COST action F3 on structural dynamics: benchmarks for structural health monitoring

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Abstract

This paper is concerned with the results from the COST Action F3 Working Group Two benchmarking exercise in Structural Health Monitoring. Data from two large-scale structures were modelled for the purposes of damage detection, location and quantification. Several analysis papers have been submitted for a special issue of *Mechanical Systems and Signal Processing* and the conclusions of each are summarised here, together with more general conclusions arising from the concerted effort.

1 Introduction: COST Action F3

This paper is concerned with the benchmarking activities of Working Group Two (WG2) of the COST Action on *Structural Health Monitoring*. In fact, the initial remit of WG2 was *Health Monitoring, Damage Detection and Force Identification*, but the final subject received no attention as SHM proved challenging enough. In more detail, WG2 was convened to investigate [1];

> 'health monitoring of structural systems (damage detection and localization in structures; safety assessment of the residual dynamics of potentially damaged structures; monitoring of buildings during and after earthquakes; health monitoring of pipeline joints from their vibration signatures; in-situ monitoring and diagnosis of mechanical equipments, rotating machinery etc.)'

As the reader will see, some of these objectives were pursued in detail while others were discarded. This should be no surprise as SHM is a rapidly evolving subject and it would have been anticipated that the focus of the action would change with time. Before discussing the benchmark papers and seeing what was actually addressed and what was accomplished, it is important to discuss the problems of SHM and how the WG2 benchmarks were positioned to engage with them.

2 Structural Health Monitoring

The problem of on-line structural health monitoring (SHM) of aircraft is a notoriously difficult one. However, the motivation for pursuing research in the field is strong; integrated SHM technology may well allow significant reductions in the cost of ownership of aircraft, both military and civil and also of civil infrastructure. There has never been a shortage of research activity in the field; however, progress has been slow and incremental and full-scale tests have been few and far between. The effort tends to be concentrated on computer simulations and relatively simple laboratory structures. The work on the WG2 benchmarks makes a substantial effort to tailor and adapt theoretical work to real structures specified and created elsewhere for full-scale test.

It is useful to think of the SHM problem in terms of a hierarchical structure, perhaps the most well-known framework is that of Rytter [2].

Level 1. (DETECTION) The method gives a qualitative indication that damage might be present in the structure.

Level 2. (LOCALISATION) The method gives information about the probable position of the damage.

Level 3. (ASSESSMENT) The method gives an estimate of the extent of the damage.

Level 4. (PREDICTION) The method offers information about the safety of the structure e.g. estimates a residual life.

Each level is more difficult than the previous. Although results have been presented supporting the existence of methods working up to level 3, they are usually based on systems and structures of low complexity and are far from convincing aerospace or civil industry of their applicability in the field. Level 4 is the most difficult of all and the problems there have simply not been addressed.

The first problem (Level 1) is of damage detection and can be posed in terms of *novelty detection*. This problem is essentially to distinguish between the normal operation of a system or structure and any anomalous conditions which may be symptomatic of damage. The benchmark papers illustrate a number of approaches.

Level 1 is distinguished by the fact that it can be accomplished without a 'model' in the usual structural dynamical use of the term. A model of some sort is needed in order to pose a template from which deviations can be measured. However this can be a static statistical representation with no predictive powers i.e. a probability distribution for a set of features measured from the normal condition of a system. This is because it is feasible to derive data from real structures in normal condition. It is not economically viable to base a diagnostic for Level 2 or above on a data-driven model. Consider the problem of damage location; an empirical model would require the existence of copies of the structure in each conceivable damage scenario in order to obtain data to train a diagnostic.

This means that Level 2 or 3 SHM requires the existence of an accurate 'physical' model for the structure. Further, it should be possible to accurately model the progression of damage through whatever mechanism i.e. a 'crack' in a model of a metallic structure should look like a real crack and given a location (crack tip and perhaps orientation) and a severity (crack length), the model should closely predict the required measurements. This is what makes damage localisation and quantification difficult. However, given the existence of the model, there are essentially two ways of making use of it. The first is the *inverse* sense and the second is the *forward* sense.

In the inverse sense, an updated physical model of the structure is obtained by matching derived features (natural frequencies, modeshapes, FRFs etc.) from the model to those observed by experiment. During the monitoring phase, measurements of the same features are periodically used for further updating. If the updating shows that the model now deviates from experiment, one has detected damage (with proper concern for statistical significance). The location of the updated elements of the model places the damage and the extent of the update required fixes the severity.

In the forward or *pattern recognition* sense, the model is used to construct features corresponding to all possible damage scenarios. One then trains a classifier (statistical, syntactic, neural [3]) which associates a diagnosis (location, extent) with each set of features. Alternatively, if a continuum of damage states is possible, one trains a *regressor* in the same way. In either case, during the monitoring phase the observed features are passed to the classifier or regressor for a diagnosis.

There are examples of both approaches in the papers which address the benchmarks. Before discussing them, a small digression will describe the benchmarking exercise.

3 The WG2 Benchmarks

The choice of the WG2 benchmarks was driven by pragmatism and economics. Although the COST action provided generous finance for travel, for organisation of meetings and for Short Term Scientific Missions, it did not provide finance for core research or the physical construction of benchmarks. While this did not prove an insurmountable problem for WG3 where much could be learnt from small rigs, it did raise issues with working groups 1 and 2 where full-scale test data was required. In the end the two groups were granted valuable data from projects funded elsewhere. In the first case from the *STEELQUAKE* structure constructed and tested at the European Laboratory for Structural Assessment (ELSA) at the Joint Research Centre (JRC) at Ispra, Italy. In the second case from the Z24 bridge in Switzerland tested as part of the SIMCES project funded by Brite Euram.

Both of the structures are very large constructions, typical of Civil Infrastructure. This inevitably meant a Civil bias for the WG2 benchmarks, but as indicated above this was the result of pragmatism. (A smaller laboratory specimen - a concrete beam - formed the basis of an early benchmark activity; however, this did not prove excessively challenging and the activity was considered closed. The reports on this can be found in the proceedings of a special session of the 1999 COST-supported conference *Identification of Engineering Systems II*, Swansea, 1999 [4].) In the early stages of the COST action there was much concern about the fact that rotating machinery might be ignored, but the cost of commissioning an appropriate rig was found to be prohibitive. However, even given the limitations of coverage, the benchmarks allowed a number of important issues in SHM to be addressed, and this will be made clear by the discussion of the papers later.

3.1 The STEELQUAKE Structure

The STEELQUAKE project was aimed to assess steel building performance and included cyclic and seismic tests on a large-size specimen at the European Laboratory for Structural Assessment (ELSA) [5]. Taking advantage of its availability at one of the laboratories within COST F3, the two-storey frame depicted in Figure 1 was proposed as a benchmark for damage detection to the participants of Working Group 2.

The definition of the measurement set-up and the experimental modal analysis were performed on the undamaged structure by R. Pascual during his short term scientific mission at ELSA-JRC in July 1998. After damage testing of the structure, a new modal analysis was conducted by J. Molina. All the data were collected and made available to the COST F3 WG2 participants [6].

3.1.1 Description of the Structure

The main dimensions of the structure are 8m by 9m by 3m. The storeys are made up of corrugated steel

Figure 1: The STEELQUAKE structure.

sheets covered by a reinforced concrete slab, forming a composite deck with orthotropic elastic properties (Figure 2). The two storeys are connected by welded steel columns and beams as shown in Figure 3. For stability reasons during the damaging tests, the transversal frames are stiffened by cross bracing in the plane parallel to the wall as can be seen in Figure 1. In the background of the picture (Figure 1), can be observed the reaction wall which supports the four pistons (not present in the picture) that allow deformation of and damage to the structure (on each frame, on each storey). The columns are made of HE300B profiles and the beams of IPE400 on the length and IPE300 on the width. Bracing is made of L60x30x5 profiles.

3.1.2 The Measurement Set-up

Modal testing using hammer excitation was performed on the undamaged and damaged structure. Fifteen accelerometers were mounted on the structure using magnets. The location of the 15 sensors was determined using the Effective Independence (EfI) method described in detail in [7]. This technique requires as input analytical eigenmodes that were com-

Figure 2: Detail of the stories.

Figure 3: Detail of a joint - first storey (internal view).

puted using the finite element program SAMCEF [8]. Basically, the sensors were placed at the corners of the storeys to catch the deformation of the columns and four of them were located in the middle of the beams to catch the bending and torsion modes of the slabs. Impact excitation was produced at four locations. For each test, eight to ten hammer impacts were produced and the time responses of each sensor were recorded. Frequency response functions (FRF) were

Figure 4: Example of a crack at a beam-to-column joint.

extracted with a resolution of 1024 points in the frequency range from 2 to 34 Hz and the experimental modal analysis was performed using the LMS Cada-X software [9].

After damage testing of the structure, cracks like the one shown in Figure 4 occurred at different locations. That type of crack developed from the welding of the longitudinal beams to the columns and affected one flange of the beam and half of its web. Even though plastic deformation was evident at every one of these joints, only three of them showed such a crack as reported in Table 1. There, the four joints at every floor are distinguished according to their position in the laboratory. Looking at Figure 1, the structure was oriented longitudinally in the E-W direction and the reaction wall (at the back in that picture) corresponded to the W side.

W (Wall Side)	E	2^{nd} Floor
-	**CRACK**	N
-	-	S
W (Wall Side)	E	1^{st} Floor
CRACK	-	N
-	**CRACK**	S

Table 1: Location of cracked beam-to-column joints

The results of the modal identification on the intact and damaged structures are listed in Table 2.

3.2 The Z24 Bridge

In the framework of Brite Euram project BE96-3157 SIMCES (System Identification to Monitor Civil Engineering Structures), a series of damage scenar-

Type of mode	Intact	Damaged	MAC
Bending 1X	3.13	2.68	0.97
Bending 1Y	3.93	3.87	0.93
Torsion 1	6.13	6.06	0.98
Bending 2Y	9.69	9.52	0.99
Bending 2X	10.82	9.90	0.69
2^{nd} floor/bending	12.27	10.69	0.78
1^{st} floor/bending	13.05	11.29	0.95
2^{nd} floor/torsion	17.70	15.09	0.94
1^{st} floor/torsion	19.03	16.19	0.95
Torsion 2	21.41	18.83	0.98

Table 2: Experimental frequencies.

Figure 5: View of Z24 bridge.

ios were applied to a prestressed concrete bridge in Switzerland to detect, localise and quantify artificially applied damage. The test set-up and damage scenarios are described here. The analysis procedure and experimental validation results of direct stiffness calculation technique were reported and discussed in [10].

3.2.1 Bridge Description

The bridge used was bridge Z24 in Canton Bern, Switzerland, connecting Koppigen and Utzenstorf. The bridge is a highway overpass of the A1, linking Bern and Zurich (Figure 5). Z24 is a prestressed bridge, with three spans, two lanes and 60m overall length. The geometry is shown in Figure 6.

3.2.2 Damage Scenarios

Within the SIMCES project a series of progressive damage tests were carried out during the summer of 1998. For a full description of all damage scenarios, instrumentation and safety considerations, the reader

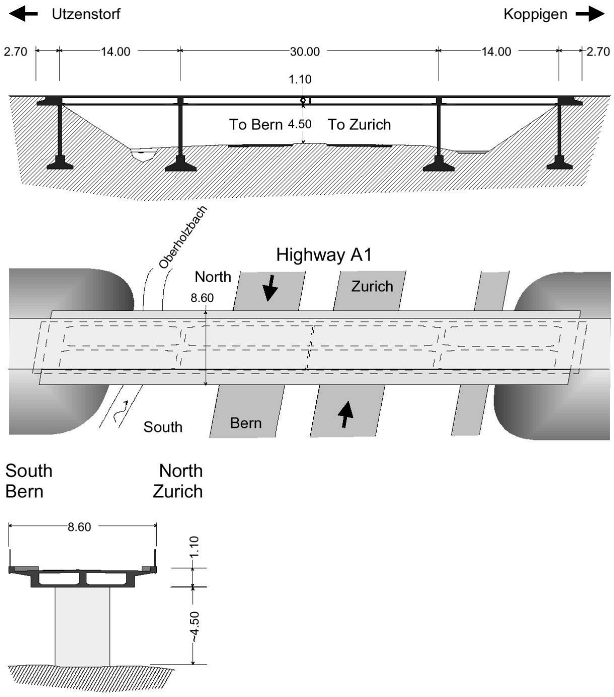

Figure 6: Top view, cross section, elevation (Kramer et al. [11]).

is referred to [11]. The first 8 scenarios are summarised in Table 1.

#	Date	Scenario
1	04.08.98	1st reference measurement
2	09.08.98	2nd reference measurement
3	10.08.98	settlement of pier, 20mm
4	12.08.98	settlement of pier, 40mm
5	17.08.98	settlement of pier, 80mm
6	18.08.98	settlement of pier, 95mm
7	19.08.98	tilt of foundation
8	20.08.98	3rd reference measurement

Table 3: Damage scenarios on Z24

The settlement is simulated by cutting the Koppigen pier and removing about 0.4m of concrete. Lowering and lifting was done by 6 hydraulic jacks. During the tests the pier rested on steel sections with a similar stiffness as the uncut concrete section.

Other damage scenarios (spalling of concrete, landslide, cut of concrete hinges, failure of anchor heads, rupture of tendons) are not considered here as they caused no or a minor degradation of bending stiffness.

3.2.3 Initial Analysis Procedure

The experimental eigenfrequencies for the first five modes are summarised in Table 2 for the different reference measurements and damage scenarios. Processing of the measurements was done by the stochastic subspace identification method [12]. Comparison of identified eigenfrequencies and damping ratios from three excitation sources, i.e. band-limited noise from shaker, impact from a drop weight and ambient sources are reported in [13].

	Mode				
Case	1	2	3	4	5
1	3.92	5.12	9.93	10.52	12.69
2	3.89	5.02	9.80	10.30	12.67
3	3.87	5.06	9.80	10.33	12.77
4	3.86	4.93	9.74	10.25	12.48
5	3.76	5.01	9.37	9.90	12.18
6	3.67	4.95	9.21	9.69	12.03
7	3.84	4.67	9.69	10.14	12.11
8	3.86	4.90	9.73	10.30	12.43

Table 4: Eigenfrequencies for Z24 Bridge.

The first mode is a symmetric bending mode. The second mode is a lateral mode. Third and fourth modes are antisymmetric bending modes with torsion of the midspan. The fifth mode is a symmetrical bending mode with highest modal displacements at the sidespans.

4 Summary

Before coming to a series of conclusions about the benchmark exercise, it is useful to look back over the individual studies and summarise the salient points.

In total, there were nine papers; five for the STEELQUAKE structure and four for the Z24 Bridge.

4.1 The STEELQUAKE Structure

The first paper by Bodeux and Golinval [15] set the scene in modelling terms. As the excitation is usually unavailable in the testing of large-scale Civil structures, output-only methods were adopted. Two approaches were tried, the first was an ARMAV approach with a prediction error parameter estimator and the second was a data-driven Stochastic Subspace Identification approach (SSI). Both used only response measurements, but both assumed that the excitations were uncorrelated white noises. (With a further assumption that the excitations are Gaussian, it is possible to estimate confidence intervals for the parameters.) The underlying model here is a modal model and the estimated parameters are natural frequencies, modeshapes and damping ratios. In terms of the natural frequencies, both the ARMAV and SSI gave very precise results which agreed with each other very well. The damping estimates showed significantly more variation - which is no surprise. Damage identification here was level 1. Damage was assumed if the confidence intervals on the natural frequencies for the 'damaged' structure were disjoint from those of the undamaged structure. The precision on the frequency estimates was such that damage was detected without difficulty. The authors pointed out that false positives may arise as a result of natural frequency changes due to environmental variations and indicated a possible way of avoiding this.

The second STEELQUAKE paper was by Mevel *et al* [16]. The authors also used an output-only stochastic subspace algorithm in order to estimate a state-space model. The features for damage detection were the residuals when the measured time data were presented to the model. A χ^2 statistic was then computed to give a scalar novelty measure. The authors applied a variant of the SSI algorithm which allowed multiple measurements from different tests to be used in forming the model. The detection results were clear, the χ^2 statistic for the damaged structure was three orders of magnitude greater than that for the undamaged structure. There was no localisation information - the method was level 1.

A paper by Görl and Link was the first to follow an FE-based approach. As the first to use a physical model it was the first with localisation and quantification potential. A fairly detailed model was employed which took into account shear deformation in the beams and modelled the floors as orthotropic structures. The initial model was updated using test data from the undamaged structure, this gave a peak percentage error of 2% over the first ten natural frequencies and the lowest MAC was 97.28. The choice of parameters to update was chosen on the basis of sensitivity analysis and a weighted least-squares approach was used to minimise an objective function constructed from frequency and modeshape residuals. The authors noted that the braces presented difficulties for the modelling but seem to have overcome this adequately. After updating, the FE model reproduced the first ten natural frequencies to within 2% again, but with a minimum MAC of 80.81. The dominant parameter changes were for the joints of the long beams. This is entirely consistent with the hinge effect induced by the cracks. The update also showed significant changes associated with the bending and torsional stiffness of the slabs, but this was not evident from visual inspection. The method used is potentially level 3 in that the parameters for updating indicate the position of the damage and the extent of the update shows the extent of the stiffness reduction or damage.

The second FE study was provided by Fritzen and Bohle [18]. This was also an updating strategy with the potential to reach level 3 in Rytter's hierarchy. A detailed model was fitted, again with shear beams and orthotropic plates. The data used for updating were natural frequencies and real normal modes. The parameters were chosen for updating by a process of subset selection. The paper actually compared four approaches to the problem: the Inverse Eigensensitivity Method (IESM), the Modal Force Residual Method (MFRM), a Pseudo-Static Method (PSM) and a Minimisation of the Errors in the Constitutive Equations (MECE) approach. All the methods gave acceptable localisation results, finding errors in the immediate locale of the three main damage sites. A false positive was also found at a first story joint and that may have been associated with plastic deformation, but no crack was found there. The extent of the updates showed that the residual stiffnesses at the joints were in the range 5-10%, consistent with the very large cracks observed. It is noteworthy that this approach (and the last [17]) cope quite happily with multiple damage sites.

The final study of the STEELQUAKE structure by Zapico *et al* [19] was also FE-model based, but with a rather different philosophy. This was the only paper which followed the forward - pattern recognition - approach to the damage assessment. The FE model was rather simpler than those in the previous two papers, using Timoshenko bar elements and distributed masses for the slabs. Only two parameters were chosen for updating and they were adjusted using a Multi-Layer Perceptron (MLP) Neural Network. The network was trained to output the updated parameters corresponding to a given set of observed natural frequencies. After updating, the elements of the stiffness matrix of the structure were found to be accurate to 0.4%. For damage assessment a further MLP was trained to give the overall damage parameters of each floor when presented with the damaged natural frequencies. The results were good on the experimental data with the network producing near zero for the undamaged floors and near unity when damaged. The stiffness matrix for the damaged structure computed on the basis of the predicted damage parameters was accurate to 8%. This was considered acceptable as an antisymmetric FE model had been used and the damage broke the antisymmetry.

4.2 The Z24 Bridge

The first study on the Z24 Bridge was that of Maeck and De Roeck [20]. As all of the papers did, it used vibrational features in order to signal damage, but in a rather different way. A damage index was constructed on the basis of modeshape curvatures with reduced stiffness at a given location producing an elevated curvature. There have been many applications of this type of approach, but they usually estimate the curvatures by a differencing process on the modeshapes which is unreliable for low sensor densities. This paper estimated the curvatures by using a penalised optimisation procedure. The penalty functions were parametrised by two constants which must be prespecified; however, they were chosen in the paper by an error criterion on individual modeshapes. The pier settlement of 80mm was easily detected and located to within the correct span by the method. Further, the index returned to its undamaged values when the settlements were reversed and the cracks closed - a fundamental requirement of any detection method. The authors observed that the natural frequencies were sensitive to the damage also.

The second paper was by the INRIA/IRISA group with Mevel *et al* [21]. For detection, the SSI approach with a χ^2 damage index was used in the form suited to multiple measurement sets. Localisation was obtained by correlating model sensitivities with a FE-model although a detailed discussion was not given. Despite the fact that the natural frequencies were rather insensitive to damage, the χ^2 statistic showed clearly the 20mm settlement - the statistic increased by three orders of magnitude over the undamaged case. The authors pre-checked the data for changes in the nature of the excitation (traffic) in order to avoid false alarms. The localisation procedure was confirmed by the visual inspection - a crack opening by 2mm was found. Because certain options were only open to the authors in an off-line analysis, they also conducted an on-line study which proved successful. Changes in the structure as a result of operational and environmental variation were cited as a problem.

A paper by Garibaldi *et al* [22] also exploited a stochastic state-space technique, namely Canonical Variate Analysis (CVA). After considering other methods, including ARMAV, this was selected because it required least user interaction. As in all the papers concerned with SSI methods, model order was an issue, the usual means of selection is by an appeal to stabilisation diagrams, but two other approaches were considered here which gave equivalent results. This paper was distinct from the others in its consideration of the different excitation tests carried out on the Z24 bridge. In addition to the ambient excitation tests, the authors analysed data from shaker and drop weight tests. The agreement in the model natural frequencies between the tests was impressive. An updated shell FE model for normal condition was obtained by using a sensitivity method. The authors pointed out that the variations in the natural frequencies due to temperature change were of the same order as the variations due to damage, but concluded 'modal parameter variations due to damage are detectable, despite the temperature effect'. In fact, the main thrust of this paper was not to detect damage as such but to establish a high-fidelity model of normal condition.

The final paper on the Z24 bridge was by Kullaa [23] and was different in a number of ways. Although the features for damage detection were once more natural frequencies and modeshapes extracted from time-series by a stochastic subspace method, the method of analysis was different. A novelty analysis based on Multivariate Statistical Process Control (MSPC) was used. This paper shows the most explicit use of the language of pattern recognition. The process of feature extraction here produces patterns with too high a dimension, so subset selection (natural frequencies only) and Principal Component Analysis (PCA) were used to reduce the dimension. The paper discussed the use of a number of control charts, which are essentially a form of outlier analysis signaling when a feature departs from normal condition. A univariate analysis was carried out using the first principal component from four natural frequencies and modeshapes and this proved to give less false positives. The multivariate analysis on four natural frequencies was more sensitive to damage but led to more false positives. Even the lowest pier settlement was detected. An on-line analysis was also conducted and the results were excellent.

5 Discussion

A number of observations can be made on the basis of the analysis above. First of all, all the participants in the benchmarks fitted a traditional structural dynamical model in the course of their analysis, either an FE model or a state-space modal model. It is interesting that of the 6 participants using the latter, all used a stochastic subspace algorithm (or close relative). This indicates clearly what the state-of-the-art is considered to be across Europe. Model order was, with one exception, chosen by an appeal to stabilisation diagrams. The evidence suggests that the technique is perfectly adequate for the type of structures considered here, even in the taxing output only case. A positive result of the survey is the conclusion that ambient excitation gives adequate models. This is critical for many of the large structures which form Civil infrastructure. Having said this, one of the authors showed that controlled excitations: shakers and drop weight, also gave excellent results.

Regarding the types of analysis. Three of the authors used an updating approach or inverse approach and one other made an appeal to a FE model in order to localise. The localisation effort was largely successful for both the STEELQUAKE and Z24 cases. One paper adopted a curvature-based damage index for the Z24 data and was able to localise the damage satisfactorily. The remaining authors all carried out some implicit or explicit novelty analysis which gave them the capability of detecting the damage. The most principled approaches, based on statistical analysis proved the most sensitive i.e. detecting the lowest level of settlement in the Z24 case. In terms of quantification, only the FE updaters made real progress although it should be remembered that a stiffness parameter reduction is not a physical characterisation of the damage; none of the methods produced a crack length. Only one of the papers made use of a mainstream machine learning technique, namely, a neural network. Although the networks proved successful in both damage detection and localisation, there are issues raised here about the use of 'black-box' methods in potentially safety-critical applications.

Several authors raised the question of environmental effects. If variations in temperature cause similar levels of variation in features as damage, it is necessary to project out the effects of temperature from the features or select features insensitive to environmental conditions. Operational conditions also matter. One author pre-checked data to ensure that the excitation statistics were meaningful i.e. that the traffic conditions were appropriate, before giving a diagnosis. This is a serious issue and is not only pertinent for the sort of large-scale steel and concrete structures considered here. If these techniques are to work on say, aircraft structures they must work when the aircraft is on the ground in desert conditions and also when the aircraft is in flight - the potential temperature range is 100 Centigrade.

Having raised the problem of damage detection in aircraft, one should examine exactly what has been established here. Vibration-based methods have proved acceptable for detecting large damage in large structures. The global nature of vibration features, which allows progress here, may inhibit progress in smaller structures where critical defects may be very small. It is likely that in such a situation, higher-frequency methods will be required.

6 Conclusions

There is no need for lengthy conclusions. The WG2 benchmarks were selected in a pragmatic fashion which limited their coverage of Engineering application domains. Within the domain of interest - large steel and concrete structures - the analysis shows that SHM techniques are adequate for the detection and localisation of large damages. Given that the subject of the action is structural dynamics, this is all that one could have hoped for, as SHM for smaller structures: aircraft, ground transportation and rotating machinery is likely to require the use of higher-frequency tools. The only other conclusion one should draw is that the COST F3 action has been of inestimable value in allowing this benchmarking exercise - much has been learnt.

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