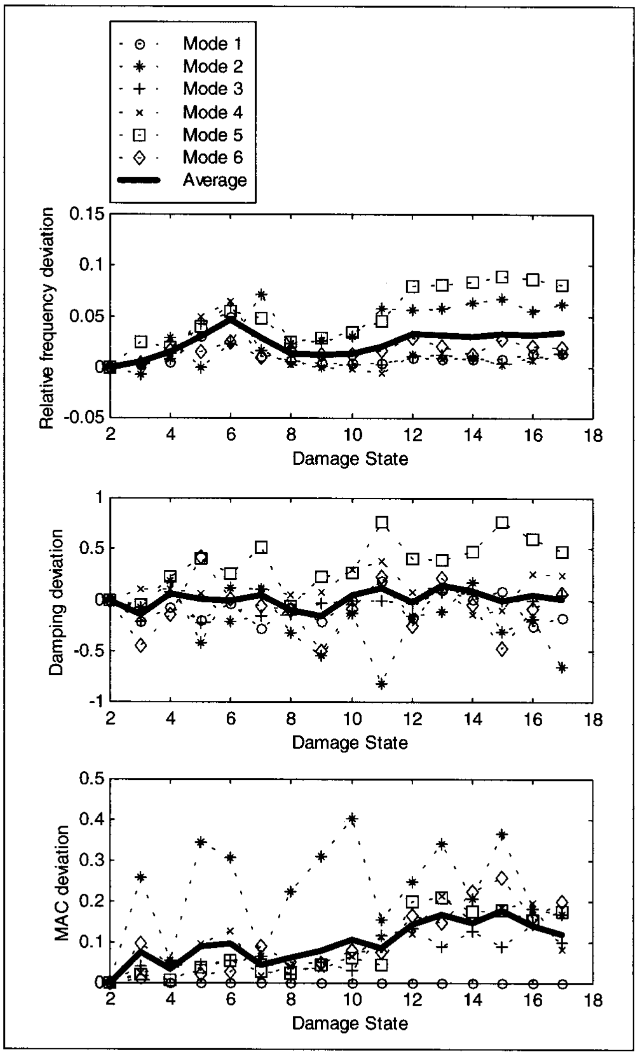

Fig. 8: **Monitoring variables for the 1D case**

including 8 degrees of freedom. Natural frequencies and damping ratios were practically the same for the 3D, 2D and the 1D case. However, somewhat more scatter was present in the damping estimates for the 1D case.

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 however temperatures are measured, and if a reliable data base is established making it possible to filter out the influence of temperature shifts and other environmental changes on the natural frequencies, then it is believed that frequency deviation should be the main tool for health monitoring.

Level 1 damage detection based on the 3D, 2D and the 1D case showed that there is only a very small difference between the results of the three cases. Only for the damping that showed an increase with damage state for cases 3D and 2D, the picture became somewhat unclear in the 1D case. However, since damping is not in general considered an important parameter for monitoring and damage detection, this weakness is considered to be of minor importance.

Thus, when the health of structure is to be monitored over a long period of time, as a main rule, it is concluded that the monitoring can be based on a relatively small number of sensors. The number of sensors should be just large enough to ensure that all desired modes can be identified safely. In practice one has to include some redundancy allowing that the failure of one or two sensors still will make it possible to perform the identifications using only the reduced number of channels. In any case it is believed—as it is illustrated in this investigation—that a realistic monitoring program can be carried out with a rather limited number of sensors.

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