# Network of European Research Infrastructures for Earthquake Risk Assessment and Mitigation

# Report

## Current status report - Inventory of field testing infrastructures and existing approaches

| | |
|---|---|
| Activity: | *COORD* |
| Activity number: | *NA6: Networking field testing infrastructures – Task 6.1* |
| Deliverable: | *Current status report - Inventory of field testing infrastructures and existing approaches* |
| Deliverable number: | *D6.1* |
| Responsible activity leader: | *AIT* |
| Responsible participant: | *AUTH, EUCENTRE, KU LEUVEN, KOERI* |
| Author: | *Herbert Friedl* |

**Seventh Framework Programme**
**EC project number: 262330**

Please use the first page of this template for the cover of your deliverable report.

**Deliverable Contributors**

| | |
|---|---|
| AIT – Austrian Institute of Technology | Herbert Friedl<br>Marian Ralbovsky<br>Thomas Öberseder<br>Rainer Flesch |
| AUTH – Laboratory of Geotechnical Earthquake Engineering of the Department of Civil Engineering of Aristotle University of Thessaloniki | Kyriazis Pitilakis |
| EUCENTRE - European Centre for Training and Research in Earthquake Engineering | Simone Peloso<br>Ricardo Monteiro |
| KOERI - Kandilli Observatory and Earthquake Research Institute | Can Zulfikar<br>Yavuz Kaya |
| KUL – Katholieke Universiteit Leuven | Guido De Roeck<br>TienThanh Bui |

## Table of Content

## List of Figures

## List of Tables

# 1 Summary

The main objective of Work Package NA6 is to harness the European strength in vibration monitoring and field testing to establish a network of research organisations and practitioners and a world leading field testing capacity with standardized approaches, with the overall aim to enhance the research on structural safety analysis and promote its implementation in building practice. One major goal was to identify key players and stakeholders and collect the state of the art by carrying out an intensive literature review.

In chapter 6 a number of relevant references and summary about the current state of the art regarding field testing and vulnerability assessment is given. Chapter 3 deals with specific field testing capabilities:

- Field testing vs. Laboratory testing
- Wireless Sensor Technologies
- Soil structure interaction
- Structural identification

The elaboration of an online questionnaire tool allowed on the one hand to get in contact with identified key players in the targeted area and on the other hand to receive valuable feedback about ongoing research on "Field Testing for Earthquake Engineering". The survey was designed to collect applications and performance of current approaches in field testing. The survey encompassed 65 questions regarding field testing like new developments, used approaches and applications, international standardization, and asked for further needs regarding training and guidelines. By using contacts from European and international organisations (EERI, IAEA, EAEE, ASCE, IABSE) a worldwide distribution of the survey in over 60 countries was assured. The first answers and feedbacks were very informative and signalised the high interest of the community. A detailed analysis of this survey is given in chapter 4.

Further developments within NA6 were to prepare several case studies (see chapter 5). The goal of the chosen case studies is to demonstrate the approach and highlight the benefits from field testing. The following case studies were selected and prepared in chapter 5:

**1.) Forced vibration testing of bridges (AIT)**

The structure is an arch bridge with total length of 157 m. Two arches with 69 m span and 18.3 m height are made of Ultra-High Performance Fibre-Reinforced Concrete UHPFRC 165/185. Each arch consists of 5 straight segments connected with joint segments. The elements were prefabricated, mounted on site and post-tensioned. The measurement was carried out as forced vibration test. Excitation was produced by a hydraulic mass reaction exciter.

**2.) Industrial building – reinforced concrete structure (AIT)**

Vibration test of a RC industrial building is presented. Forced vibration test was carried out as primary test and ambient vibration test as complementary experiment. Aim of vibration testing was to acquire real dynamic structural properties for more accurate earthquake assessment. Modal properties identified from measurements were used to validate or update a computational model of the building that would be used for earthquake assessment.

**3.) SDOF system – Soil structure interaction effect (AUTH)**

Numerical modeling of the tests (free and forced vibration) was performed on the model structure in Euroseis test experimental site. Different codes were used (ABAQUS and ANSYS). The main focus was on the different modeling configurations and details in 2-D and 3-D for both the structure and the soil-foundation system.

This study will concentrate on the following tasks:

- Performance of two sets of free tests with increasing level of loading and frequency excitation
- Preparatory studies and tests for the forced vibration testing to be performed in the coming weeks
- Finalization of the laboratory geotechnical tests to derive the dynamic and mechanical properties of the foundation soil materials
- Analysis of the experimental data and comparison with the numerical results

**4.) SDOF System – Benchmarking of different test methods (AUTH)**

The prototype structure was designed to be excited by "Pull-out" and "Forced Vibration" tests. The forced vibration tests have not been completed yet; they are going to be performed by a shaker mounted on the roof and on the foundation slab, with different oscillation configuration in order to promote swaying and/or rocking. The level of the shaking – needed to promote the SFSI - is still to be determined from parametric numerical analyses performed by AUTH and the characteristics of the available shakers to be provided by ITSAK.

**5.) Sanctuary of Vicoforte (EUCENTRE)**

The Sanctuary of Vicoforte is a monumental church located in Vicoforte, a small town close to Cuneo in the north-west of Italy. It is known for having the largest elliptical dome in Europe. Recently, the Sanctuary has been undergoing a delicate renovation process including interventions on the main dome and on some other structural parts. Dynamic identification tests were used to characterise the structure, its behaviour under wind excitation and to have a benchmark to compare to after the execution of the retrofit interventions.

**6.) Malpensa airport (EUCENTRE)**

The Malpensa Airport, the main airport of Milan (Italy), is currently undergoing an enlargement phase including the construction of new buildings and connection paths between them. Within this framework, the designers asked for the execution of dynamic identification tests on different structures for the validation of the finite element models used for the design. The experimental activities were performed using different excitations - ambient and forced vibrations - for the structures under investigation

**7.) Building structure (KU Leuven)**

Floor vibration tests of two buildings are carried out. Modal identification of the floor under both forced and ambient excitation is the main focus of this investigation. In one building the measurement campaign is planned in two phases. The first phase is to test the floor while the building is under construction. The second phase is when the construction is completed. The main objective is to study the changes of natural frequencies and damping values. In the other building, the purpose of the test is to characterize the dynamic behaviour of a light floor.

**8.) Bridge structure (KU Leuven)**

As illustrative and representative examples for vibration based monitoring, two complete field tests were carried out on two rail bridges in Spain. One is an old, multi-span continuous truss bridge of the rail line between Seville and Alcázar and the other is a recently-built multi-span continuous box girder pre-stressed concrete bridge on the high-speed train line between Madrid and Barcelona. The purpose of this test campaign is to have a baseline dynamic reference of the structures so that a long term SHM system can be designed. The tests were performed to identify natural frequencies, mode shapes and modal damping factors due to ambient (natural) excitation.

**9.) Residential Apartment Buildings (KOERI)**

The recordings from two residential buildings (one with infill walls and one without) in Istanbul will be studied and compared. ARTEMIS and KOERI's in-house software (some subroutines from KOERI-MIDS) will be used for data analysis.

**10.) Suspension bridge (KOERI)**

The ambient vibration testing recordings on the Second Bosphorus Bridge have already been studied and compared with the analytical results. Currently, these recordings are re-evaluated in a Matlab Code written by KOERI (some subroutines from KOERI-MIDS) and its results will be compared with the analytical results and past studies.

# 2 Introduction

For the purposes of this research, field testing is defined as the complete range of activities in this topic, like building or bridge structural response/performance studies, Soil-Foundation-Structure Interaction (SFSI) studies or response/performance studies for geo-structures or soil deposits. The aim of this report is to collect existing knowledge about field testing equipment and techniques for geotechnical and structural earthquake engineering experimentation.

Understanding structural response to earthquake loading is essential for the design of new structures and the assessment and seismic upgrading of existing structures. Further, the prediction of soil liquefaction and subsidence, lateral spreading, and site amplification is based on small-strain laboratory and field testing, which do not duplicate the nonlinear strain levels that occur during major earthquakes. For analysing the response of near-surface geological layers to earthquake loading mobile, large-scale field equipment can be used to produce dynamic motion in the ground comparable to that produced during earthquakes, so the loading conditions that structures are designed for.

Field experiments will significantly enhance the fundamental knowledge of earthquake effects associated with the behaviour of structures and geological layers, thereby reducing loss-of-life and economic losses from future earthquakes.

## *2.1 The role of field testing methods for structures*

In-situ investigations are in general necessary not only for earthquake assessment, but also for health monitoring and evaluation of the existing condition of the structure. In principle it is possible to measure the following physical parameters:

- Strains
- Displacements, velocities, accelerations (transverse and rotational components, even if measurement of rotational components is less easy)

From measured time histories the dynamic properties (modal parameters) can be obtained. Dynamic field tests or in-situ testing methods are necessary for the evaluation of the dynamic behaviour (modal parameters) and the actual condition of existing structures. In-situ measurements are recommended in order to verify the numerical models and to increase the reliability of the numerical approaches. A first FE-model of the tested structure, which is elaborated on the basis of the design documents, can be fitted to measured results by an optimisation approach. This procedure is called "model updating". Dynamic in-situ measurements are also recommended for the verification of the effects of structural changes and strengthening applied to buildings.

By means of experimental modal analysis the identification of the modal parameters (natural frequencies, mode shapes, and damping ratios) of a structure from input and/or output measurements is possible. For this purpose, the mode shapes of a structure must be excited measurably. In the case of input/output measurements also the frequency dependent impedance (force vibration velocity ratio) is obtained. The modal parameters are used to perform ongoing earthquake analysis or assessment. Since the updated model is based on measured results, it represents in a realistic manner the behaviour of the structure during the starting phase of a seismic event. Using this model for an earthquake analysis it is possible to forecast and therefore to point out the weak spots in structural members, if any.

To measure the vibration response of a structure, a sufficient excitation should be available. This excitation is basically possible in two different ways, on the one hand by forced excitation and on the other by ambient excitation.

# 3 Overview about field testing capabilities

## *3.1 Field testing vs. Laboratory tests*

In the following, a comparison between the application of the dynamic identification (DI) techniques within a laboratory testing campaign or as part of an on-site investigation is reported. An attempt is done at underlining differences in these experimental techniques arising from the test location. Furthermore, common points and criticalities will be discussed too.

When performing structural dynamic identification tests, the main goal is clearly the evaluation of the dynamic characteristics (natural periods/frequencies, mode shapes, damping values) of the structure under inspection. Utilization of the DI results, however, may differ between lab and on-site tests (see Table 1).

Table 1. Possible use of results obtained from DI tests

| **Laboratory testing** | **On-site testing** |
|---|---|
| ▪ Evaluation of the performances of structures realized with novel technologies | ▪ Assessment of the performances of existing structures |
| ▪ Evaluation of the effectiveness of a newly developed retrofit technique | ▪ Validation of numerical models used for the structural design |
| ▪ Validation of numerical models for general purposes (e.g. experimental data interpretation), possibly using model updating techniques | ▪ Validation of refined numerical models representing the actual condition of the structure, possibly using model updating techniques |
| | ▪ Evaluation of those aspects related to the limitation of use of structures due to oscillatory/dynamic movements |
| | ▪ Estimation of the effectiveness of a retrofit or upgrading intervention |
| | ▪ Structural Health Monitoring |

Most of the time a dynamic identification is performed as part of a laboratory testing campaign, the final scope is the tuning of a finite element (FE) model representing the specimen. The numerical model can then be used for several reasons that will not be discussed within this framework. The execution of multiple DI after different phases of testing is of particular relevance in the laboratory context since it allows to evaluate possible structural modifications. This is best achieved when combining the identified dynamic parameters with model updating procedures. This is a very useful approach when investigating the performances of new construction or retrofit techniques.

On the other side, performing dynamic identification on-site gives the investigator the opportunity to reach several targets. The modal characteristic detected by the DI can be used as a starting point towards the final goal of assessing the performances of an existing structure: these parameters are the basis for the estimation of the effects of a seismic induced excitation. Furthermore, also in the case of on-site tests, the results of

the DI can be used together with the modal updating techniques to tune an FE model. The reasons for this operation can be various, amongst them: (i) trying to validate a numerical model used for design; (ii) have a reliable model representing the structure in its actual condition, clearly including soil-structure interaction. In both the previously mentioned cases, a correct model of the structure is the first step towards the final goal of performance assessment.

Performing a DI on a structure before and after an event allows to estimate the change in structural characteristics. It is worth mentioning that it does not matter whether the structure has been improved or weakened by an external event; DI remains an effective tool to evaluate the structural modifications. As an example, it is possible to evaluate the effectiveness of interventions aiming at the structural retrofit or at the seismic upgrade and involving changes in the stiffness of the structure (e.g. geometric modification of the structural elements or inclusion of new ones). On the other hand, another example of such a type of situation is a DI performed prior to and after a seismic event able to damage the structure. In this case, through the evaluation of the dynamic characteristics of the structure, a trained investigator can indirectly estimate various parameters such as the location of the main damages, the residual structural capacity and the structural reliability. The evaluation of the modifications is achieved by comparison of the current state with a previous benchmark state. This benchmark state, however, is not always given, particularly in cases of sudden, unexpected external excitations such as seismic events or blast loads.

Despite the differences in the targets of laboratory and on-site investigations, a number of issues are common to the execution of DI tests. In the following, a list of common points is reported and briefly commented underlining particular cases regarding both lab and on-site tests.

The instrumentation used to record the vibrations (i.e. most commonly the acceleration or the velocity) across the structure does not depend on the location of the tests (lab or on site). The selection of the measuring devices is essentially connected to the target natural periods of the structure and to the intensity of the excitation or, on the other hand, to the amplitude of the movements to be recorded. This last means that the instruments will vary shifting from forced to ambient vibration, and vice versa, but choosing accelerometers or geophones (i.e. velocimeters) does not lead to different final results if both the instruments sensibility and the operational range suit with the magnitude of the signals to be recorded.

A particular case is the execution of DI tests on a shaking table, hence inside a lab. A convenient way to excite a structure placed on a shaking table is to use white noise vibrations. This has the advantage of exciting with the same energy content all the frequencies of interest, being clearly compatible with the shaking table characteristics. When using this type of excitation, "strong motion" instruments must be adopted in order not to saturate the signals going beyond their operational range.

The choice between forced or ambient vibration will be made on the basis of considerations other than the location of the structure under investigation. Nevertheless, some differences can arise when the choice is to use forced vibrations for on-site tests. This is due to the fact that, while everything can be adequately planned to use forced vibrations within a laboratory testing campaign, working on-site with forced vibrations can sometimes be difficult because of problems connected to the site accessibility and the eventual transportation of heavy machinery (e.g. vibrodynes). As a matter of fact, the on-site installation of the device imposing the vibrations to the structure can be sometimes difficult and, when feasible, it can be a source of unwanted disruption. Furthermore, an advantage of performing DI in a lab is that the execution of a number of tests exploiting different vibration sources to excite the specimen can be done with a reduced effort.

The same problems related to unwanted disruption can also affect the installation of instruments. Although in most of the cases the connection of the instruments used for the acquisition at the investigated structure is not a big issue, it can be problematic

when dealing with structures belonging to the historic wealth. As an example, when trying to identify the dynamic characteristics of a building, it is essential that the instruments are rigidly connected to the structure. If on a recent structure it would be enough to place the recording devices on the floor slabs; the slabs of historical structures are often very flexible, hence the instruments must be fixed to the walls. Although a few bolts are normally enough to assure the proper connection, to drill the required holes in the structure can result to be not acceptable (e.g. fresco walls).

With regard to the quality of the acquired signal, differences can arise between tests performed in a laboratory or on site. Although all acquired signals contain some electric noise, working on-site may additionally add some electric noise. As an example, signal acquisitions can be very difficult when working close to high-voltage electric lines such as underground or urban railways. A few approaches exist to circumvent this problem. One solution is to shield the acquisition cables; nevertheless this can be unfeasible or it can be insufficient, as in the case of DI performed on large building implying the necessity of using very long acquisition cables for the connection of the instruments to the acquisition system. Another option, when applicable, is to perform the DI tests when the source of noise is not present: in any case, this solution may imply to work with strict time schedules or at unusual hours, e.g. at night, probably causing logistic problems.

The problem of having a tight time window available to perform the on-site tests is not limited to the previously mentioned case. Quite often, when performing tests on public or critical structures (e.g. airport buildings or bridges), the only solution to minimise the interference between the execution of the tests and the utilisation of the structure to be characterised is to work at night or with very limited time at disposal. It is worth mentioning that interferences are present both in-situ as well as in the lab: the experimental instrumentation can cause limitation to the use of the structure and the use of the structure can be an unwanted source of excitation leading to a disturbance in the acquired signal. As an example of this last case, it is sufficient to think about the vibrations induced by people walking close to very sensitive instruments (i.e. geophones) during the execution of the identification of a building. It is not rare that this kind of vibrations can lead to the saturation of the acquired signals or can add energy to a frequency clearly not related to the structural characteristics. This kind of problem is clearly affecting the performance of the DI since the available margins of error and the time to solve unexpected problems are clearly imposing strong constraints on the experimental activities. Obviously, these issues related to the time schedule of the DI tests do not affect the laboratory investigations: the staff of the lab should have enough experience to properly schedule all the required experimental activities.

The inclusion of soil-structure interaction effects is a further big difference between the use of DI inside a lab or on-site. When operating on-site the structure is clearly placed on the ground; this implies that the soil-structure effects will influence the results of the DI, particularly in the case of "intense" forced vibrations. This interaction must be properly accounted for during the subsequent phases of data reduction and results interpretation. On the contrary, when working on a specimen located inside a laboratory, the DI tests will lead to the determination of the characteristics of the bare structure as in most of the cases the prototype structure is fixed to the strong floor of the laboratory or to the shaking table. Although the differences induced by the soil-structure interaction effects cannot be neglected, the final targets of laboratory and on-site DI tests are probably different, hence the modification of the results cannot be considered an advantage nor a drawback of one of the possible test location. The inclusion of soil-structure interaction effects is surely adding information to the results of the DI, nevertheless it also implies a more complex results interpretation due to the fact that the boundary conditions of the problem are an additional unknown.

The following Table 2 underlines possible criticalities to the execution of both laboratory and on-site DI tests.

Table 2. Comparison of possible criticalities in performing DI test in lab or on-site

| **Criteria** | **Laboratory DI tests** | **On-site DI tests** |
|---|---|---|
| Determination of the dynamic characteristics of the structure | + | + |
| Input for model updating | + | + |
| Problems related to unwanted disruption | - | +/- |
| Soil-structure interaction | - | + |
| Logistic problems related to time scheduling | - | +/- |
| Unwanted vibration sources | - | +/- |
| Electric disturbance | - | +/- |
| Full scale structure | +/- | + |
| Aged structure | - | + |

### *3.2 Wireless Sensor Technologies*

Important factors, decisive whether or not a particular (small) damage can be detected, are the choice of sensors (type, sensitivity, location, number) and the covered frequency interval. Often, ambient measurements only deliver quantitatively good information of the lower modes. A way to extend the frequency interval is the use of a (rather small) artificial excitation source, extra to the ambient disturbances. Proper sensors and signal conditioning are not a major problem in the case of forced vibration test (FVT) because the intensity of the signals to be measured remains more or less constant. Nevertheless, the force excitation level should be high enough to prevail upon the ambient influences.

There are a number of sensor technologies that have been used in vibration based testing. Following are the most common systems:

- Traditional technology (electric signals)
- Optical fiber technology
- Non-contact measurements (microwave interferometry, laser, radar, etc.)
- Wireless sensors
- GPS

Classical and novel sensors have been extensively and continuously updated in a number of review publications (Catbas et al. 2011; Doebling et al. 1996; Sohn et al. 2004). It is believed that two new technologies, optical fibers and wireless sensors, will become more and more popular. The former offers the possibility of measuring dynamic strains, the latter offers the advantage of increasing setup speed. Whereas, non-contact measurements are less reliable in outside laboratory conditions and GPS technology is promising but not yet widely used (due to low resolution). Current trends in sensing techniques have been addressed extensively in the latest International Conference on Structural Dynamics – EURODYN2011 - in three different mini-symposia: MS11 – Experimental techniques, MS12 – System identification and structural health monitoring, MS26 – Non-contact measurements (De Roeck et al. 2011). The following discussion will provide insight into the wireless sensor technology.

*Wireless sensors*

In comparison to high costs and high expenditure of time associated with the wired structural monitoring systems, the use of wireless sensors is advantageous in terms of simple configuration and rapid deployment. Recently, wireless sensors have gained popularity in the civil engineering community for SHM (Cunha et al. 2006; Caetano et al. 2007; Ceriotti et al. 2009; Reynders et al. 2011). Wireless acquisition systems are often assembled from wireless sensor units that combine sensing, data acquisition and communication capacities in a single device (Lynch, 2002; Spencer et al. 2004; Lei et al. 2005; Lynch et al. 2006; Pakzad et al. 2008).

For structural monitoring, a wireless sensor network consists of wireless sensor units. Time synchronization is often one of the most important issues (Lynch et al. 2006), since asynchronous data have significant effects on identified mode shapes, though their effects on the identified frequencies and damping ratios are negligible (Lei et al. 2005). There are two types of time synchronization problems to be addressed: the first one is temporal jitter, or clock deviation, which is shown as non-uniform sampling intervals within one wireless sensor unit; and the second one is spatial jitter, which is characterized by time-synchronization errors among different wireless sensors (Pakzad et al. 2008). A temporal jitter problem can often be solved by using a time clock with high accuracy (GeoSIG 2011) and a spatial jitter problem can usually be solved by deploying time synchronous Wi-Fi network among the wireless sensors.

Recently, several large scale, complete field tests have been successfully carried out using wireless sensors only. Detailed descriptions on the use of this instrumentation are given in the case study section.

## *3.3 Soil structure interaction*

### *3.3.1 Generalities*

Soil–foundation–structure interaction is, as the name implies, the mutual interaction between the soil and the superstructure (foundation and structure). During earthquake shaking, the propagation of seismic waves leads to transient soil deformations. These deformations force the foundation and the supported superstructure to oscillate. In turn, the oscillating superstructure generates inertial forces that transfer dynamic stresses at the foundation, which are transmitted back to the soil. Thus, an additional wave-field emanating away from the soil–foundation interface is created. This wave-field introduces further deformations in the soil. These phenomena occur simultaneously. However, it is quite usual to separate the two successive phenomena, into the so-called "kinematic" and "inertial" interaction (Fig. 1). The final response of the soil-foundation-structure system can be obtained as the superposition of the two phenomena (Gazetas, 1983; Pecker, 1984).

*Kinematic interaction*

Kinematic interaction refers to the effects of the seismic transient waves (propagating in the soil) on the response of soil–foundation system, when it is considered to be massless and fixed connection at the foundation – soil interface is assumed. The main effect of introducing this rigid massless system in the soil is the modification of the seismic input motion at the foundation with respect to the free-field motion (FFM). The modified foundation input motion is usually referred to as "foundation input motion" (FIM). The modification of the FIM is attributed to the results of the wave incoherence, inclination and/or embedment of the foundation (Stewart et al. 1999). The FIM consists of a translational and a rotational component, the latter introduced by the vibration of the foundation due to the absence of symmetry, or as a result of the wave incoherence or inclination. In most of the cases, the FIM seems to be de-amplified with respect to the FFM. This deamplification is negligible in case of surface foundations, while it seems to be more important in case of embedded and pile foundations.

*Inertial interaction*

Inertial interaction refers to the complete response of the soil–foundation–structure system to the excitation of D'Alembert forces associated with the acceleration of the superstructure due to the FIM (calculated taking into account the kinematic interaction effects). In case of surface foundations without a large rigid foundation slab, the kinematic interaction effects seem to be negligible as compared to the inertial interaction effects. Coupling of kinematic and inertial interaction is achieved by calculating the foundation impedance functions at the soil – foundation interface.

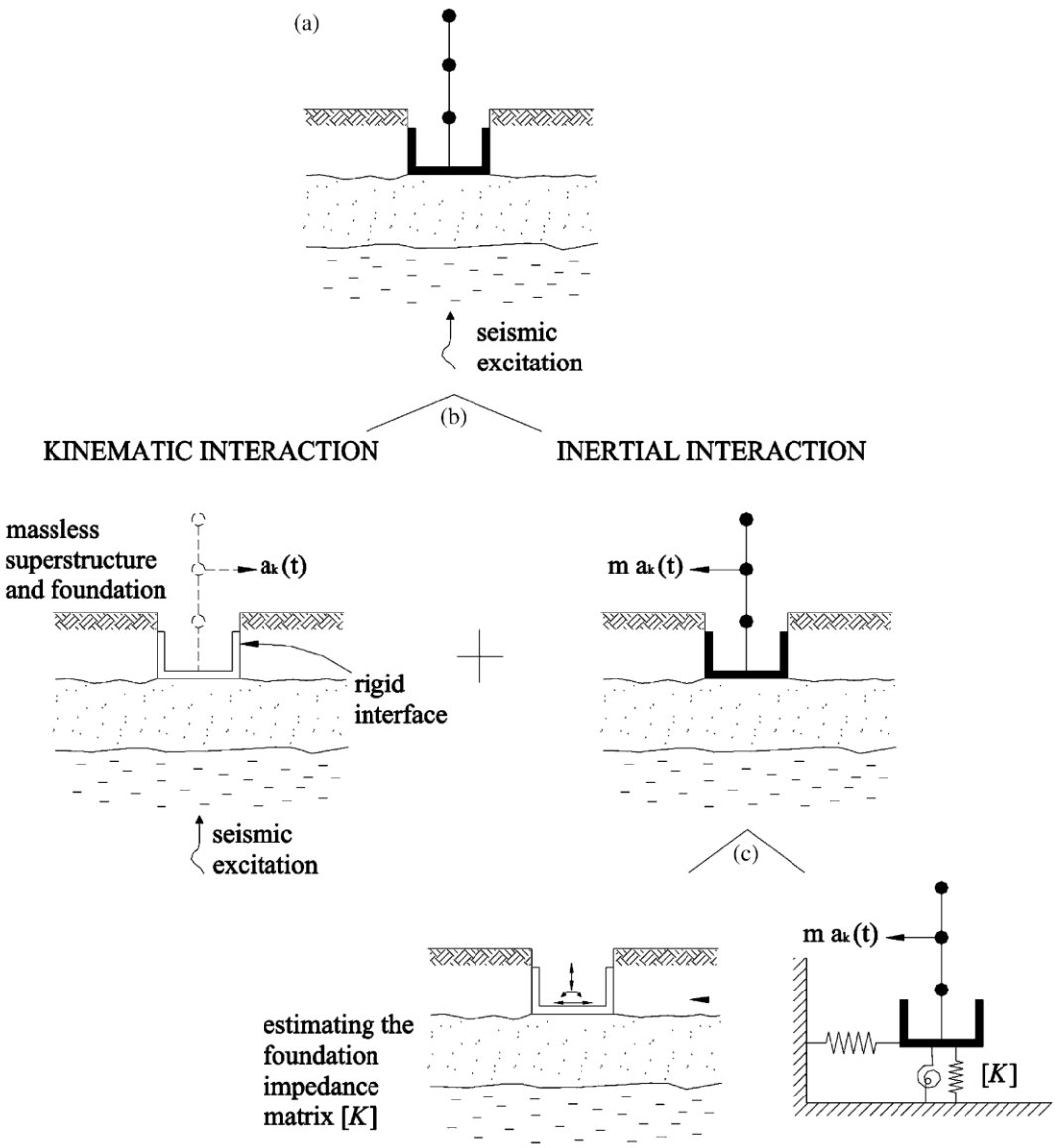



Fig. 1. (a) The SSI phenomenon, (b) decomposition into kinematic and inertial interaction, (c) two-step analysis of inertial interaction (modified after Kausel et al. 1976 and Mylonakis et al. 2006)

### *3.3.2 Methods for SFSI analysis*

The developed methods of modeling and analysing the SFSI can be grouped in three major categories:

- direct methods
- substructure methods
- hybrid methods

*Direct methods*

Direct methods analyse the complete soil-foundation-structure system in its integrity. Usually, a finite element method is implemented for the determination of the system response in a single step. The analysis can be performed either in the time domain or in the frequency domain. The main advantage of the FEM is that material or geometric nonlinearities and heterogeneities of the system can easily be incorporated. When the analysis is performed in the time domain, linear or nonlinear material behaviour can be

incorporated through specially defined constitutive laws. Analysis in the frequency domain can be performed in cases where linear or equivalent linear behaviour for the soil deposit is assumed. The latter assumption leads to a simplified description of the nonlinear behaviour of the soil. The main drawback of direct methods is the high computational cost of the performed analysis, as well as the inadequacy of satisfying the wave radiation condition to infinity.

*Substructure methods*

Substructure methods separate the problem into several subdomains. Based on the dynamic equilibrium and the kinematic constraints between the interacting components of the system (the soil, the foundation and the superstructure), the global domain can be well defined. After this proper definition, each subdomain can be modelled and analysed with the most appropriate and effective method. The final response of the system is determined using the superposition technique. The latter requires the assumption of linear elastic or viscoelastic behaviour of the components of the system and the problem can be solved only in the frequency domain. The main drawback of substructure methods is that they cannot capture the nonlinearities (geometrical and material) or the heterogeneities of the model in a detailed manner. As soil exhibits nonlinear behaviour even under weak seismic ground motions, substructure methods may lead to overestimation of the internal forces of the substructure, ignoring, at the same time, the permanent deformations of the soil.

*Hybrid methods*

The hybrid methods combine the advantages of direct and substructure methods. Taking the advantage of substructuring the soil–foundation–superstructure model into a bounded structure domain and an unbounded soil domain, the analysis can be performed in a more sufficient way, allowing the implementation of the finite element method (FEM) for the structure domain and the boundary element method (BEM) for the soil domain. In the bounded structure domain, a portion of the soil adjacent to the structure is also modelled with FEM. This area that is extended to an artificial boundary (the so called "general soil structure interface") is substantially affected by the structure. Beyond this area the effects of the structure existence on the soil response is considered to be negligible. The kinematic coupling between these domains is established with a dynamic impedance concept calculated for general soil structure interface.

### *3.4 Structural identification*

In vibration based testing, after data collection, modal parameters (eigenfrequencies, damping ratios, mode shapes and modal scaling factors) have to be extracted in a two-step procedure. In the first step (system identification), a model of the structure is identified from the data, e.g. a state-space model or a non-parametric frequency response function. In the second step (modal analysis), the modal parameters of the identified model are calculated and divided into true (physical) modes and spurious modes, that appear due to measurement noise, modeling errors, harmonics, etc.

This section presents a number of recent innovations and developments in system identification methods for field testing. Special attention is being paid to techniques making use of operational data (service load testing – AVT). One important aspect is the consideration of the uncertainty on the extracted modal parameters from measured accelerations, displacements and/or strains. Uncertainty can be due to the finite number of data samples, colored excitation, non-stationary structural behaviour, mass loading, inherent non-linearity, etc. It is crucial to estimate this uncertainty because it propagates in the subsequent exploitation of the experimental results.

*Elimination of environmental influences*

The elimination of the environmental influences from the identification results is crucial. The addressed phenomenon is the spontaneous change of the dynamic signature of a

structure by variations of the natural environment. Although this observation is not yet completely understood and certainly not quantified, the main factor is believed to be the temperature. Its effects can be:

- change of Young's moduli of structural materials like concrete, but also of secondarily contributing materials like superimposed asphalt deck
- modification of boundary stiffness: variation in fixation degree, difference in soil stiffness, e.g. summer versus winter conditions
- opening or closing of existing cracks
- variations in moisture content (can contribute to this environmental influence even though to a lesser extend)
- changes of span length, especially of bridges

Several statistical methods have been proposed to cope with this disturbance (Peeters et al. 2001a; Deraemaeker et al. 2008). Measurements of the influencing parameters (e.g. temperature) may or may not be included in these methods. If they are not included, these statistical methods intrinsically assume that the dynamic signature is influenced in a different way by structural damage than by changing temperatures or other environmental parameters. In general, a temperature increase causes a decrease in eigenfrequencies, while the mode shapes are much less affected. Structural damage will have a selective influence on modes: especially these eigenfrequencies will decrease with corresponding mode shapes that show high modal strains at the locality of occurred damage. Also the mode shapes themselves will be modified.

*Variance of extracted modal parameters*

One of the most robust and accurate system identification methods for OMA of civil engineering structures turned out to be the stochastic subspace identification (SSI) method and its reference-based generalization (SSI/ref) (Peeters et al. 1999). Two implementations with similar accuracy are available: covariance-driven (SSI-cov) and data-driven (SSI-data) ones. Till now, a drawback of the SSI method has been that, from the identification algorithm itself, no information was available concerning the accuracy of the identified modal parameters.

From repeated tests, some information about the statistical variation of the modal parameters can be obtained. In a recent paper (Reynders et al. 2008a), novel covariance formulae are derived for the system matrices obtained from SSI/ref by a single experiment. Furthermore, it is indicated how covariances of modal parameters (eigenfrequencies, damping ratios and mode shapes can be calculated from the covariances of the system matrices). Only the covariance-driven implementation is covered since it is the most straightforward one and since its covariance analysis is computationally much less expensive than that of the data-driven implementation, while the estimated modal parameters turn out to be of similar accuracy.

### *3.4.1 System identification*

Many texts exist that give extensive review and comparison of system identification methods applied in vibration based field testing (e.g. Peeters et al. 2001b; Reynders et al. 2011a; Reynders, 2009). For FVT, well established methods exist, either in time or frequency domain. In case of AVT, picking of the peaks (PP) observed in frequency spectra was for a long time current practice, at least in civil engineering. Nowadays, more powerful system identification methods are being used, like the enhanced frequency decomposition method (Brincker et al. 2007), stochastic subspace identification (SSI) method (Peeters et al. 1999) and the Polymax method (Peeters et al. 2004), offering a good combination between processing speed and accuracy. With these methods, better results are obtained for the true mode shapes (and not operational deflection shapes as in the case of PP). Besides, closely spaced modes can be more easily separated. A convenient tool for finding the correct physical poles of the structure is the so-called stabilization diagram, showing the "stability" of poles at increasing model orders.

The first step of system identification is adopting a certain model structure. Afterwards, the parameters of the chosen model are estimated from measurement data. A wide range of model structures have been proposed in system identification literature, see for instance Ljung, 1999. It is convenient to list here several popular models.

### *3.4.2 Physical models*

The finite element formulation is the most common and realistic representation of a physical model. In the case of a linear dynamic model with general viscous damping, the equilibrium can be expressed by the following system of ordinary differential equations in a matrix form:

$$\mathbf{M}\ddot{\mathbf{q}}(t)+\mathbf{C}_{\mathbf{V}}\dot{\mathbf{q}}(t)+\mathbf{K}\mathbf{q}(t)=\mathbf{B}_2\mathbf{u}(t) \tag{1}$$

where $\mathbf{M}$, $\mathbf{C_V}$, $\mathbf{K}$ are the mass, viscous damping and stiffness matrices; $\mathbf{q}(t)$ is the displacement vector at continuous time $t$ and $\mathbf{B}_2\mathbf{u}(t)$ forms the vector of nodal forces.

*Continuous-time state-space model*

By re-arranging (1) and assuming that **M** has full rank, a continuous-time state-space model can be obtained by the following state-space equation:

$$\mathbf{x}(t)=\mathbf{A}_{\mathbf{C}}\mathbf{x}(t)+\mathbf{B}_{\mathbf{C}}\mathbf{u}(t) \tag{2}$$

where $\mathbf{A_C}$ is the state-space matrix and $\mathbf{x}(t)$ is the state of the structure, which contains both the displacement and velocity vectors of the dynamic system.

*Discrete-time state-space model*

Although the above physical model is a good representation of a vibrating structure, it is not directly useful in an experimental modeling context. First, this equation is continuous in time, whereas measurements are available as discrete time samples. Even if the time was a continuous variable, solving the continuous-time state-space description analytically for an given input $\mathbf{u}(t)$ is usually difficult (Peeters et al. 2001b). Therefore, it is natural to convert this model to discrete time:

$$\begin{aligned} \mathbf{x}_{k+1} &= \mathbf{A}\mathbf{x}_k+\mathbf{w}_k & \mathbf{w}_k &= \mathbf{B}\mathbf{u}_k \\ \mathbf{y}_k &= \mathbf{C}\mathbf{x}_k+\mathbf{v}_k & \mathbf{v}_k &= \mathbf{D}\mathbf{u}_k + \mathbf{n}_{y,k} \end{aligned} \tag{3}$$

where $\mathbf{y}_k$ is the output vector; $\mathbf{x}_k$ is the discrete state-vector; $\mathbf{w}_k$ is the process noise, typically due to disturbances, modeling inaccuracies and most importantly due to the unknown excitation of the structure; $\mathbf{v}_k$ is the measurement noise typically from sensors and also due to the unknown excitation; $k$ is the time instant. The matrix **A** is the state transition matrix that completely characterizes the dynamics of the system by its eigenvalues; and **C** is the output matrix that specifies how the internal states are transformed to the outside.

*Other models:* Auto-regressive moving average family, maximum likelihood methods, frequency domain models. Detailed discussions on these models can be found in a review (Peeters et al. 2001b).

### *3.4.3 Parameters estimation*

*The peak peaking method (PP)*

The method is named after the key step of the method: the identification of eigenfrequencies as the peaks of a spectrum plot. The spectrum around an eigenfrequency is estimated under assumption of low damping and well-separated eigenfrequencies. If these conditions are violated, the method identifies operational deflection shape which is the superposition of several closely spaced mode shapes.

Whatever intuitive and subjective tasks this method may be, it can be suitable for getting a first, quick idea of the structural modes before performing a more detailed analysis.

*The Complex Mode Indication Function (CMIF)*

As suggested by the name, the CMIF was originally intended as a tool to count the number of modes that is present in measurement data. As a useful byproduct, the CMIF also identifies the modal parameters from Frequency Response Functions (FRFs) by selecting peaks of the singular values (Shih et al. 1988). When applied to output-only data, the singular values of the power spectral density (PSD) matrix computed as the sum of the identified positive PSD (PSD+) matrix and its transpose are used; the method is sometimes also referred to as Frequency Domain Decomposition (FDD). It is noted that modal damping ratios are not provided for these methods, as nonparametric damping ratio estimation is not reliable (Reynders et al. 2011a).

*The poly-reference least squares complex frequency domain method (pLSCF)*

Originally, the least-squares complex frequency-domain (LSCF) estimation method was introduced to find initial values for the iterative maximum likelihood method. The method estimates a so-called common-denominator transfer function model. Quickly it was found that these "initial values" yielded already very accurate modal parameters with a very small computational effort. The most important advantage of the LSCF estimator is the fact that very clear stabilization diagrams are obtained.

The poly-reference least squares complex frequency domain estimator (pLSCF), also known under its commercial name Polymax (Peeters et al. 2004), is a poly-reference version of the LSCF method, using a so-called right matrix-fraction model. In this approach, also the participation factors are available when constructing the stabilization diagram. The main benefits of the poly-reference method are the facts that the singular value decomposition (SVD) step to decompose the residues can be avoided and that closely spaced poles can be separated. The method was introduced in Peeters et al. 2004 together with a simulated example including a pair of closely spaced modes. In pLSCF, both the deterministic algorithm that starts from FRF data and the stochastic algorithm that starts from PSD+ data, can be implemented, which make them applicable to both FVT and AVT.

*Time domain subspace identification method*

Coming back to Equation (3) with the assumptions that $\mathbf{u}_k$ and $\mathbf{n}_{y,k}$ are white noise, that an infinite amount of data is available, that the structure is completely linear with invariant properties in time and that the chosen system order equals real system order. In reality, these conditions are at least partially violated: the noise is colored, harmonics might be present, the amount of data is finite, the properties are influenced by temperature, moisture, non-stationary loads, etc. and the system order is chosen by the user. Therefore, the identified matrices are only estimates: $\hat{\mathbf{A}}$, $\hat{\mathbf{C}}$. From a statistical point of view, there are 3 types of errors in $\hat{\mathbf{A}}$ and $\hat{\mathbf{C}}$:

- bias of the model: ($\hat{\mathbf{A}}$, $\hat{\mathbf{C}}$) contains true and spurious modes
- bias of the modes: identified modes of the true system might be biased
- variance of the modes: modes of ($\hat{\mathbf{A}}$, $\hat{\mathbf{C}}$) may be subject to variance errors

Bias errors can be partly removed with a stabilization diagram. On the contrary, variance errors can only be estimated (Reynders et al. 2008a; De Roeck et al. 2008).

*Combined deterministic-stochastic subspace identification*

The combined deterministic-stochastic subspace identification method (CSI) takes both artificial and ambient excitation into account so that the amplitude of the artificial excitation can be small compared to that of ambient excitation. This helps to broaden the identified eigenfrequency range to higher modes and to increase the quality of the modal

parameters such as to estimate the absolute scaling factor of the identified mode shapes. The reference-based version (CSI/ref) was introduced to enhance accuracy and computing efficiency of the method in which the reference outputs play an important role (Peeters et al. 1999; Reynders et al. 2008b). Candidates for the reference outputs are reference sensors overlapping among different test setups because they are placed at optimal locations on the structure, where it is expected that all modes of vibration are present in the measured data. However, additional sensors may be included as references in the identification of one setup. On the contrary, by using only the reference outputs for constructing the subspace of past outputs, it is possible to lower the influence of noisy channels by not selecting them as references. Because these channels are still used for the construction of the subspace of future outputs, the useful information that is present in these outputs, e.g. modal displacements, is not lost.

CSI/ref is particularly suited for dynamic field testing of large constructions and it has been applied to the benchmark case of the Z24 bridge producing the best result so far. The detailed formulation of the method and the implementation of different CSI algorithms can be found in (Peeters et al. 1999; Reynders et al. 2008a; Reynders et al. 2008a; Reynders et al. 2010).

#### *3.4.4 Modal analysis*

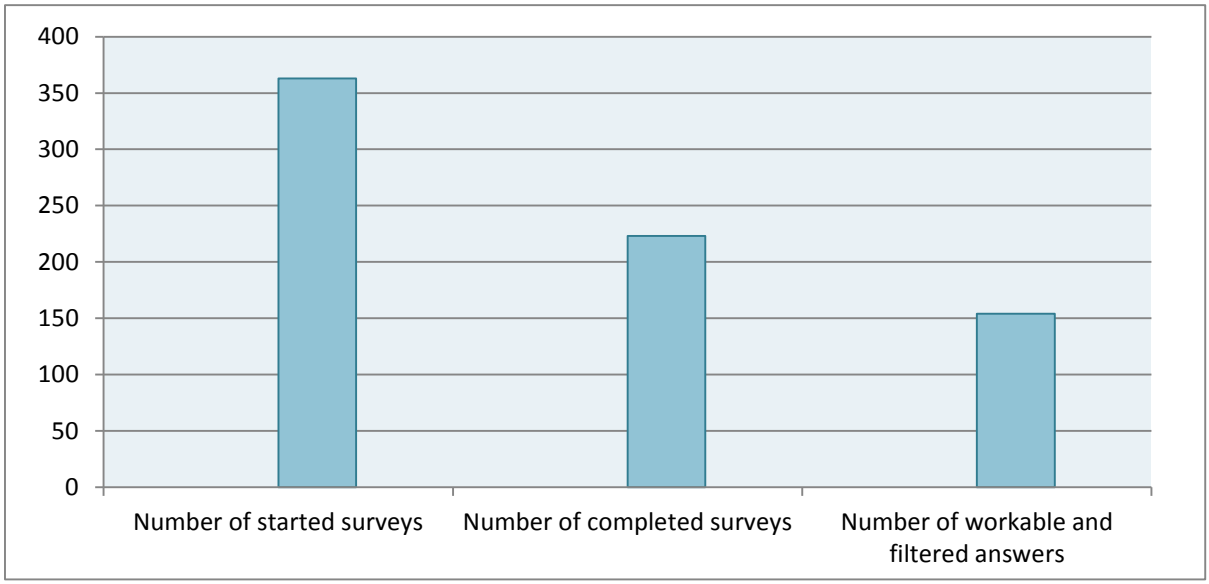
A handy tool for finding the correct physical poles of the structure is the so called stabilization diagram, showing the "stability" of poles at increasing model orders. If model order is higher than the true system order, also the noise is modeled, but the mathematical poles that arise in this way are different for different model orders if the noise is purely white. So, the true (physical) system poles can be detected by comparing the modal parameters for different model orders. In this way, also weakly excited system poles, that appear only at high model orders, can be detected.

In case the noise is colored, there could exist noise poles that show up at the same frequency for different orders in the stabilization diagram. However, due to their high damping ratios or to the corresponding complex or unrealistic mode shapes, they can be separated from the true system poles. It is also important to select the system poles at a sufficiently high model order, for which the system and noise dynamics have been decoupled. So to clear out the stabilization diagram, for a certain model order, when comparing with the poles of the lower model order, only those poles for which the relative difference in eigenfrequency, damping ratio and MAC value is below a threshold value, are plotted (Reynders et al. 2008a).

However, the stabilization diagram can still show some mathematical poles, especially for high model orders. This prevents not only the full automatization of analysis, but also can it make the selection of the physical poles from the stabilization diagram a time consuming step. For this reason an extra criterion - the modal transfer norm adopted from model reduction theory (Goethals et al. 2002) - is introduced into the stabilization diagram in order to facilitate the full automatization of both FVT and AVT with subspace identification methods (Reynders et al. 2011b).

# 4 Results of Questionnaires

| | |
|---|---|
| Number of started surveys | 363 |
| Number of completed surveys | 223 |
| Number of workable and filtered answers | 154 |

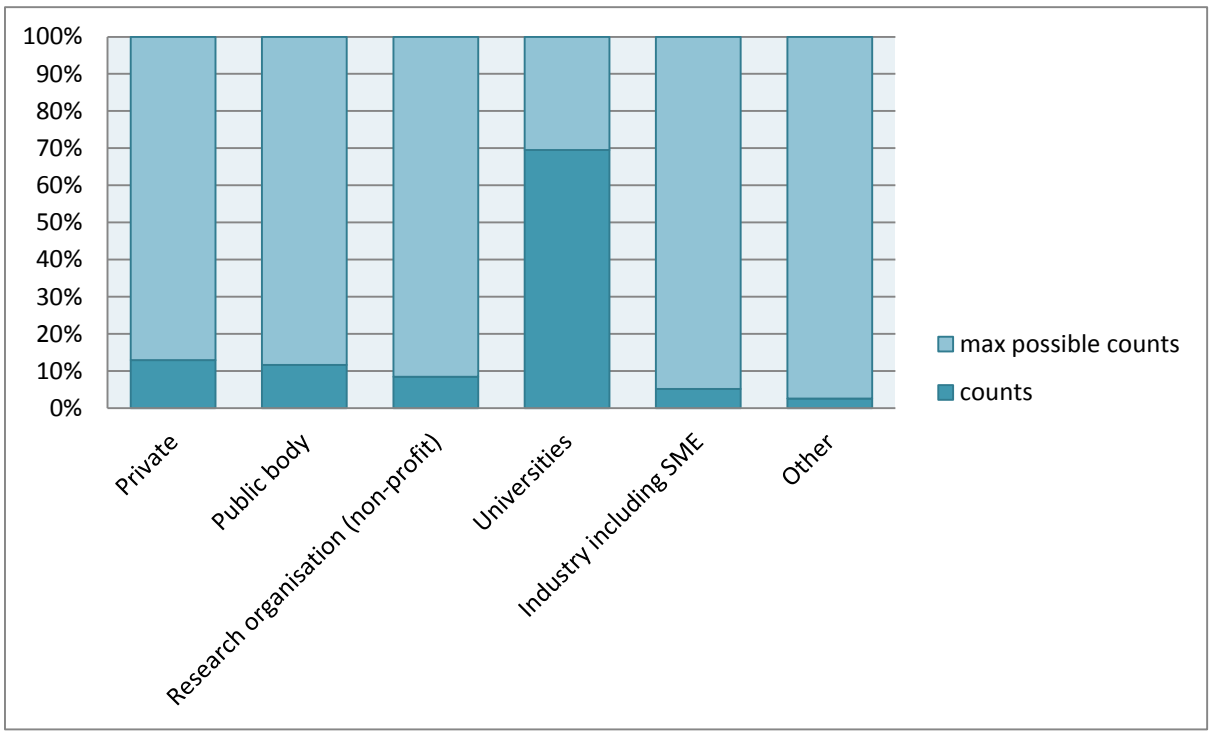


## Section: General

| **Question 1: Have you used any measurement technologies with respect to field testing?** | |
|---|---|
| | Count |
| Yes | 154 |
| No | 0 |
| Amount | 154 |

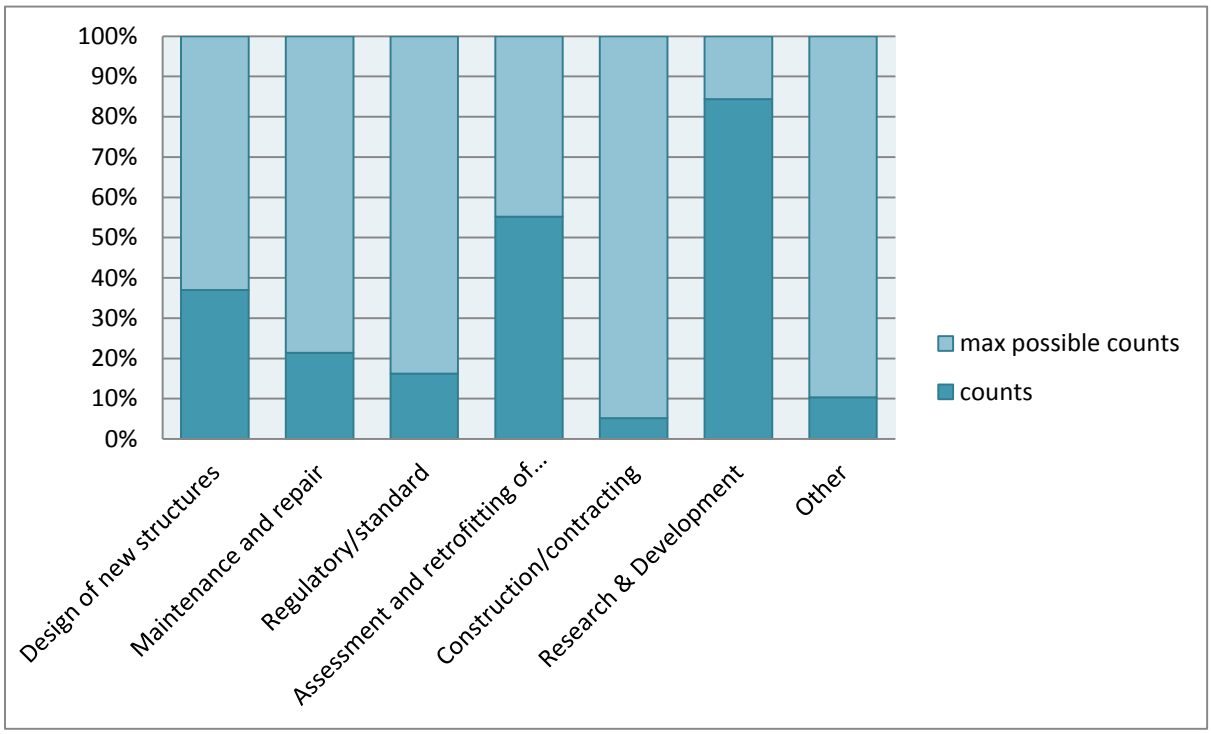
| **Question 2: Is your organisation private, public or both?** (multiple answers are possible) | |
|---|---|
| | Count |
| Private | 20 |
| Public body | 18 |
| Research organisation (non-profit) | 13 |
| Universities | 107 |
| Industry including SME | 8 |
| Other | 4 |
| Amount | 170 |

| Maximum possible counts per answer (both questions): | **154** |
|---|---|



| **Question 3: What is the special area of work (within earthquake engineering) of your organisation mainly specialised in?** (multiple answers are possible) | |
|---|---|
| | Count |
| Design of new structures | 57 |
| Maintenance and repair | 33 |
| Regulatory/standard | 25 |
| Assessment and retrofitting of existing structures | 85 |
| Construction/contracting | 8 |
| Research & Development | 130 |
| Other | 16 |
| Amount | 354 |

Maximum possible counts per answer: **154**

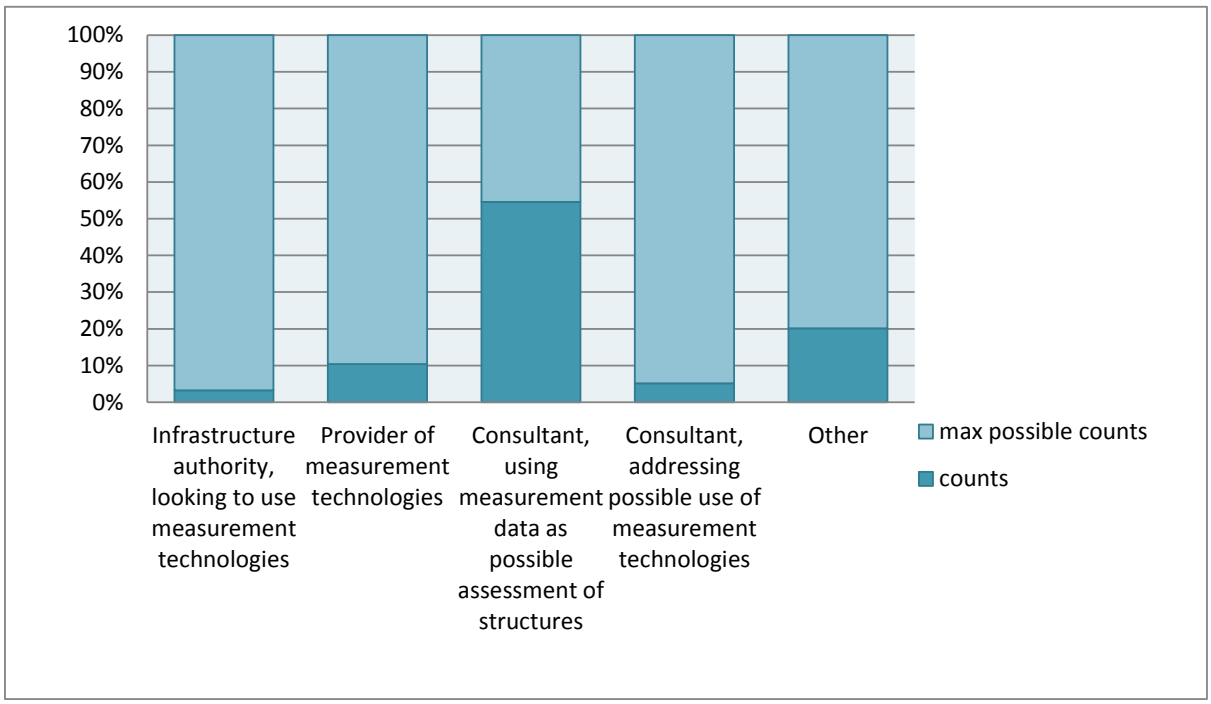



| **Question 3A: Please provide a short summary of the main services your organization offers in relation to EQ Engineering.** | |
|---|---|
| | Count |
| Fill in | 122 |
| No Answer | 32 |
| Amount | 154 |

Maximum possible counts per answer: **154**

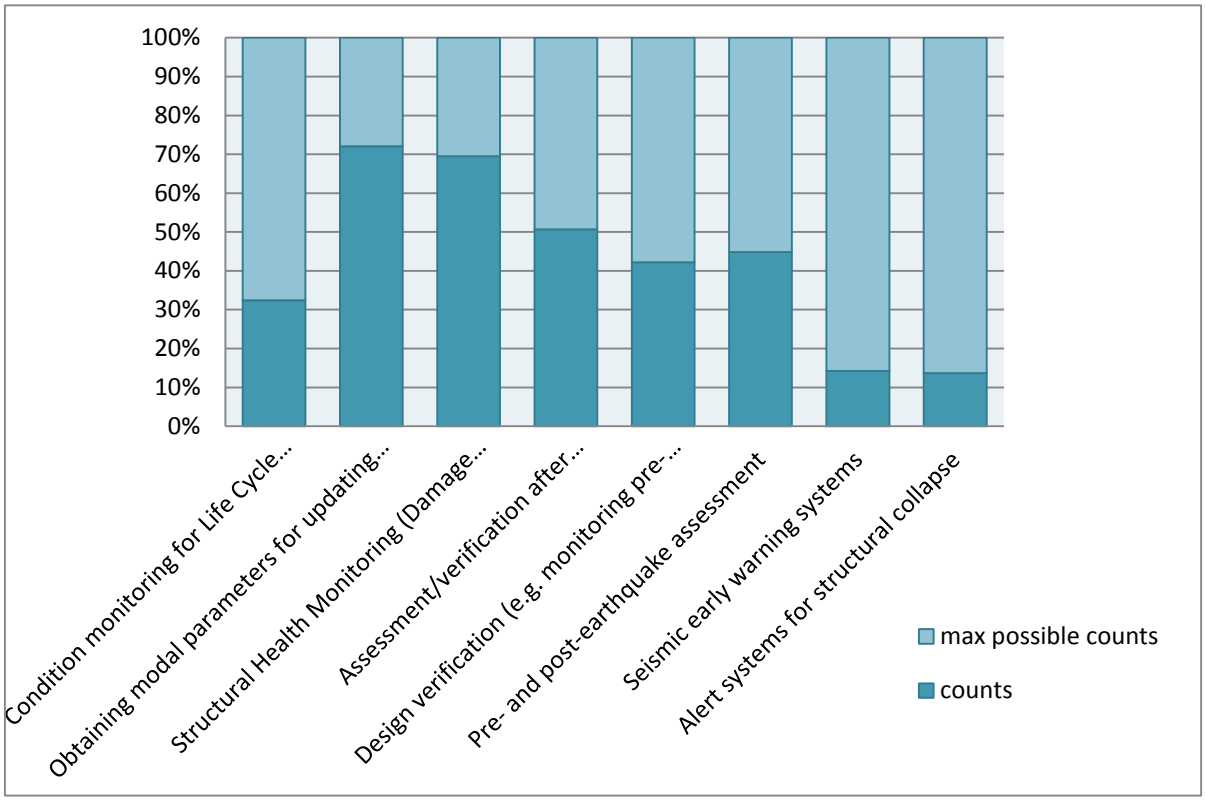
| **Question 4: What is the focus of your company with respect to field testing and measurement technologies?** (multiple answers are possible) | |
|---|---|
| | Count |
| Infrastructure authority, looking to use measurement technologies | 5 |
| Provider of measurement technologies | 16 |
| Consultant, using measurement data as possible assessment of structures | 84 |
| Consultant, addressing possible use of measurement technologies | 8 |
| Other | 31 |
| Amount | 144 |

Maximum possible counts per answer: **154**



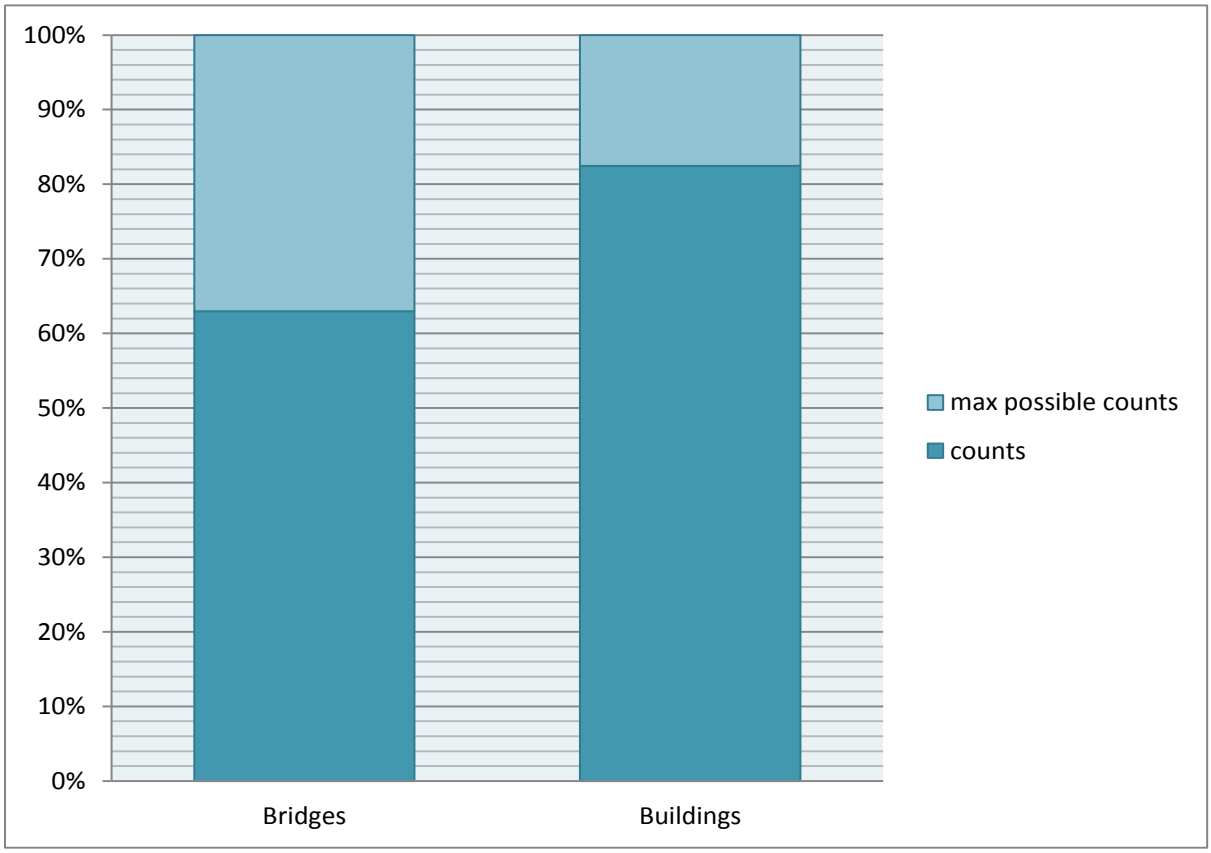
| **Question 5: Select the type of application for which you have used field testing technologies?** | |
|---|---|
| | Count |
| Condition monitoring for Life Cycle Assessment (e.g. vibration levels, etc.) | 74 |
| Obtaining modal parameters for updating approaches (Model reliability improvement) | 111 |
| Structural Health Monitoring (Damage detection) | 107 |
| Assessment/verification after retrofitting/strengthening | 78 |
| Design verification (e.g. monitoring pre-stress forces, etc.) | 65 |
| Pre- and post-earthquake assessment | 69 |
| Seismic early warning systems | 22 |
| Alert systems for structural collapse | 21 |
| Amount | 547 |

Maximum possible counts per answer: **154**



| **Question 6: Which kind of structures do you investigate with in-situ field testing?** | |
|---|---|
| | Count |
| Bridges | 97 |
| Buildings | 127 |
| Total section answers | 224 |

Maximum possible counts per answer: **154**

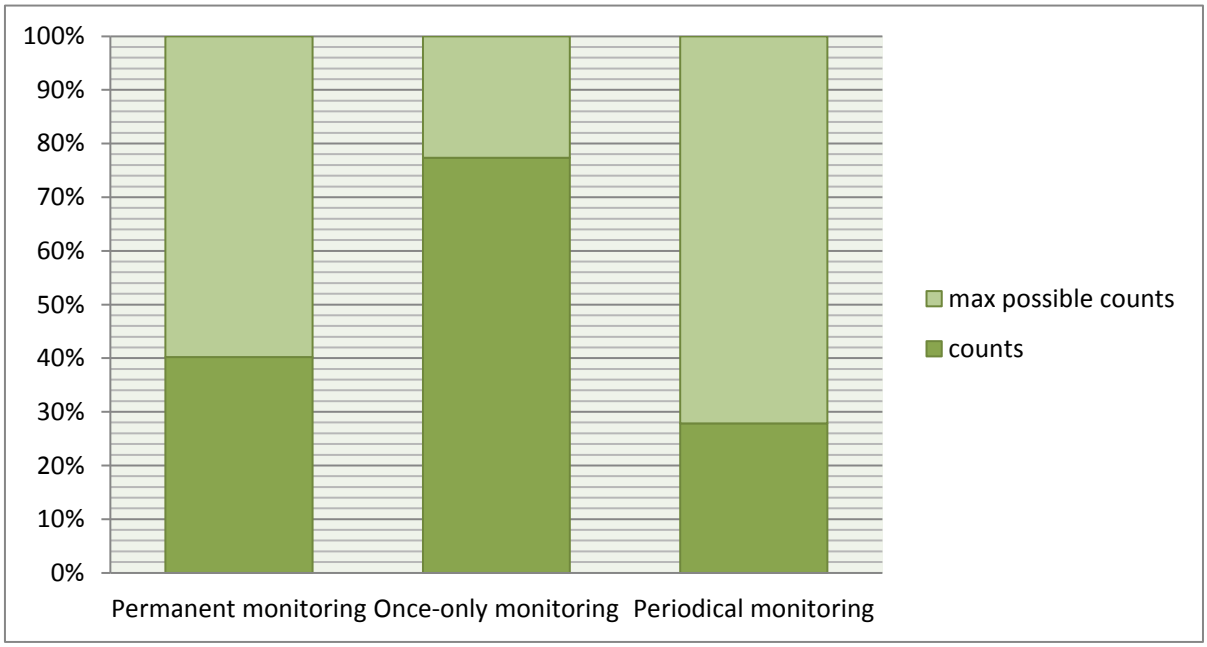


## Section: Bridges

**97 participants completed this section**

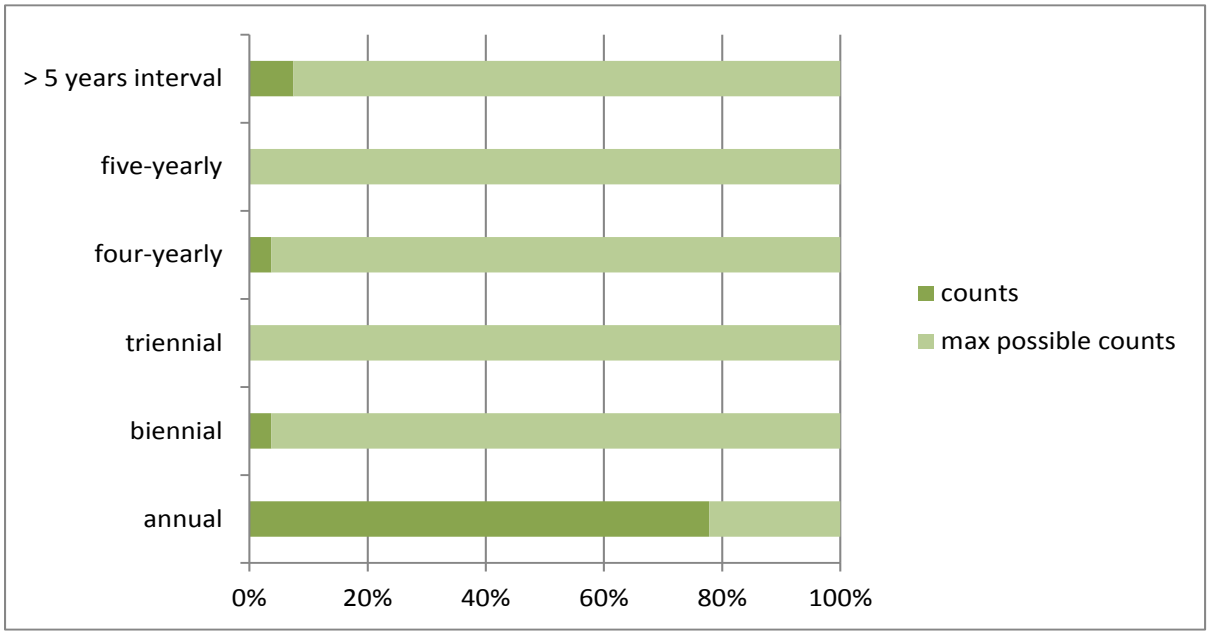
| **Question 7: What is the monitoring rate?** (multiple answers are possible) | |
|---|---|
| | Count |
| Permanent monitoring | 39 |
| Once-only monitoring | 75 |
| Periodical monitoring | 27 |
| Amount | 141 |

Maximum possible counts per answer: **97**



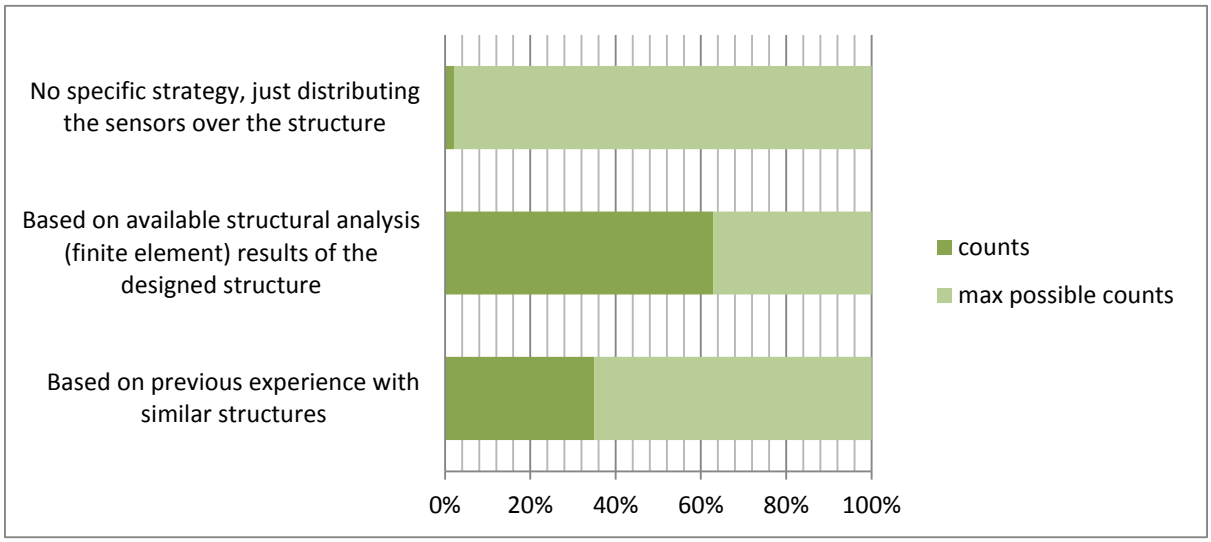

| **Question 7A: If Periodical monitoring:** (multiple answers are possible) | |
|---|---|
| | Count |
| annual | 21 |
| biennial | 1 |
| triennial | 0 |
| four-yearly | 1 |
| five-yearly | 0 |
| > 5 years interval | 2 |
| Amount | 25 |

Maximum possible counts per answer: **27**




| **Question 8: How do you design the measurement grid and the instrumentation before the actual field test?** | |
|---|---|
| | Count |
| Based on previous experience with similar structures | 34 |
| Based on available structural analysis (finite element) results of the designed structure | 61 |
| No specific strategy, just distributing the sensors over the structure | 2 |
| Amount | 97 |

| Maximum possible counts per answer: | **97** |
|---|---|



**Question 9: How many bridges of the following types have you assessed with permanent field testing?**

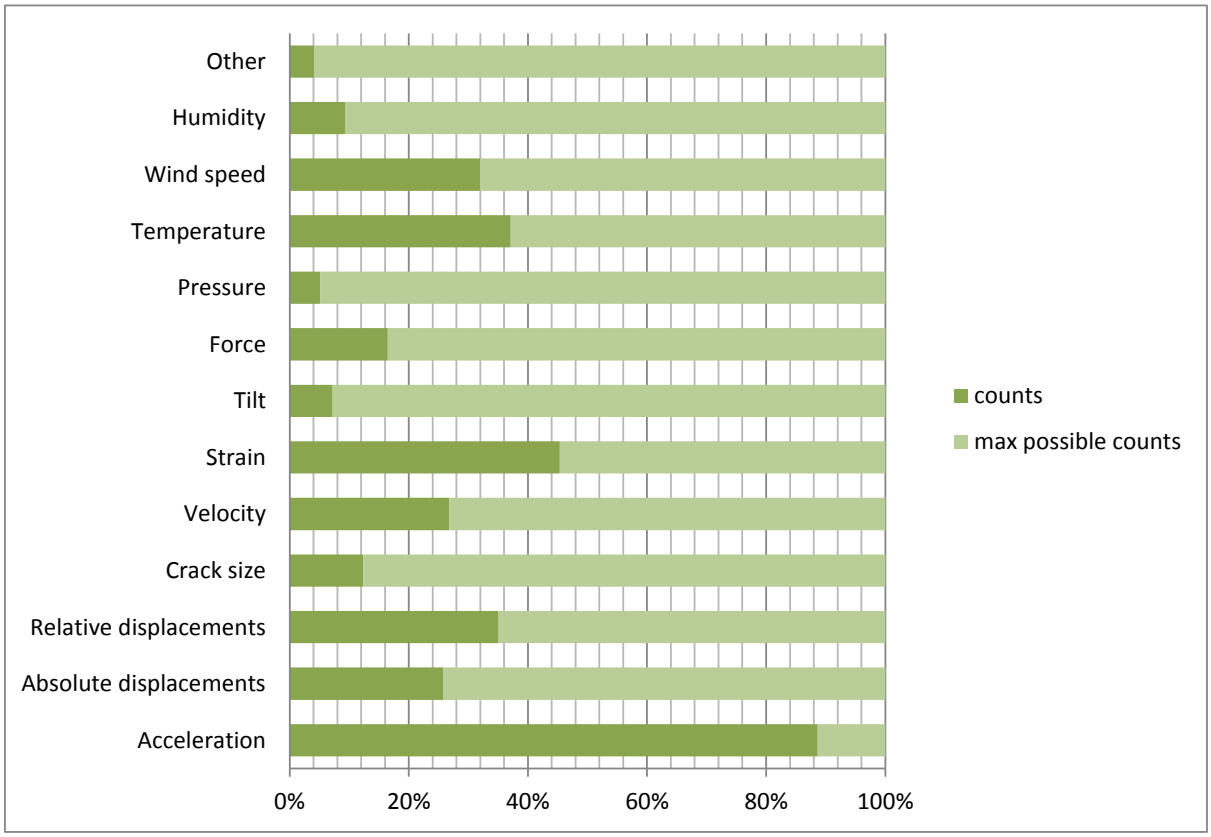
| | with <5 sensors | Bridges with <10 sensors | with <20 sensors | Bridges with <30 sensors | with <40 sensors | Bridges with >40 sensors |
|---|---|---|---|---|---|---|
| Arch bridge | 8 | 11 | 9 | 1 | 1 | 0 |
| Cable-net bridge | 0 | 0 | 0 | 0 | 0 | 0 |
| Cable-stayed | 1 | 2 | 7 | 3 | 2 | 11 |
| Covered bridge | 0 | 0 | 0 | 0 | 0 | 0 |
| Girder bridge | 13 | 30 | 17 | 6 | 2 | 21 |
| Hyperbolic paraboloid bridge | 0 | 0 | 4 | 0 | 0 | 4 |
| Movable bridge | 0 | 2 | 1 | 0 | 0 | 1 |
| Pontoon bridge | 0 | 0 | 0 | 0 | 0 | 0 |
| Rigid frame bridge | 3 | 3 | 4 | 0 | 0 | 0 |
| Stressed ribbon bridge | 0 | 3 | 0 | 0 | 0 | 0 |
| Suspension bridge | 0 | 3 | 3 | 1 | 1 | 4 |
| Trestle bridge | 0 | 0 | 0 | 0 | 0 | 0 |
| Truss bridge | 3 | 7 | 2 | 0 | 1 | 3 |

**Question 10: How many bridges of the following types have you assessed with only-once or periodical field testing?**

| | Bridges with <5 sensors | Bridges with <10 sensors | Bridges with <20 sensors | Bridges with <30 sensors | Bridges with <40 sensors | Bridges with >40 sensors |
|---|---|---|---|---|---|---|
| Arch bridge | 21 | 13 | 30 | 14 | 1 | 5 |
| Cable-net bridge | 12 | 0 | 2 | 0 | 0 | 1 |
| Cable-stayed | 29 | 5 | 26 | 8 | 0 | 3 |
| Covered bridge | 1 | 0 | 3 | 0 | 0 | 0 |
| Girder bridge | 60 | 57 | 87 | 15 | 11 | 19 |
| Hyperbolic paraboloid bridge | 2 | 0 | 2 | 0 | 0 | 1 |
| Movable bridge | 0 | 0 | 1 | 0 | 0 | 3 |
| Pontoon bridge | 0 | 1 | 1 | 0 | 0 | 1 |
| Rigid frame bridge | 29 | 16 | 30 | 4 | 1 | 1 |
| Stressed ribbon bridge | 0 | 0 | 4 | 0 | 1 | 1 |
| Suspension bridge | 19 | 5 | 11 | 0 | 2 | 4 |
| Trestle bridge | 0 | 0 | 1 | 0 | 0 | 0 |
| Truss bridge | 40 | 10 | 31 | 10 | 3 | 4 |

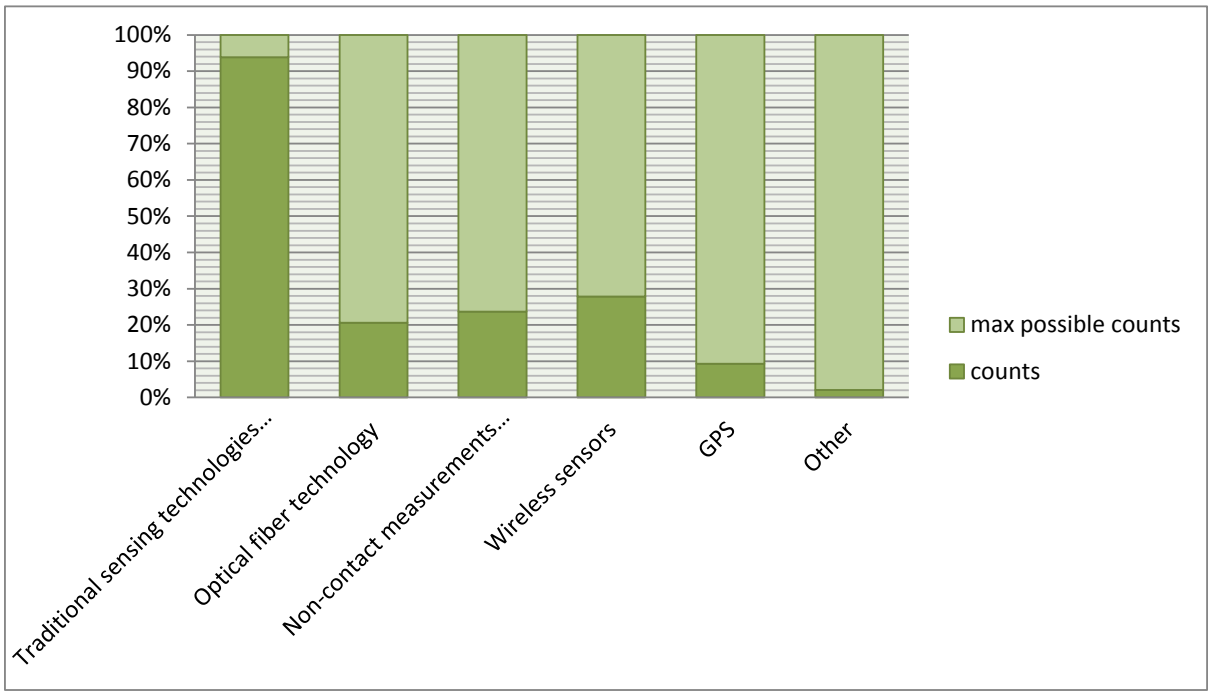
| **Question 11: Which parameters are monitored?** (mandatory) (multiple answers are possible) | |
|---|---|
| | Count |
| Acceleration | 86 |
| Absolute displacements | 25 |
| Relative displacements | 34 |
| Crack size | 12 |
| Velocity | 26 |
| Strain | 44 |
| Tilt | 7 |
| Force | 16 |
| Pressure | 5 |
| Temperature | 36 |
| Wind speed | 31 |
| Humidity | 9 |
| Other | 4 |
| Amount | 335 |

| Maximum possible counts per answer: | **97** |
|---|---|



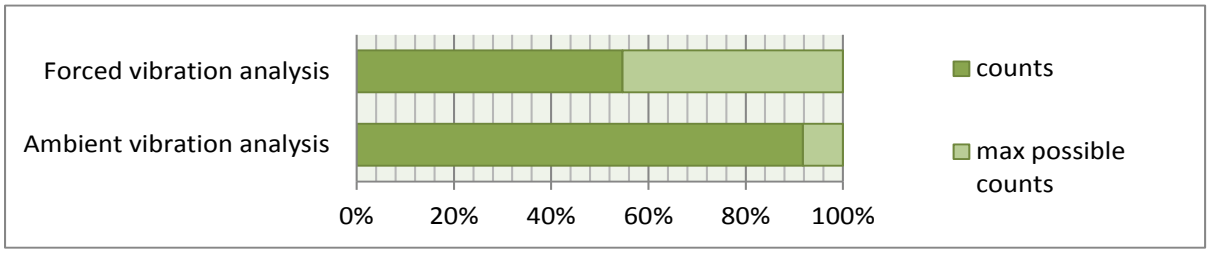
| **Question 12: Which sensor technologies are in use?** (mandatory) | |
|---|---|
| | Count |
| Traditional sensing technologies (electric signals) | 91 |
| Optical fiber technology | 20 |
| Non-contact measurements (Microwave interferometry, Laser, etc.) | 23 |
| Wireless sensors | 27 |
| GPS | 9 |
| Other | 2 |
| Amount | 172 |

Maximum possible counts per answer: **97**



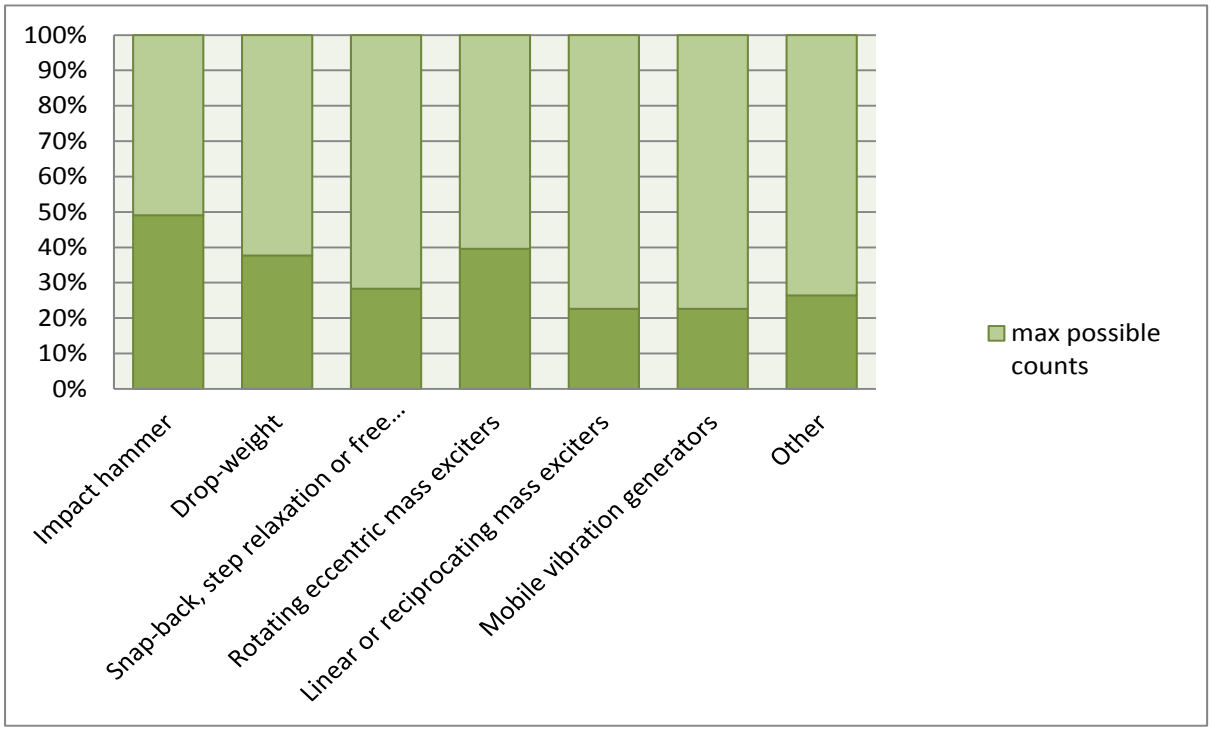

| **Question 13: Which excitation do you use for field testing measurements?** (multiple answers are possible) | |
|---|---|
| | Count |
| Ambient vibration analysis | 89 |
| Forced vibration analysis | 53 |
| Amount | 142 |

Maximum possible counts per answer: **97**



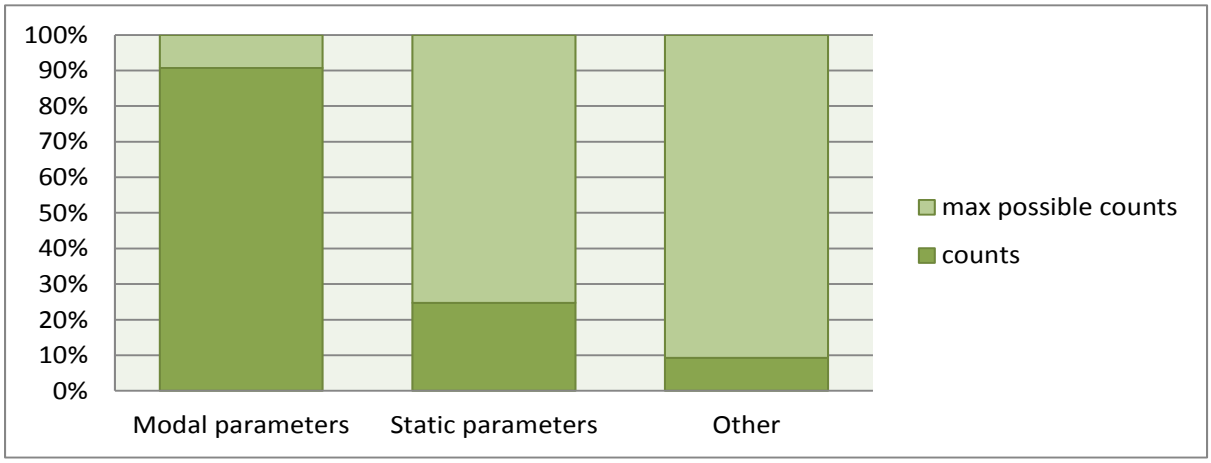
| **Question 13A: If you use forced vibration analysis, please specify:** | |
|---|---|
| | Count |
| Impact hammer | 26 |
| Drop-weight | 20 |
| Snap-back, step relaxation or free vibration | 15 |
| Rotating eccentric mass exciters | 21 |
| Linear or reciprocating mass exciters | 12 |
| Mobile vibration generators | 12 |
| Other | 14 |
| Amount | 120 |

Maximum possible counts per answer: **53**



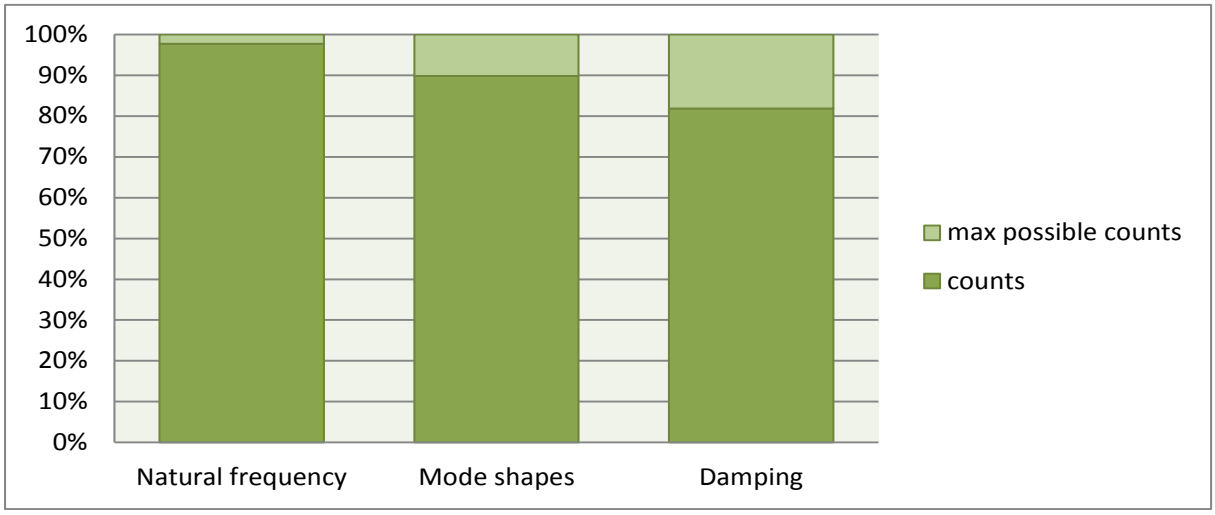

| **Question 14: Which structural parameters do you extract from measurement data?** | |
|---|---|
| | Count |
| Modal parameters | 88 |
| Static parameters | 24 |
| Other | 9 |
| Amount | 121 |

Maximum possible counts per answer: **97**



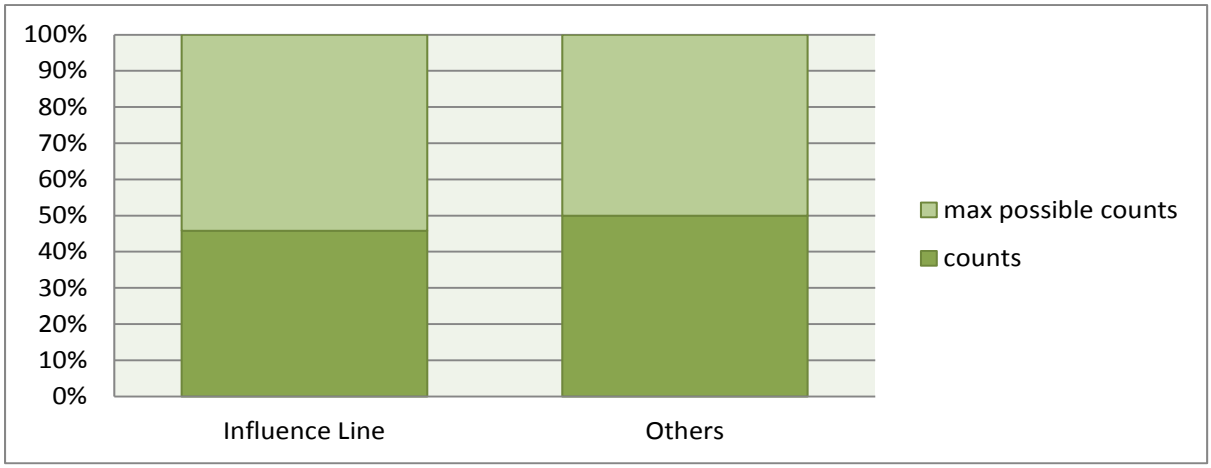

| **Question 14A: If you use Modal parameters:** | |
|---|---|
| | Count |
| Natural frequency | 86 |
| Mode shapes | 79 |
| Damping | 72 |
| Amount | 237 |

| Maximum possible counts per answer: | **88** |
|---|---|



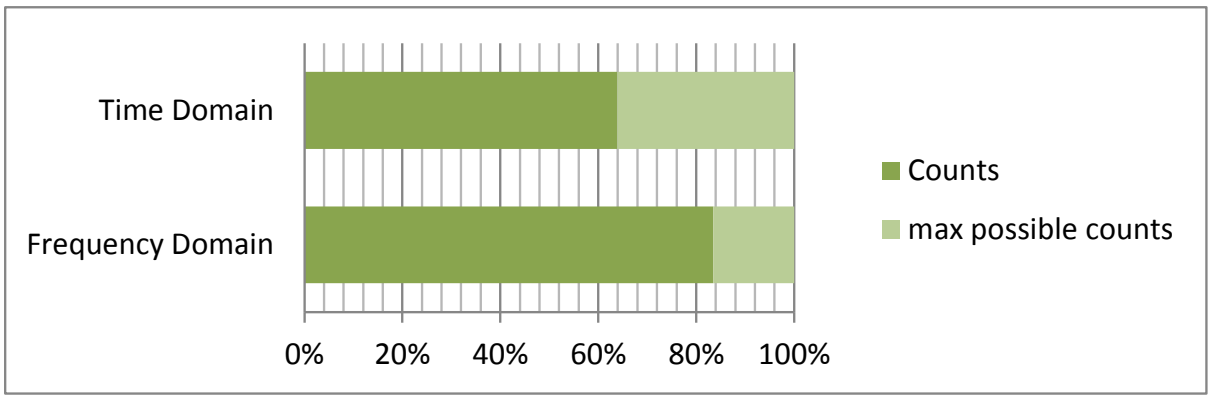
| **Question 14B: If you use Static parameters:** | |
|---|---|
| | Count |
| Influence Line | 11 |
| Others | 12 |
| Amount | 23 |

Maximum possible counts per answer: **24**



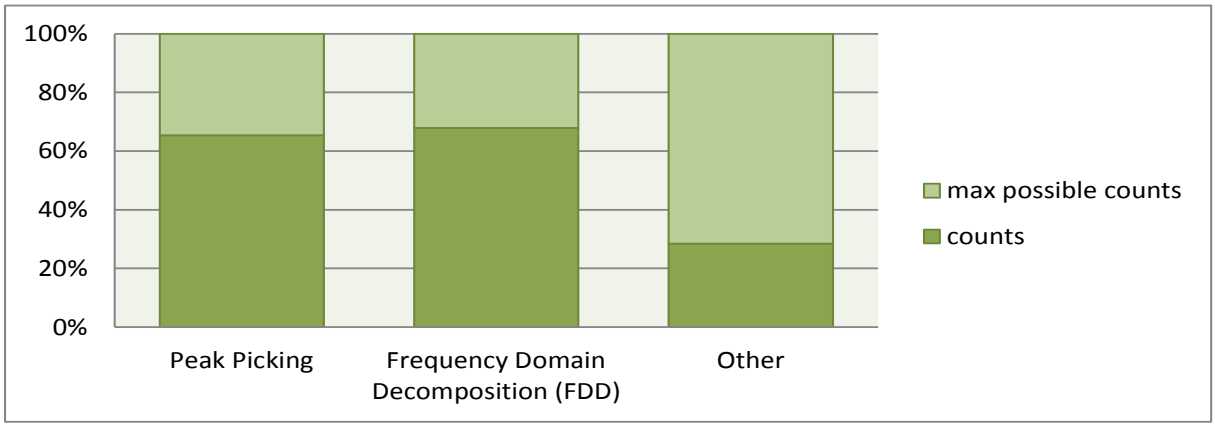

| **Question 15: Which analysis method for data processing do you mainly use?** | |
|---|---|
| | Count |
| Frequency Domain | 81 |
| Time Domain | 62 |
| Amount | 143 |

Maximum possible counts per answer: **24**



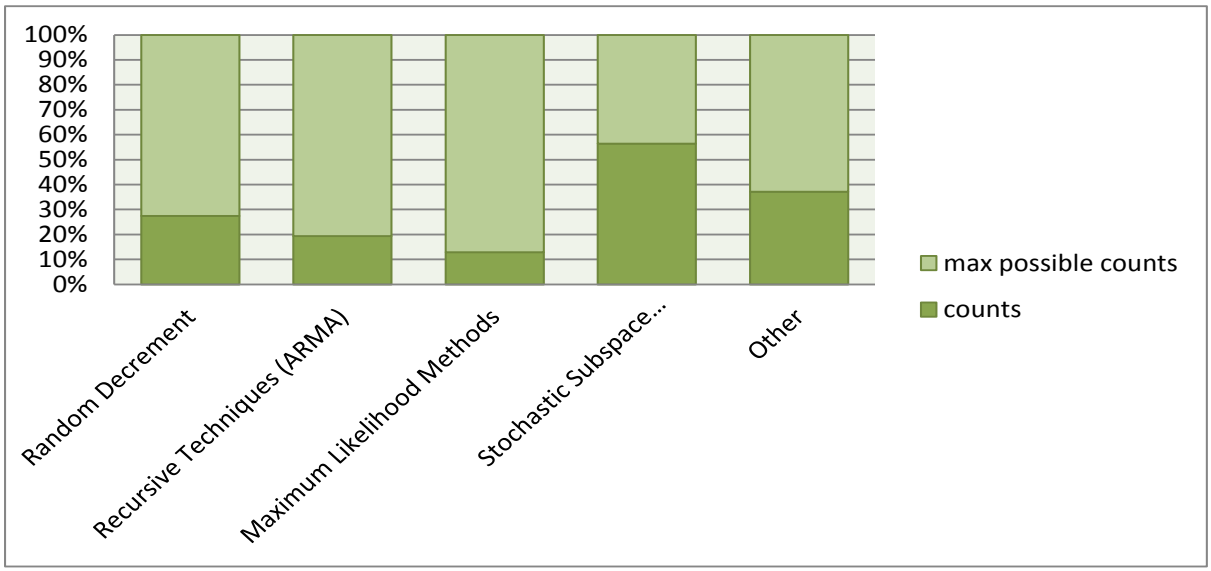
| **Question 15A: If you use Frequency Domain:** | |
|---|---|
| | Count |
| Peak Picking | 53 |
| Frequency Domain Decomposition (FDD) | 55 |
| Other | 23 |
| Amount | 131 |

Maximum possible counts per answer: **81**




| **Question 15B: If you use Time Domain:** | |
|---|---|
| | Count |
| Random Decrement | 17 |
| Recursive Techniques (ARMA) | 12 |
| Maximum Likelihood Methods | 8 |
| Stochastic Subspace Identification Methods (SSI) | 35 |
| Other | 23 |
| Amount | 95 |

Maximum possible counts per answer: **62**

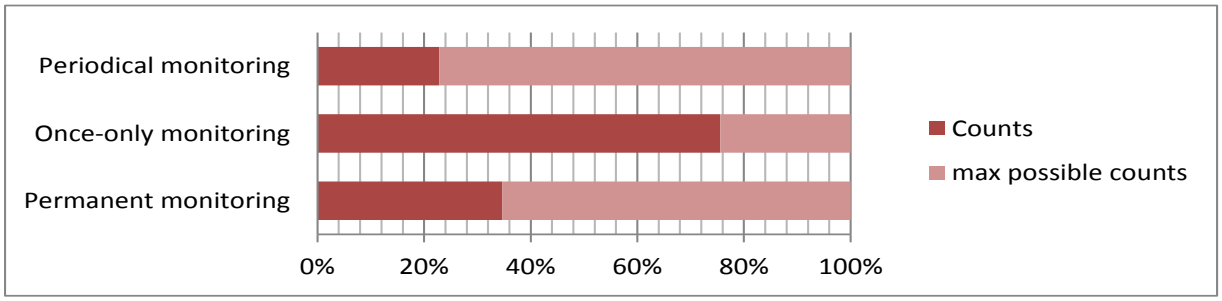


## Section: Buildings

**127 participants completed this section**

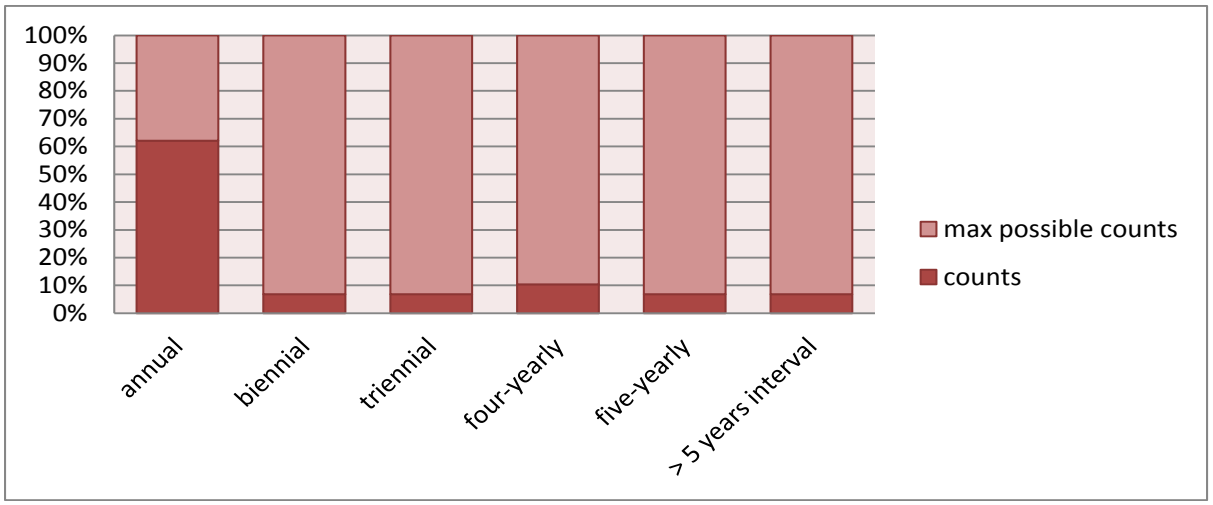
| **Question 16: What is the monitoring rate? (multiple answers are possible)** | |
|---|---|
| | Count |
| Permanent monitoring | 44 |
| Once-only monitoring | 96 |
| Periodical monitoring | 29 |
| Amount | 169 |

Maximum possible counts per answer: **127**



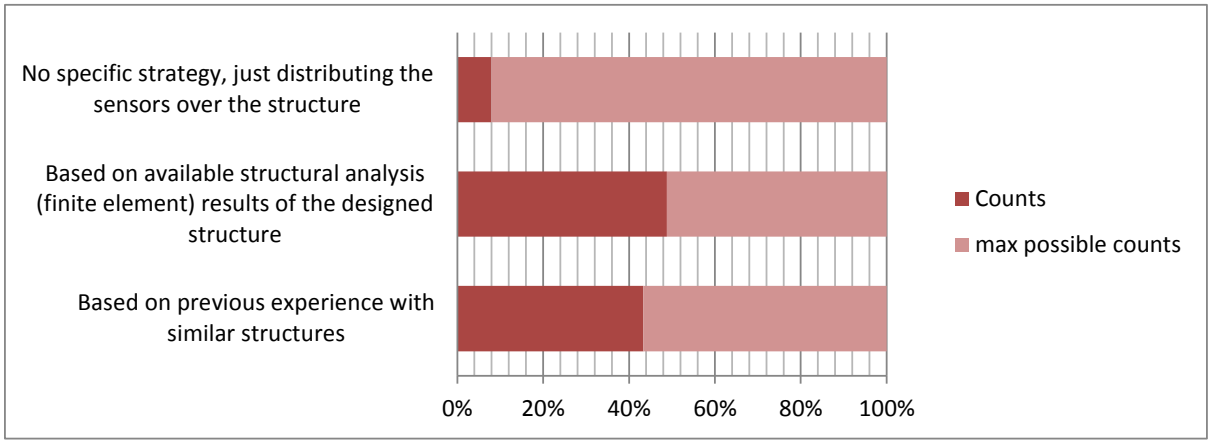

| **Question 16A: If Periodical monitoring:** | |
|---|---|
| | Count |
| annual | 18 |
| biennial | 2 |
| triennial | 2 |
| four-yearly | 3 |
| five-yearly | 2 |
| > 5 years interval | 2 |
| Amount | 29 |

Maximum possible counts per answer: **29**



| **Question 17: How do you design the measurement grid and the instrumentation before the actual field test?** | |
|---|---|
| | Count |
| Based on previous experience with similar structures | 55 |
| Based on available structural analysis (finite element) results of the designed structure | 62 |
| No specific strategy, just distributing the sensors over the structure | 10 |
| Amount | 127 |

Maximum possible counts per answer: **127**



**Question 18: How many Residential buildings have you assessed with permanent field testing?**

Residential buildings permanent

| | Buildings with **<5 sensors** | Buildings with **<10 sensors** | Buildings with **<20 sensors** | Buildings with **<30 sensors** | Buildings with **<40 sensors** | Buildings with **>40 sensors** |
|---|---|---|---|---|---|---|
| steel, reinforced concrete; **number of stories ≤5** | 13 | 10 | 17 | 108 | 1 | 0 |
| steel, reinforced concrete; **number of stories >5** | 26 | 28 | 610 | 26 | 0 | 2 |
| Masonry; **number of stories ≤5** | 5 | 13 | 12 | 30 | 0 | 1 |
| Masonry; **number of stories >5** | 10 | 6 | 0 | 0 | 0 | 1 |
| other | 11 | 10 | 10 | 0 | 0 | 0 |

**Question 19A: How many Industrial buildings have you assessed with permanent field testing?**

Industrial buildings permanent

| | Buildings with **<5 sensors** | Buildings with **<10 sensors** | Buildings with **<20 sensors** | Buildings with **<30 sensors** | Buildings with **<40 sensors** | Buildings with **>40 sensors** |
|---|---|---|---|---|---|---|
| steel, reinforced concrete; **number of stories ≤5** | 14 | 9 | 5 | 0 | 0 | 0 |
| steel, reinforced concrete; **number of stories >5** | 12 | 5 | 2 | 0 | 0 | 5 |
| Masonry; **number of stories ≤5** | 6 | 8 | 5 | 0 | 0 | 0 |
| Masonry; **number of stories >5** | 0 | 0 | 0 | 0 | 0 | 0 |
| other | 10 | 10 | 10 | 0 | 0 | 0 |

**Question 19B: How many Life Line Structures have you assessed with permanent field testing?**

Life Line Structures permanent

| | Buildings with **<5 sensors** | Buildings with **<10 sensors** | Buildings with **<20 sensors** | Buildings with **<30 sensors** | Buildings with **<40 sensors** | Buildings with **>40 sensors** |
|---|---|---|---|---|---|---|
| steel, reinforced concrete; **number of stories ≤5** | 10 | 8 | 24 | 0 | 0 | 0 |
| steel, reinforced concrete; **number of stories >5** | 2 | 7 | 2 | 0 | 0 | 3 |
| Masonry; **number of stories ≤5** | 5 | 11 | 12 | 0 | 0 | 0 |
| Masonry; **number of stories >5** | 0 | 0 | 3 | 0 | 0 | 0 |
| other | 11 | 10 | 11 | 0 | 0 | 0 |

**Question 19C: How many Historical buildings and monuments have you assessed with permanent field testing?**

| ID | Country | Material | Height | Buildings with < 10 sensors | Buildings with < 20 sensors | Buildings with < 30 sensors | Buildings with < 40 sensors | Buildings with > 40 sensors |
|---|---|---|---|---|---|---|---|---|
| 79 | Italy | Marble | 100 | | | 1 | | |
| 96 | Spain | Masonry-church | 15m | | 3 | | | |
| 109 | Italy | Masonry | | | 3 | | | |
| 130 | Italy | Masonry | 90 m | | 2 | | | |
| 166 | Italy | Stone masonry | 37m | 1 | | | | |
| 208 | Italy | Masonry | 12 | | | | | 1 |
| 220 | Switzerland | Masonry | | 2 | 2 | | 2 | 3 |
| 316 | Turkey | Masonry | | | 2 | | | |
| 336 | Turkey | Concrete | 20 | 5 | 5 | | | |
| 369 | Greece | Masonry | 7m-16m | 10 | | | | |
| 390 | Italy | Masonry | | 2 | 6 | | | |
| 394 | Italy | Masonry | | | | 5 | | |
| 130 | Italy | Masonry | 50 m | 2 | | | | |
| 166 | Italy | Brick masonry | 111m | 1 | | | | |
| 336 | Turkey | Masonry | 50 | 5 | 5 | | | |
| 130 | Italy | Masonry | 50m | 2 | 2 | | | |
| 336 | Turkey | Masonry | 30 | 5 | 5 | | | |

**Question 20: How many Residential buildings have you assessed with only-once and periodical field testing?**

Residential buildings only-once/periodical

| | Buildings with **<5 measurement points** | Buildings with **<10 measurement points** | Buildings with **<20 measurement points** | Buildings with **<30 measurement points** | Buildings with **<40 measurement points** | Buildings with **>40 measurement points** |
|---|---|---|---|---|---|---|
| steel, reinforced concrete; **number of stories ≤5** | 400 | 62 | 134 | 8 | 2 | 7 |
| steel, reinforced concrete; **number of stories >5** | 253 | 69 | 99 | 12 | 4 | 2 |
| Masonry; **number of stories ≤5** | 145 | 67 | 37 | 6 | 0 | 1 |
| Masonry; **number of stories >5** | 10 | 15 | 4 | 2 | 9 | 0 |
| other | 10 | 0 | 12 | 0 | 0 | 0 |

**Question 20A: How many Industrial buildings have you assessed with only-once and periodical field testing?**

Industrial buildings only-once/periodical

| | Buildings with **<5 measurement points** | Buildings with **<10 measurement points** | Buildings with **<20 measurement points** | Buildings with **<30 measurement points** | Buildings with **<40 measurement points** | Buildings with **>40 measurement points** |
|---|---|---|---|---|---|---|
| steel, reinforced concrete; **number of stories ≤5** | 40 | 21 | 20 | 23 | 1 | 1 |
| steel, reinforced concrete; **number of stories >5** | 16 | 5 | 6 | 11 | 0 | 0 |
| Masonry; **number of stories ≤5** | 4 | 5 | 0 | 5 | 1 | 1 |
| Masonry; **number of stories >5** | 0 | 0 | 0 | 0 | 0 | 0 |
| other | 8 | 0 | 0 | 0 | 0 | 0 |

**Question 20B: How many Life Line Structures have you assessed with only-once and periodical field testing?**

Life Line Structures only-once/periodical

| | Buildings with **<5 measurement points** | Buildings with **<10 measurement points** | Buildings with **<20 measurement points** | Buildings with **<30 measurement points** | Buildings with **<40 measurement points** | Buildings with **>40 measurement points** |
|---|---|---|---|---|---|---|
| steel, reinforced concrete; **number of stories ≤5** | 33 | 16 | 6 | 8 | 0 | 0 |
| steel, reinforced concrete; **number of stories >5** | 2 | 5 | 2 | 1 | 0 | 0 |
| Masonry; **number of stories ≤5** | 5 | 11 | 5 | 3 | 0 | 0 |
| Masonry; **number of stories >5** | 5 | 7 | 0 | 0 | 0 | 0 |
| other | 5 | 0 | 1 | 0 | 0 | 0 |

**Question 20C: How many Historical buildings and monuments have you assessed with only once and periodical field testing?**

**#1**

| ID | Country | Material | Height | Buildings with < 5 sensors | Buildings with < 10 sensors | Buildings with < 20 sensors | Buildings with < 30 sensors | Buildings with < 40 sensors | Buildings with > 40 sensors |
|---|---|---|---|---|---|---|---|---|---|
| 76 | Bangladesh | Masonry | 20 ft | 10 | | | | | |
| 80 | Italy | Masonry | 15.5 m | 1 | | | | | |
| 96 | Spain | Masonry-belltower | >25 | 15 | 3 | | | | |
| 98 | Peru | Masonry | 20m | 1 | 1 | | | 1 | |
| 109 | Italy | Masonry | | | | 25 | | | |
| 125 | Turkey | Masonry | 12 | 1 | | | | | |
| 143 | Trinidad and Tobago | Masonry | 15 m | | | | 25 | | |
| 166 | Italy | Stone masonry | 37 | | 1 | | | | |
| 181 | France | Masonry | | | 8 | | | | |
| 186 | Taiwan | Brick | 8 m | 2 | | | | | |
| 207 | Switzerland | Stone masonry | 13 | | | | | 1 | |
| 208 | Italy | Natural stones | 7 | | | | | | 1 |
| 233 | Austria | Stone | 20 | | | 2 | | | |
| 285 | Greece | Marble | 20m | 1 | | | | | |
| 292 | Italy | Masonry | 50 | | | 1 | | | |
| 298 | FYROM | Masonry | 5-15m | ~20 | | | | | |
| 312 | Croatia | Masonry | 12 | 5 | | | | | |
| 317 | Croatia | Stone masonry | 40 | 1 | | | | | |
| 326 | Japan | Wood | 6m | 8 | | | | | |
| 335 | FYROM | Marble | 40m | | | 2 | | | |
| 361 | Switzerland | Natural stone | 60 | | | 3 | | | |
| 389 | France | Masonry | | | 8 | | | | |
| 390 | Italy | Masonry | | | 15 | | | | |
| 399 | Italy | Masonry | 30 | | 1 | | | | |

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 409 | United Kingdom | Masonry | 20m | | 1 | | | | |
| 416 | Spain | Masonry | 100 | | | 1 | | | |

**#2**

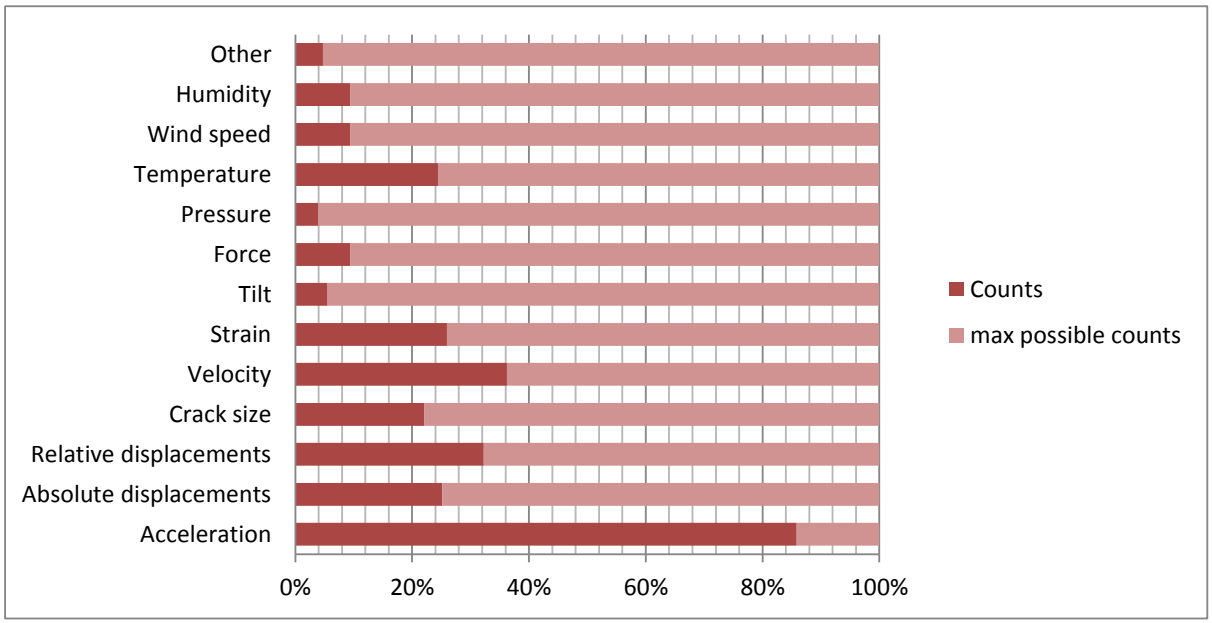
| ID | Country | Material | Height | Buildings with < 5 sensors | Buildings with < 10 sensors | Buildings with < 20 sensors | Buildings with < 30 sensors | Buildings with < 40 sensors | Buildings with > 40 sensors |
|---|---|---|---|---|---|---|---|---|---|
| 80 | Italy | Masonry | 12.2 m | 1 | | | | | |
| 96 | Spain | Masonry-towers | >25 | 10 | | | | | |
| 98 | Peru | Masonry | 10m | | | | | 2 | |
| 125 | Turkey | Masonry | 10 | 1 | | | | | |
| 125 | Turkey | Masonry | 10 | 1 | | | | | |
| 166 | Italy | Brick masonry | 111 | | 1 | | | | |
| 208 | Italy | Masonry | | | | | | 1 | |
| 233 | Austria | Masonry | | | | | 20 | | |
| 292 | Italy | Masonry | 35 | | | 1 | | | |
| 317 | Croatia | Stone masonry | 10-25 | 10 | | | | | |
| 326 | Japan | Wood | 15m | 1 | | | | | |
| 335 | FYROM | Masonry | up to 100m | | | | 20 | | |

**#3**

| ID | Country | Material | Height | Buildings with < 5 sensors | Buildings with < 10 sensors | Buildings with < 20 sensors | Buildings with < 30 sensors | Buildings with < 40 sensors | Buildings with > 40 sensors |
|---|---|---|---|---|---|---|---|---|---|
| 166 | Italy | Brick-stone masonry | | | 5 | | | | |
| 292 | Italy | Masonry | 40 | | | 1 | | | |
| 335 | FYROM | Stone masonry | 20-30m | | | 18 | | | |

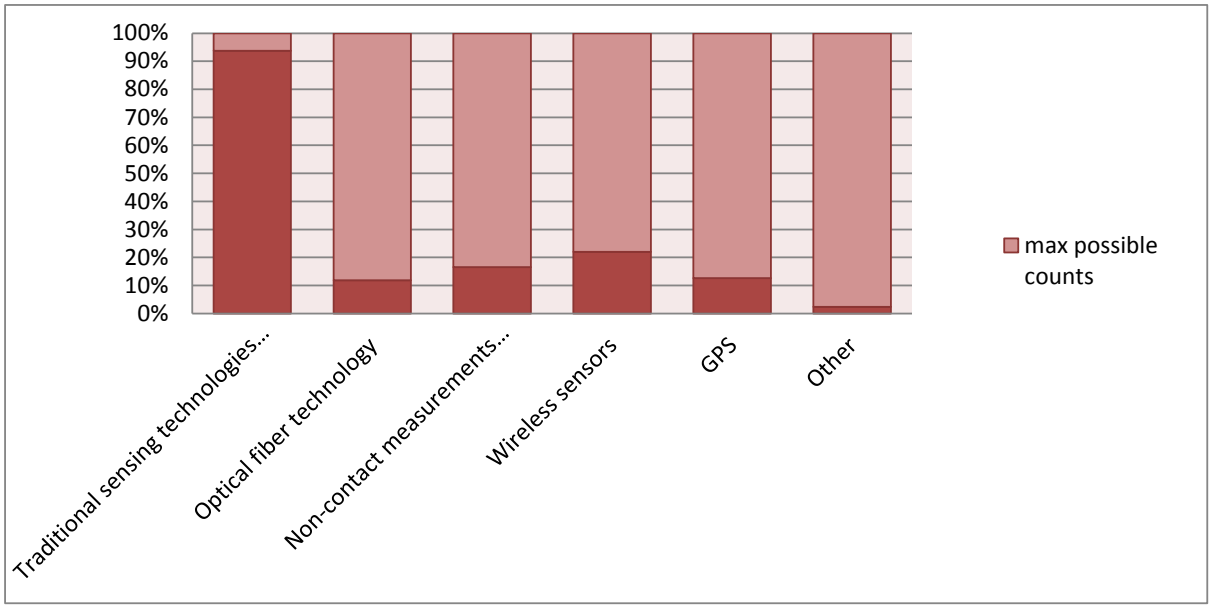
| **Question 21: Which parameters are monitored?** (mandatory) (multiple answers are possible) | |
|---|---|
| | Count |
| Acceleration | 109 |
| Absolute displacements | 32 |
| Relative displacements | 41 |
| Crack size | 28 |
| Velocity | 46 |
| Strain | 33 |
| Tilt | 7 |
| Force | 12 |
| Pressure | 5 |
| Temperature | 31 |
| Wind speed | 12 |
| Humidity | 12 |
| Other | 6 |
| Amount | 374 |

| Maximum possible counts per answer: | **127** |
|---|---|



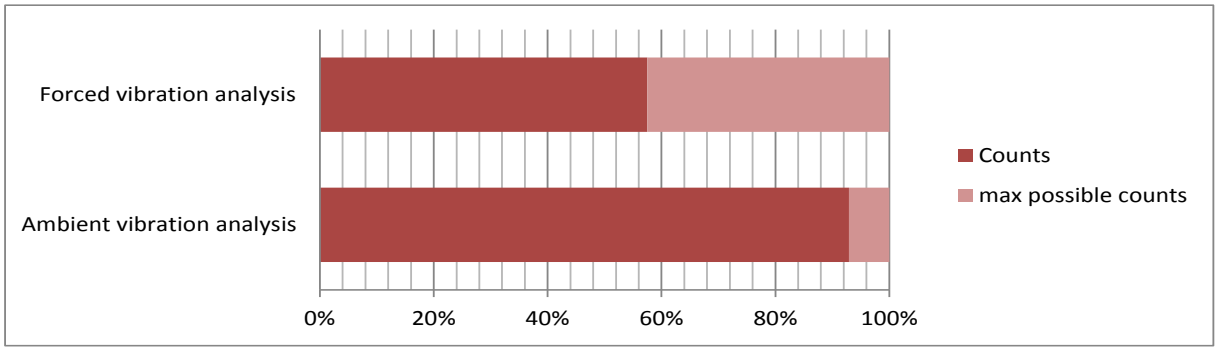
| **Question 22: Which sensor technologies are in use?** (mandatory) | |
|---|---|
| | Count |
| Traditional sensing technologies (electric signals) | 119 |
| Optical fiber technology | 15 |
| Non-contact measurements (Microwave interferometry, Laser, etc.) | 21 |
| Wireless sensors | 28 |
| GPS | 16 |
| Other | 3 |
| Amount | 202 |

Maximum possible counts per answer: **127**



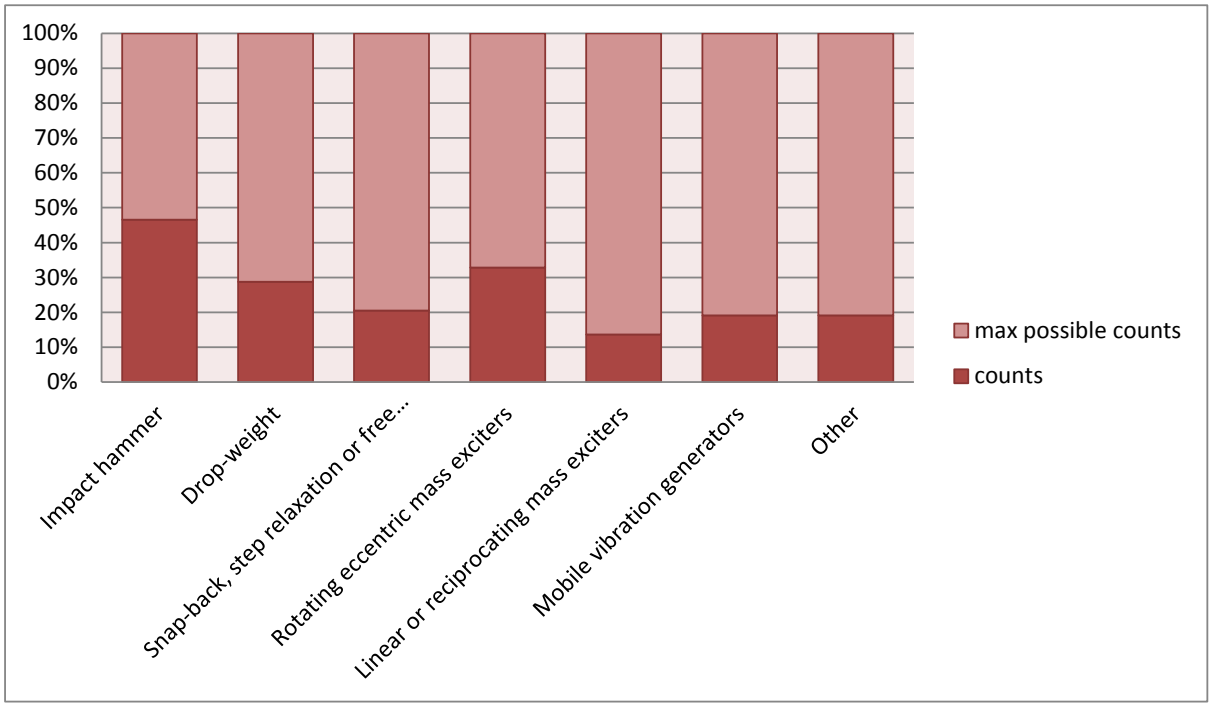

| **Question 23: Which excitation do you use for field testing measurements?** (multiple answers are possible) | |
|---|---|
| | Count |
| Ambient vibration analysis | 118 |
| Forced vibration analysis | 73 |
| Amount | 191 |

Maximum possible counts per answer: **127**



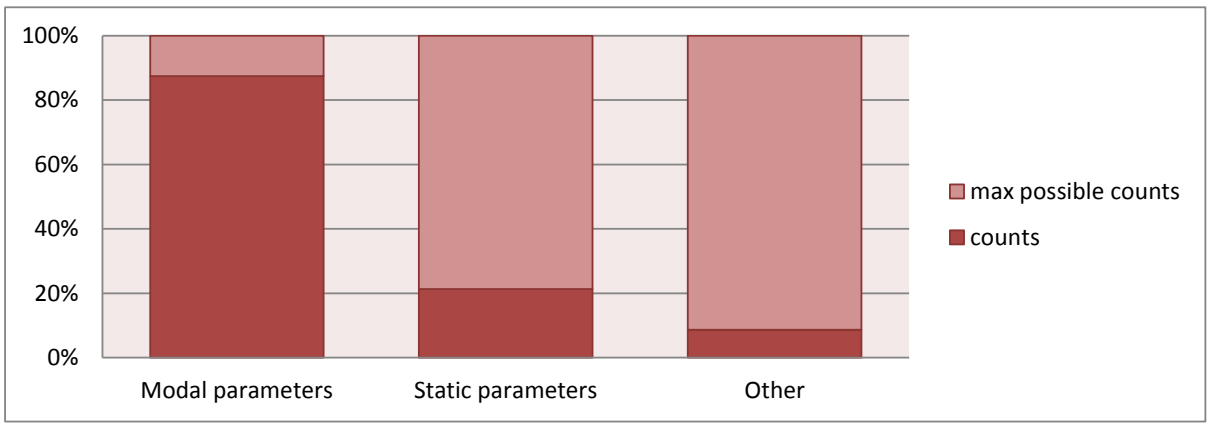
| **Question 23A: If you use forced vibration analysis, please specify:** | |
|---|---|
| | Count |
| Impact hammer | 34 |
| Drop-weight | 21 |
| Snap-back, step relaxation or free vibration | 15 |
| Rotating eccentric mass exciters | 24 |
| Linear or reciprocating mass exciters | 10 |
| Mobile vibration generators | 14 |
| Other | 14 |
| Amount | 132 |

Maximum possible counts per answer: **73**



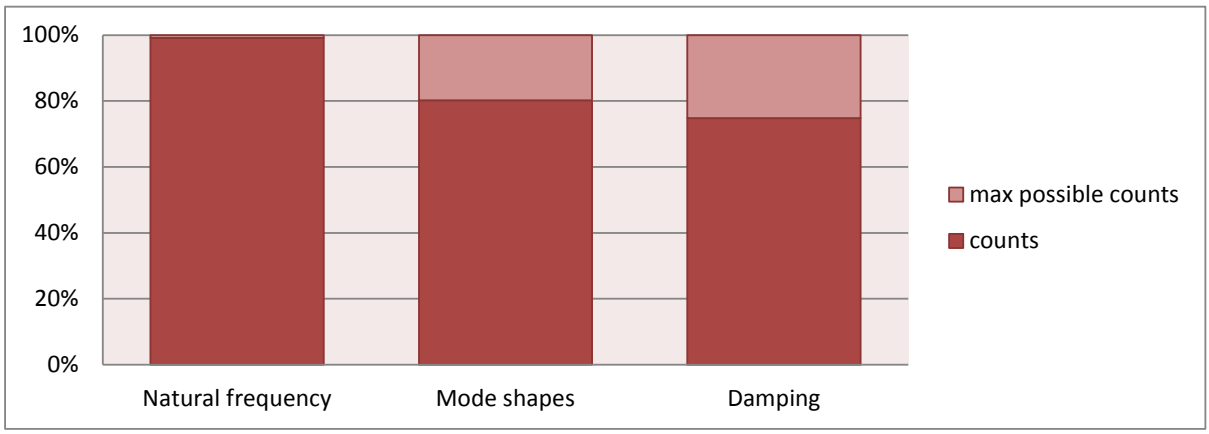

| **Question 24: Which structural parameters do you extract from measurement data?** | |
|---|---|
| | Count |
| Modal parameters | 111 |
| Static parameters | 27 |
| Other | 11 |
| Amount | 149 |

Maximum possible counts per answer: **127**



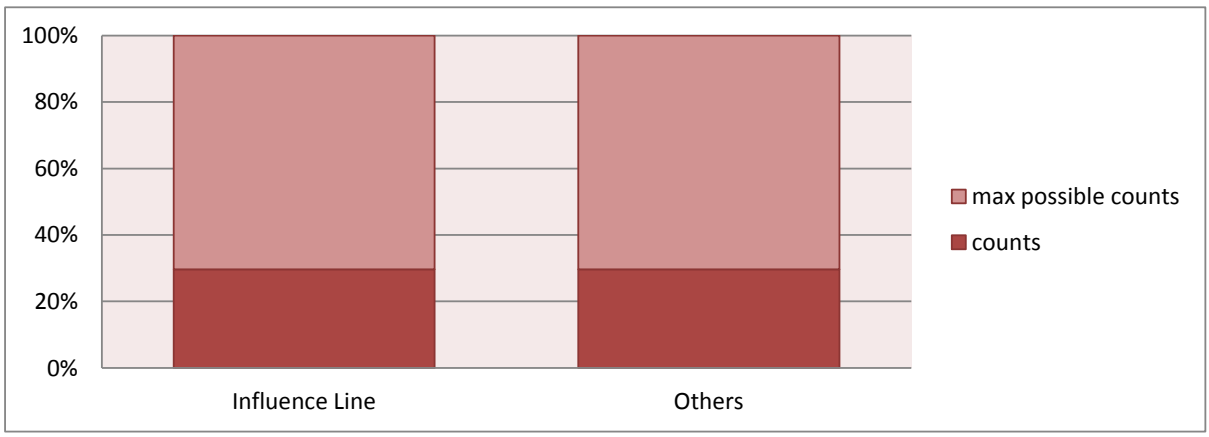

| **Question 24A: If you use Modal parameters:** | |
|---|---|
| | Count |
| Natural frequency | 110 |
| Mode shapes | 89 |
| Damping | 83 |
| Amount | 282 |

| Maximum possible counts per answer: | **111** |
|---|---|



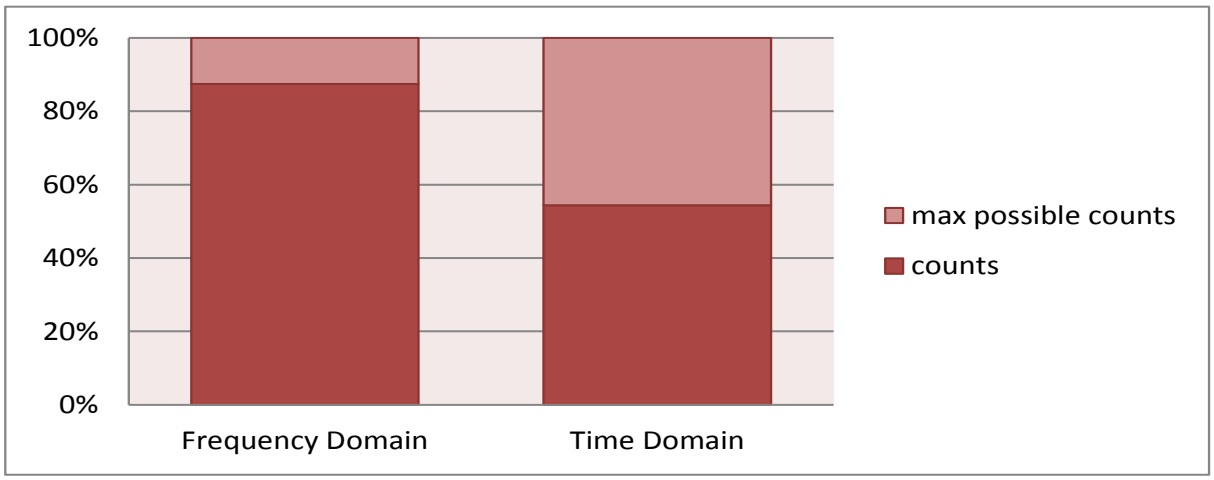

| **Question 24B: If you use Static parameters:** | |
|---|---|
| | Count |
| Influence Line | 8 |
| Others | 8 |
| Amount | 16 |

| Maximum possible counts per answer: | **27** |
|---|---|



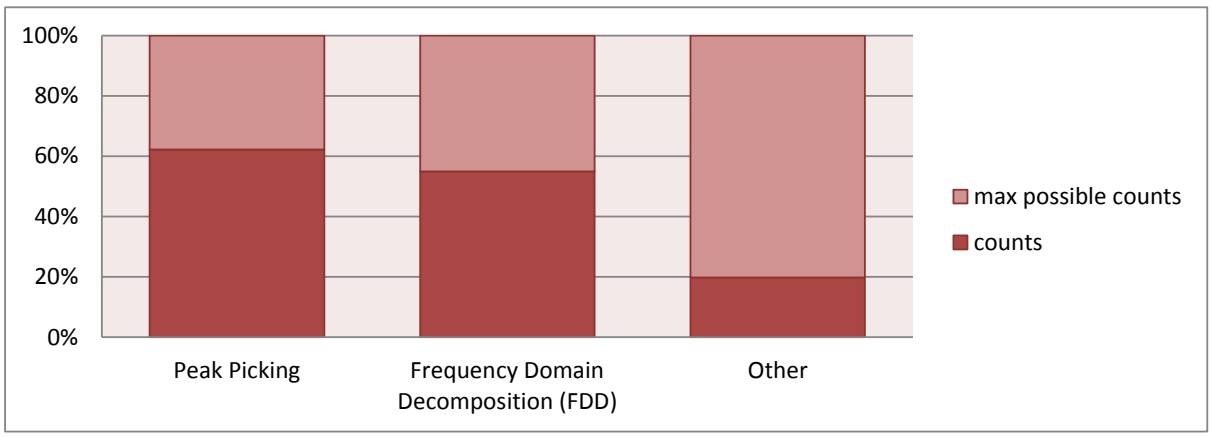

| **Question 25: Which analysis method for data processing do you mainly use?** | |
|---|---|
| | Count |
| Frequency Domain | 111 |
| Time Domain | 69 |
| Amount | 180 |

| Maximum possible counts per answer: | **127** |
|---|---|



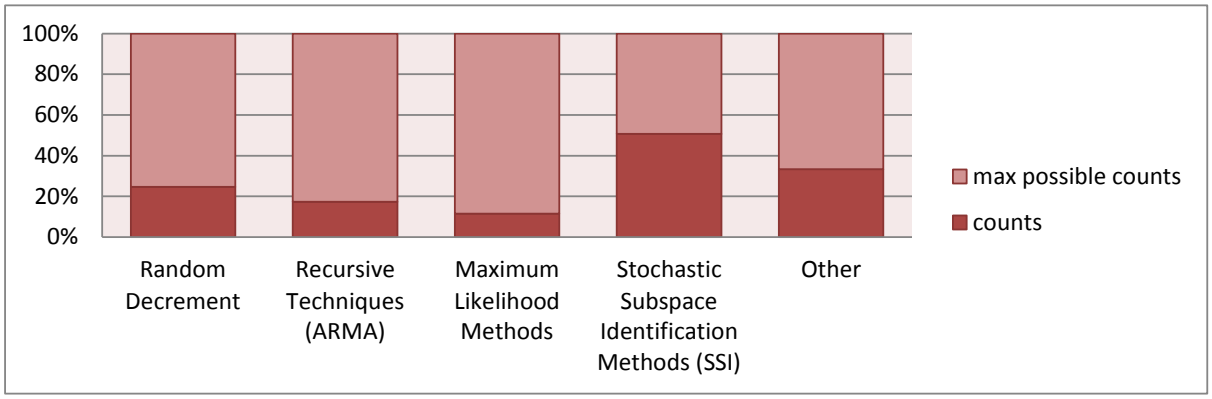

| **Question 25A: If you use Frequency Domain:** | |
|---|---|
| | Count |
| Peak Picking | 69 |
| Frequency Domain Decomposition (FDD) | 61 |
| Other | 22 |
| Amount | 152 |

| Maximum possible counts per answer: | **111** |
|---|---|




| **Question 25B: If you use Time Domain:** | |
|---|---|
| | Count |
| Random Decrement | 17 |
| Recursive Techniques (ARMA) | 12 |
| Maximum Likelihood Methods | 8 |
| Stochastic Subspace Identification Methods (SSI) | 35 |
| Other | 23 |
| Amount | 95 |

| Maximum possible counts per answer: | **69** |
|---|---|



**Question 26: For field testing, what types of sensors and how many in total are used in your company?**

| ID | Country | Vibration sensors | Geophones | Displacement transducers | Temperature sensors | Strain gauges | Load cells | Inclinometer | Acoustic sensors | Ultrasonic sensors | Wind speed sensors | Humidity sensors | Others |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 79 | Italy | >40 | | | | | | | | | | | |
| 94 | Chile | >40 | | | >40 | >40 | | | | | | | |
| 97 | Portugal | >40 | | | | >40 | | | | | | | |
| 104 | Denmark | >40 | >40 | | | >40 | | | | | | | |
| 109 | Italy | >40 | | | | | | | | | | | |
| 115 | China | >40 | | >40 | | >40 | | | >40 | | | | |
| 132 | Belgium | >40 | | | | | | | | | | | |
| 141 | United States | >40 | | | | | | | | | | | |
| 187 | Saudi Arabia | >40 | | | | | | | | | | | |
| 189 | United States | >40 | | | | | | | | | | | |
| 212 | Turkey | >40 | | | | | | | | | | | |
| 236 | Turkey | >40 | | | | | | | | | | | |
| 240 | Japan | >40 | | | | | | | | | | | |
| 250 | Belgium | >40 | | | | | | | | | | | |
| 284 | France | >40 | | | | >40 | | | | | | | |
| 316 | Turkey | >40 | | | | | | | | | | | |
| 334 | France | >40 | >40 | | | | | | | | | | |
| 368 | Romania | >40 | | | | | | | | | | | |
| 386 | Greece | >40 | | | | | | | | | | | |
| 390 | Italy | >40 | | >40 | | >40 | >40 | | | | | | |
| 394 | Italy | >40 | | | | | | | | | | | |
| 333 | Pakistan | | | | | >40 | | | | | | | |
| 324 | Turkey | | | >40 | | >40 | | | | | | | |
| 296 | United States | | | | | >40 | | | | | | | |
| 263 | Germany | | | | | >40 | | | | | | | |
| 246 | Slovenia | | | | | >40 | | | | | | | |
| 242 | United Kingdom | | | | | >40 | | | | | | | |
| 220 | Switzerland | | | >40 | >40 | >40 | | >40 | | | | | >40 |
| 208 | Italy | | >40 | | >40 | >40 | | >40 | | | | | |
| 122 | United States | | | >40 | >40 | >40 | | | | | | | |
| 73 | Spain | | | | | >40 | | | | | | | |
| 181 | France | | >40 | | | | | | | | | | |
| 366 | Italy | | >40 | | | | | | | | | | |
| 214 | Portugal | | | >40 | | | | | | | | | |
| 269 | Netherlands | | | | >40 | | | | | | | >40 | |
| 162 | Austria | | | | | | | | >40 | | | | |
| 157 | Italy | | | | | | | | | | | | >40 |

## Section: Final

**154 participants completed this section**

| **Question 27: Are there national standards or guidelines in your country that impose field testing as a tool for Earthquake Engineering?** | |
|---|---|
| | Count |
| Yes | 27 |
| Under development | 23 |
| No | 101 |
| Amount | 151 |

| **Question 27A: If yes, please give the reference:** | |
|---|---|
| | Count |
| National standards regarding field testing for bridges | 12 |
| National standards regarding field testing for buildings | 14 |
| National standards regarding field testing for other structures | 2 |
| Other | 8 |
| Amount | 36 |

**Question 27B-D National standards regarding field testing for bridges / buildings / other structures:**

| | | Bridges | Bridges | Buildings | Buildings | Other Structure | Other Structure | Other Structure |
|---|---|---|---|---|---|---|---|---|
| ID | Country | Issued by | Enforcement year | Issued by | Enforcement year | Type | Issued by | Enforcement year |
| 104 | Denmark | VD | ? | | | | | |
| 115 | China | Ministry of Railways | 2004 | | | | | |
| 166 | Italy | Ministero dei lavori pubblici | 1990 and 2008 | Ministero dei lavori pubblici | 1996 and 2008 | | | |
| 172 | Italy | Italian government | ns | Italian government | na | | | |
| 209 | Russia | СНиП* 11-7-81 «Строительство в сейсмических районах» Минстрой –М., 2002 | 2002 | | | | | |
| 221 | Slovakia | SUTN Bratislava, SK | 1993 -1995 | SUTN Bratislava, SK | 1993 -1995 | Calculation of building structures loaded by dynamic effect of machines, STN 73 0032s | SUTN Bratislava, SK | 1993 -1995 |
| 284 | France | ? | ? | ? | ? | | | |
| 333 | Pakistan | | | Pakistan Standards and Quality Control Authority | 2006 | | | |
| 345 | Germany | DIN | 2000 | DIN | 2000 | | | |
| 348 | Italy | | | Ministro Infrastrutture e Intero | 2008 | | | |
| 364 | Austria | | | ASI | 20xx | | | |
| 368 | Romania | | | p100/ | 2006 | | | |
| 387 | Germany | DIN 2222 | | DIN | 2222 | | | |
| 392 | Italy | Ministero dei Lavori Pubblici | 2008 | Ministero dei Lavori Pubblici | 2008 | | | |
| 395 | Italy | Government | 2008 | Government | 2008 | | | |
| 439 | France | | | ec | 2008 | | | |
| 460 | Vietnam | Ministry of Transport | 1998 | Ministry of Construction | 1990- 2011 | Dyke, Dam | Ministry of Agriculture and Rural Development | 1990- 2011 |

**Question 27E: If the development of guidelines is still under progress, please briefly describe the content and goal of the efforts**

| ID | Country | Answer |
|---|---|---|
| 112 | United States | Nuclear NRC |
| 138 | Turkey | Bridge and tall building guidelines |
| 161 | Turkey | Especially after the guide for a seismic design of high rise buildings was proposed. Since SHM will become essential tool for operational purpose. It is aimed to fill the gap in design and application on the site supplying the results from structural tests as well as for operation, early warning and detecting any progressive damage in early stage (SHM). |
| 297 | Italy | Using the results obtained during the DPC-RELUIS 2008-2010 and DPC-RELUIS 2010-2013 projects, the guidelines about the field testing on structures and buildings are developing. |
| 324 | Turkey | The development of guidelines under progress |
| 326 | Japan | How to handle the equipments safety during the field testing range of application data acquisition data analysis |

**Question 27F: If not, are there voluntary standards or regulations issued by national authorities or associations that supply these approaches? (Please give the reference)**

| ID | Country | Answer |
|---|---|---|
| 63 | Austria | National code for conducting vibration measurements |
| 86 | Taiwan | None |
| 122 | United States | unsure |
| 143 | Trinidad and Tobago | This procedure is still in the infancy in all Latin American countries. |
| 145 | Argentine | There is any type of standard or regulations |
| 155 | Turkey | The standards offered by USGS |
| 189 | United States | ASCE is printing a Committee Report on Structural Identification of Constructed Facilities including buildings and bridges. This should be available soon and an e-copy may be viewed. |
| 237 | Iran | Dynamics Behavior of Structures Books, Papers -Existing research works, as My MSc Thesis , "Investigation on Dynamic Behavior of structures with semi-rigid beam-to-column connections" -FEMA guidelines -ASCE guideline for seismic evaluation of the existing building, -Americal Lifeline Alignment -etc. |
| 239 | Switzerland | None |
| 266 | Indonesia | Usually we refer to USA codes or ISO standards |
| 279 | Germany | N/D |
| 292 | Italy | DPCM Presidente del Consiglio dei Ministri 2011 |
| 294 | United States | not sure, I work in wind engineering |
| 298 | FYROM | no, internal and worldwide recommended procedures |
| 300 | Spain | European Standars |
| 303 | Romania | no |
| 312 | Croatia | ENV 1998-3 |
| 323 | Taiwan | no regulation issued by national authorities |
| 363 | Peru | For the moment, not yet |
| 388 | Spain | I don't know |
| 389 | France | I don't know |

| | | |
|---|---|---|
| 409 | United Kingdom | US standards are usually the ones I find most useful |
| 428 | Japan | Guideline of monitoring issued by Japan Society of Civil Engieers (JSCE) |
| 452 | Germany | don't know |

## Question 28: Are there any needs for Information and Training for field testing?

| ID | Country | Answer |
|---|---|---|
| 68 | Germany | Offshore instrumentation. Wireless measurement techniques. Optical sensor technique. Data acquisition esp. for high frequencies. |
| 76 | Bangladesh | Thank you for this questionnaire |
| 96 | Spain | It is important to stay current on new techniques existing in the field of monitoring structures. It is even important to have retraining activities and share experiences with experts in the structural monitoring. On numerous occasions the collaborative activity between institutions can improve both knowledge of a particular study. |
| 98 | Peru | There are lack of studies and experts on experimental testing in Latin America, besides our need of new techniques to assess the behaviour of existent structures. All the previous information I filled up in the questionnaire have done during my PhD studies in Europe which were focused precisely in Dynamic Monitoring. |
| 112 | United States | We need to know and understand what instuments can do and their limitations. |
| 114 | Italy | It is necessary to inform the customer about information that experimental tests can produce, above all concerning damping measurements (also for different level of excitation). It would be interesting to develop a guide about Field Testing (probably it already exists, I don't know for my ignorance, sorry). |
| 121 | Italy | European Standardisation |
| 122 | United States | I am not representing a company, rather a university research program targeting testing of structures and structural elements (e.g. piles, footings) under dynamic loading. |
| 132 | Belgium | We would like to inquire how the vibrational field testing applies in different countries and regions. |
| 143 | Trinidad and Tobago | IT WOULD BE DESIRABLE TO HAVE A STANDARD PROCEDURE AND TRAINING TO DO EARTHQUAKE ENGINEERING TESTS IN THE FIELD, EVERYBODY IS DOING THE TEST ACCORDING TO PAST EXPERIENCES OR WHAT THEY BELIEVE IS ENOUGH TO ACCOMPLISH THE JOB |
| 144 | Indonesia | I think this kind of training it very important and useful because with field testing we can obtain real time global information about the building condition. But unfortunately there is no codes about this matter. That's why a training from from the experienced is needed. |
| 155 | Turkey | I want to learn and apply the more easy and efficient field testing methods |
| 161 | Turkey | information is always essential as developments in the test market grove. |
| 166 | Italy | Politecnico is organising continuing education courses within this topic. |
| 186 | Taiwan | A standard will be helpful. |
| 189 | United States | Field testing is a combination Neurosurgery and Battle |
| 233 | Austria | There is a need for Conultants to know abaout recent developments |
| 239 | Switzerland | Training in simplified modal testing without extremely sophisticated instrumentation (for which, usually not enough money is available in practice) |
| 246 | Slovenia | New measurement technologies, wireless sensor networks, modal analysis. |
| 266 | Indonesia | Currently we are interesting to install the structural monitoring system in tall building |
| 293 | Australia | Instruction in basic dynamics |

| | | |
|---|---|---|
| 296 | United States | Incorporate in courses |
| 323 | Taiwan | Need information from other countries (such as Standard). |
| 324 | Turkey | calibration of sensors;guidelines for influance of noise depending of sensor type;guidelines for influance of noise depending of cable lenght;classification of ambient vibration rang for applicability examined existing structure;sensors' fixetion particularites for historical structuras; unification and standartization request to the system identificatin and upgating methods(and appropraet software);data importablity and exportablity between vearios software; particularites of non linear analysis and so on |
| 334 | France | Need for field testing without any commercial background ! Just presenting the present possibilities and limitations |
| 335 | FYROM | Damage detection of structures based on forced and ambient vibration tests. Exchange of excperience is also wellcomed and appriciated. |
| 348 | Italy | Information: dissemination of knowledge would help in i) the relationships with owners of structures ii) use of new technologies; |
| 399 | Italy | In Italy, undergraduate university courses give very good bases on earthquake engineering, but as a conunterpart do not provide as many information on lab and field testing |
| 409 | United Kingdom | Training is always required, of course. |
| 422 | United States | familiarity and expertise in electronic test equipment |
| 425 | Greece | notes on usage and applicability |
| 450 | Belgium | This depends on future legislation. |
| 460 | Vietnam | -We would like to receive and learn new Specifications |

# 5 Case studies

## *5.1 Forced vibration testing of bridges*

### *5.1.1 Quick facts*

| | |
|---|---|
| Bridge type: | Arch road bridge |
| Objective: | Updated computational model for safety assessment |
| Number of measurement points: | 28 |
| Number of excitation points: | 2 |
| Sensors / frequency range: | accelerometers / 0.05-500 Hz |
| Exciter / frequency range: | hydraulic mass reaction exciter / 0-80 Hz |
| Used excitation types: | sine-sweep, ambient |
| System identification method: | combined deterministic-stochastic subspace identification |

### *5.1.2 Bridge description*

The structure is an arch bridge with a total length of 157 m. Two arches with 69 m span and 18.3 m height are made of Ultra-High Performance Fibre-Reinforced Concrete UHPFRC 165/185. Each arch consists of 5 straight segments connected with joint segments (Fig. 2). The elements were prefabricated, mounted on site and post-tensioned. The high-strength material allowed slender cross section with regular wall thickness of 6 cm. The arches carry a reinforced concrete deck with two road lanes and a footpath. The bridge (Fig. 3) was opened to traffic in October 2010 and is the first arch bridge utilizing UHPFRC in Austria.

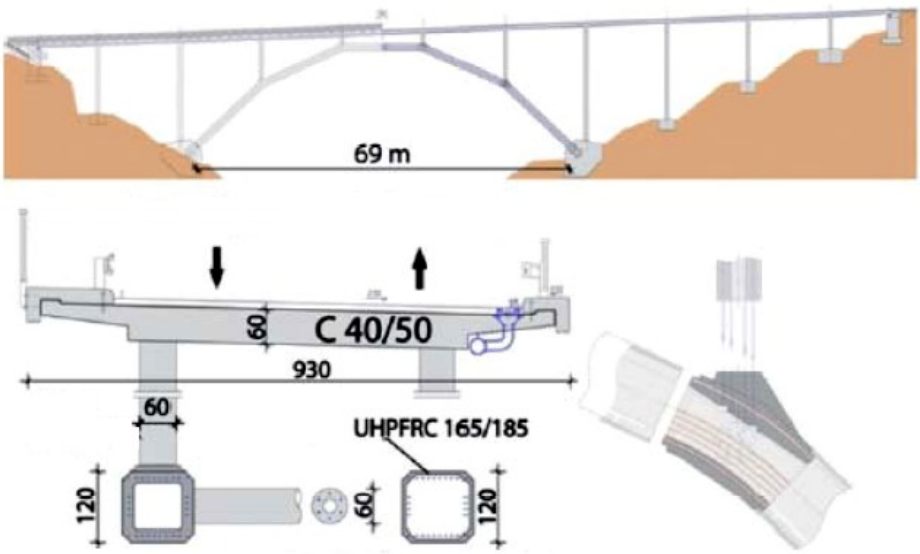



Fig. 2. Bridge geometry

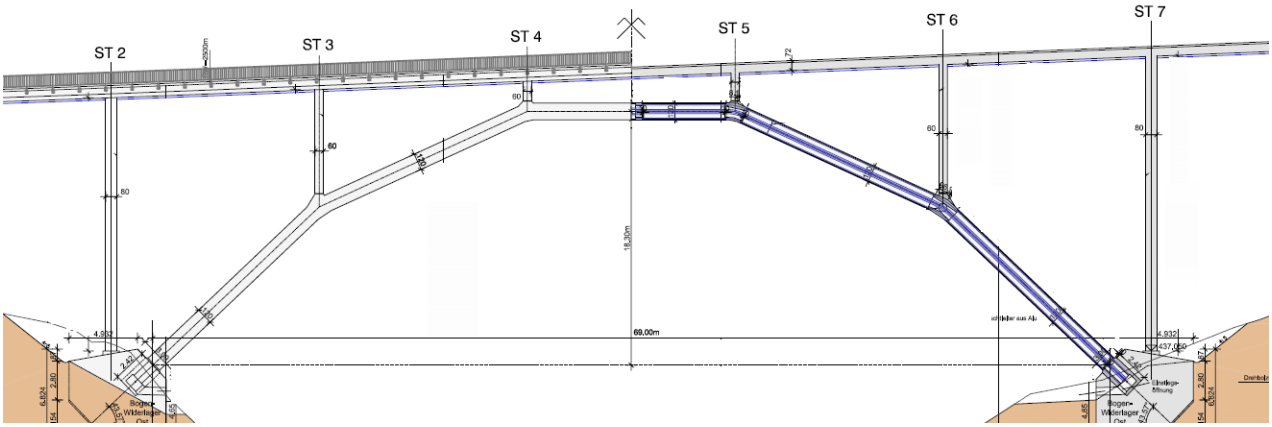
Fig. 3. Bridge "Wild" in Völkermarkt, Austria

### *5.1.3 Objective*

Objective of the test is to investigate real dynamic properties of the bridge. In further steps, the measurement results should be used to improve computational model of the bridge in order to aid safety assessment and life-cycle management of the bridge.

### *5.1.4 Methodologies of measurement*

The measurement was carried out as forced vibration test. Excitation was produced by a hydraulic mass reaction exciter. The position of excitation and measurement points is displayed in Fig. 4. Two excitation positions (marked with 'F1', 'F2') and 28 measurement points (marked with red circles) were planned. The measurement grid was different for the two bridge sides. On one side (points 1-20) the grid was dense, with regular spacing of 5 m and in two regions the spacing was reduced to 2.5 m (Fig. 5) for better recognition of higher modes. On the other side (points 51-70), only few points were measured to capture the torsional behavior. Points 52, 57, 58, 61, 65, and 69 were measured and other points in 51-70 range were interpolated in the evaluation. Excitation position was eccentric in the cross-section, so that torsional modes would also be excited.



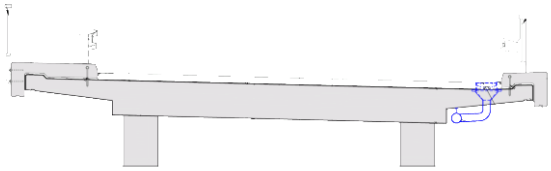


Fig. 4. Measurement setup

The measurement was carried out using 7 sensors in four setups. Additionally, a reference sensor was placed near position 'F2' during excitation at 'F1' and vice versa. We have avoided placing reference sensor near the excitation point because of local effects of excitation force. The measured directions were lateral (Y) and vertical (Z). Longitudinal direction (X) was measured only at 2 points. The sensors were high-sensitivity (10 V/g) accelerometers (Fig. 7) with a frequency range of 0.05-500 Hz and amplitude range ±0.5g.

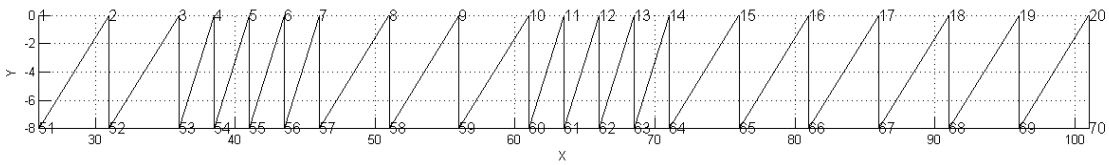



Fig. 5. Measurement point grid

The excitation was introduced as slow sine-sweep in frequency range 0.5 – 40 Hz. Measurement time in one setup was 5-10 minutes. Additionally, ambient vibrations have been measured. Sampling frequency was 512 Hz.

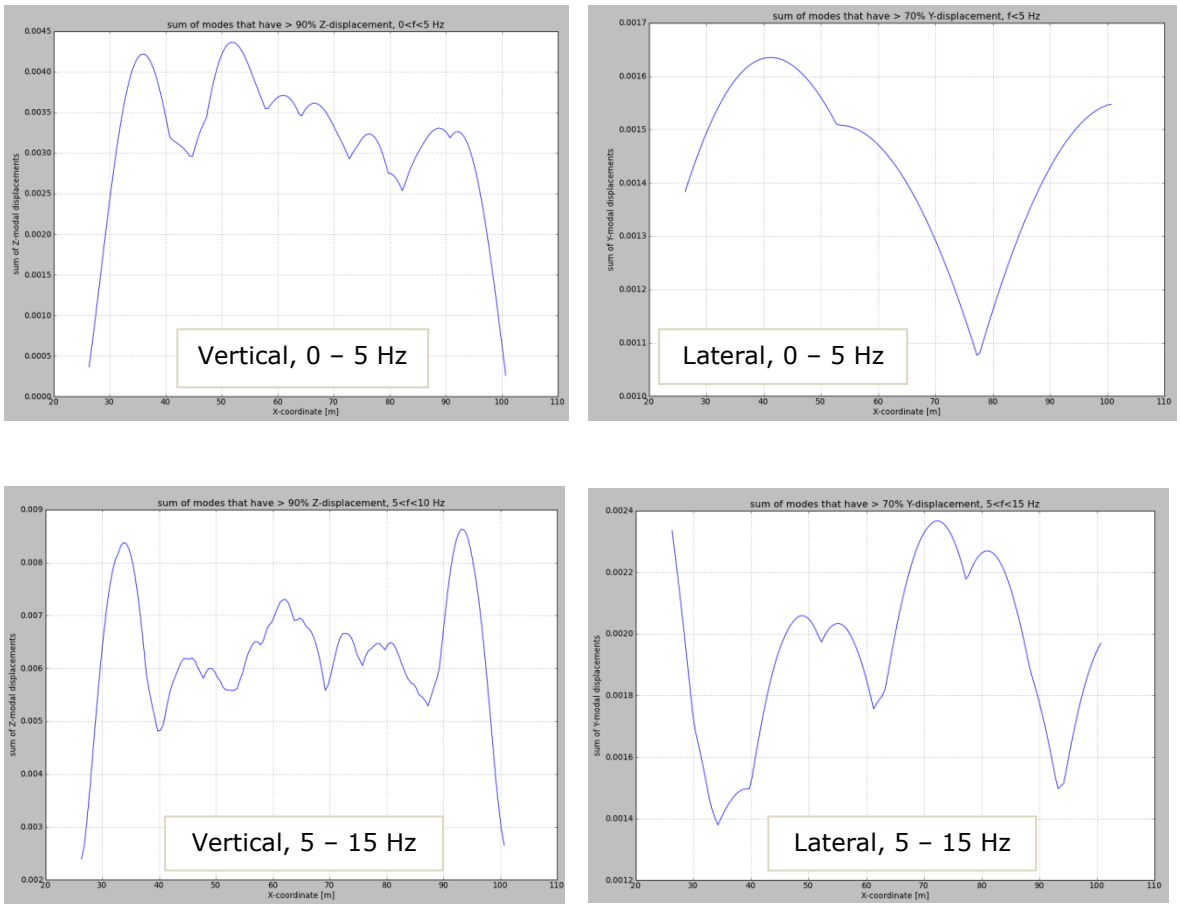
The reaction mass exciter utilizes a reaction mass of 1432 kg (Fig. 6) that moves at max. ±125mm displacement and can produce a force up to 30 kN (depending on frequency).

Fig. 6. Exciter on the bridge (left) and reaction masses detail (right)

The excitation direction was vertical. Although the design of the exciter would also allow horizontal excitation, it was not used in this case because doweling of the exciter to the bridge would be required.

Fig. 7. Accelerometers on the bridge

Design of the measurement setup was done using preliminary calculation of a finite-element model. Computational modal analysis of the model showed a large number of eigenmodes: 22 global eigenfrequencies in the range 0 – 13 Hz, and several local eigenfrequencies. The aim of ideal placing of excitation and measurement points is to avoid nodal points and to choose places with high amplitudes at possibly all modes. Placement of excitation and reference points are especially important. For better illustration of suitable positions, a sum of mass-normalized mode shape amplitudes has been calculated (Fig. 8).



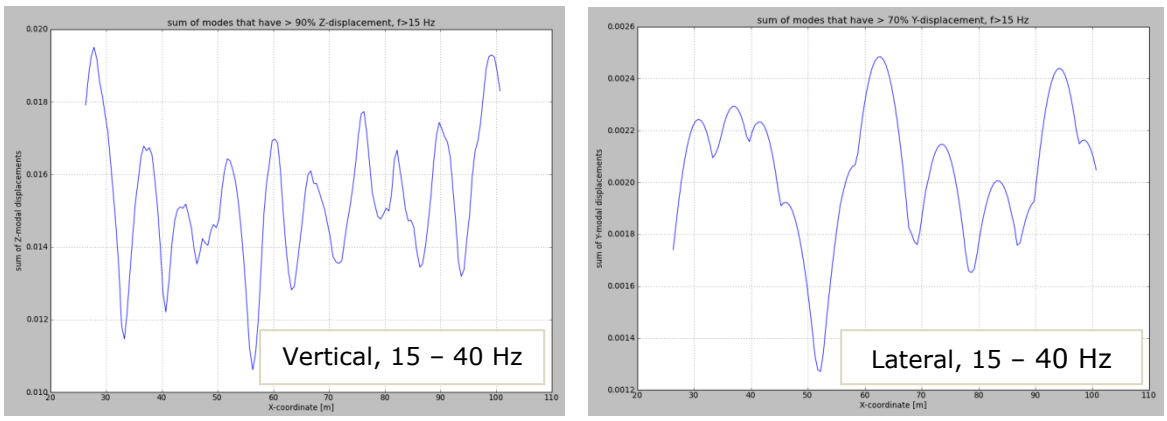



Fig. 8. Sum of mass-normalized mode shape amplitudes

Additionally, important modes (first three vertical, lateral and torsional modes) have been checked individually. From these considerations, excitation and reference point positions were fixed at coordinates x=52 m for 'F1' and x=77 m for 'F2' position.

### *5.1.5 System Identification Methods*

The forced vibration test data was evaluated using reference-based combined deterministic-stochastic subspace identification (Reynders et al. 2008b). The reference sensor placed near position 'F1' or 'F2' included lateral (Y) and vertical (Z) measurement directions, and both were used as reference channels. The combined deterministic-stochastic subspace identification method is designed especially for dealing with vibration data from mixed forced and ambient test. Presence of ambient vibrations is normally not desirable during a forced test, but it cannot be avoided. In forced tests of larger bridges, ambient vibrations are a significant part of total vibrations. The combined deterministic-stochastic subspace method can use both forced and ambient part of the vibrations in the identification.

The bridge was opened to traffic during the test. One lane was closed for the reaction mass exciter, while the other lane was left open to traffic. The traffic produced certain amount of non-measured (ambient) excitation. Total energy of the ambient excitation was low compared to forced input, because traffic intensity on this bridge is low.

### *5.1.6 Data Analysis*

Data from the two excitation positions were evaluated separately. Hence, two sets of frequencies, damping factors and mode shapes were obtained. The two obtained sets of modal parameters showed slight differences, as it can be seen from Table 3. The reason for differences between the two modal parameter sets is, besides accuracy of the identification, also small eigenfrequency changes of the bridge caused by changing bridge temperature. It can be observed that the second test delivers consistently lower frequencies than the first test. The mean temperature measured inside the bridge was 19.76°C during the first test, while during the second test it was 22.45°C. Due to temperature difference of 2.76°C, eigenfrequencies of the second test (excitation at 'F2') are expected to be 0.3-0.7% lower. The relationship between eigenfrequencies and temperature on this bridge has been studied using data from a monitoring system (Ralbovsky et al. 2011).

The subspace identification was carried out up to a model order of 200. For better handling (and quicker calculation), measurement data was downsampled to 51.2 Hz. The downsampled data allowed identification of frequencies up to ca. 22 Hz. Fig. 9 shows an example of a stabilization plot of one measurement setup.

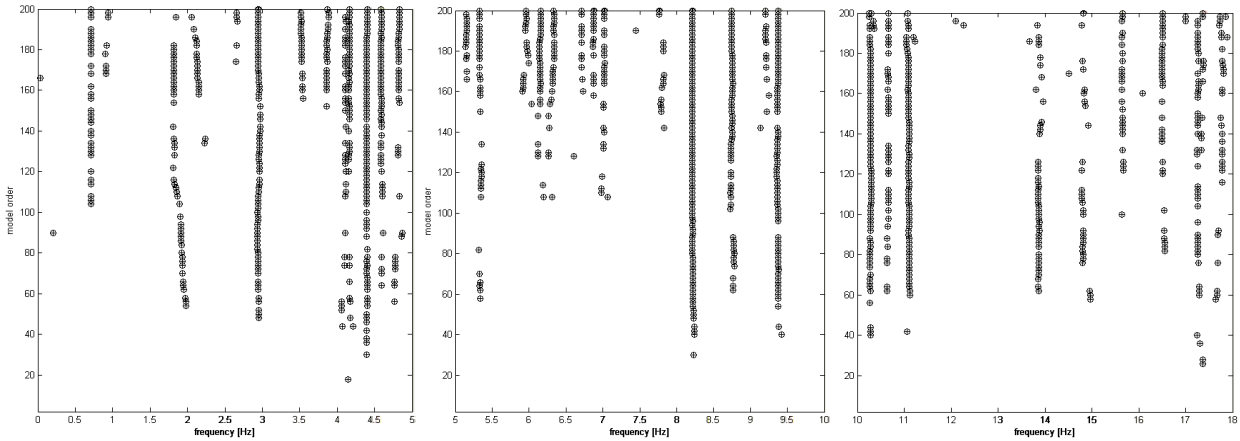


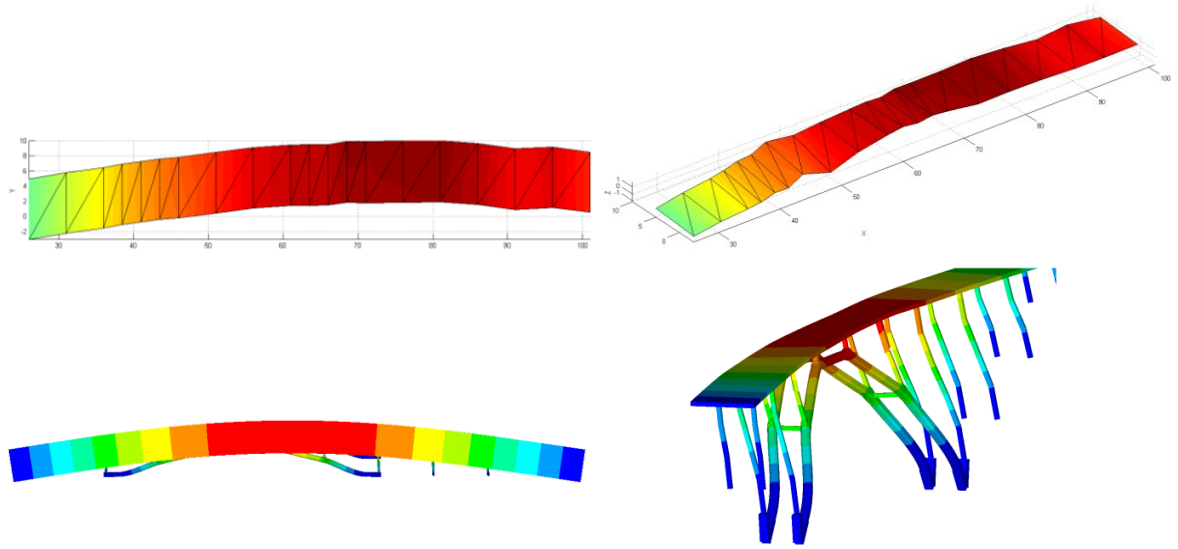

Fig. 9. Stabilization plot example

Table 3. Summary of identified eigenfrequencies

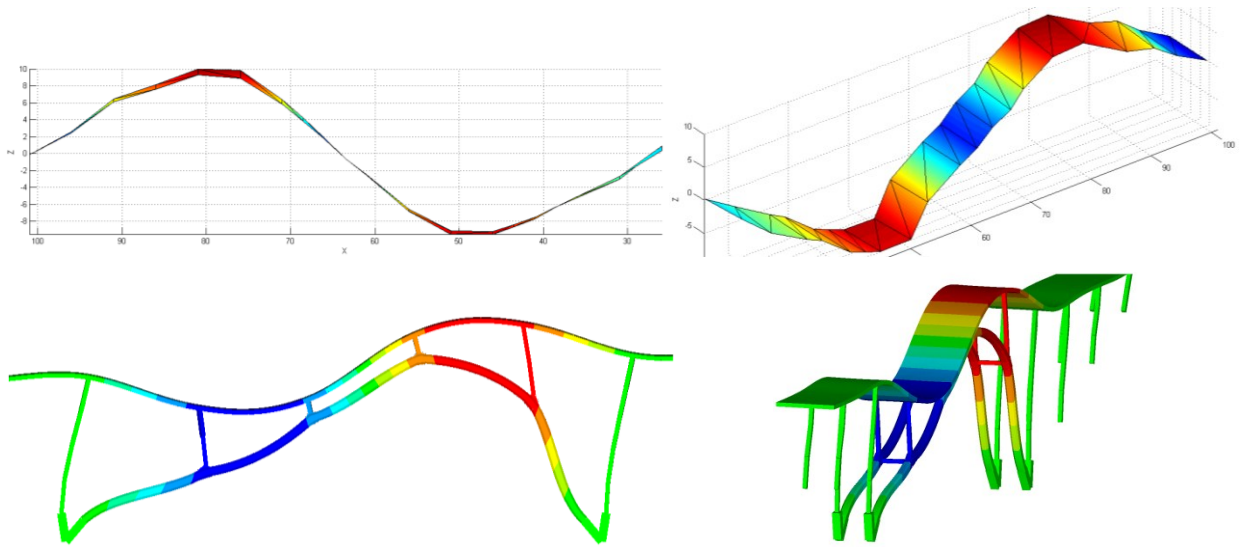
| Mode | Excitation at F1 | | Excitation at F2 | | FE-Model | Description, number of waves in the shape between points 1 and 20 |
|---|---|---|---|---|---|---|
| | Freq. [Hz] | Damp. [%] | Freq. [Hz] | Damp. [%] | Freq. [Hz] | |
| 1 | 0.71 | 2.3 | 0.72 | 2.0 | 0.74 | Lateral, ca. 0.5 wave |
| 2 | 2.05 | 6.6 | 1.99 | 6.5 | 1.14 | Vertical, 1 wave |
| 3 | — | — | — | — | 2.38 | Lateral, 1 wave |
| 4 | — | — | 2.87 | 1.9 | 2.79 | Vertical, 1.5 waves |
| 5 | — | — | — | — | 3.21 | Vertical |
| 6 | 4.14 | 1.8 | 4.11 | 1.3 | 3.93 | Vertical, 2 waves |
| 7 | — | — | — | — | 4.19 | Torsional, 1 wave |
| 8 | 4.40 | 0.6 | 4.37 | 1.4 | 4.61 | Torsional, 1 wave |
| 9 | 5.36 | 1.8 | 5.29 | 1.0 | 5.60 | Torsional, 1 wave |
| 10 | — | — | 6.22 | 2.2 | 5.36 | Vertical, 2.5 waves |
| 11 | — | — | 6.99 | 0.8 | 7.95 | Torsional, 1.5 waves |
| 12 | 8.20 | 1.2 | 8.15 | 0.9 | 9.54 | Torsional, 2 waves |
| 13 | 8.73 | 1.7 | 8.66 | 1.5 | — | Vertical, 3.5 waves |
| 14 | 9.37 | 1.8 | 9.28 | 1.7 | 9.04 | Vertical, 3.5 waves |
| 15 | 10.27 | 1.3 | 10.20 | 1.0 | 12.45 | Torsional, 2.5 waves |
| 16 | 11.09 | 2.2 | 10.96 | 1.9 | 11.14 | Vertical, 4 waves |
| 17 | 14.83 | 1.0 | 14.77 | 0.9 | — | Torsional |
| 18 | 15.66 | 0.5 | 15.61 | 0.5 | — | Torsional |

Results of the subspace identification were compared to computational modal analysis of a shell finite-element model. The FE-model has not (yet) been updated. The frequencies are compared in the table above. Assignment between measured and calculated modes has been done primarily using the mode shape. For each measured mode shape, a calculated mode shape was sought that matched best. Three calculated mode shape could not be identified in the measurements (2$^{nd}$ lateral, one vertical and 1$^{st}$ torsional), while three measured mode shapes (8.7 Hz, 14.8 Hz, 15.6 Hz) could not be matched to FE-calculations. Matching is more difficult in higher mode shapes because of higher shape complexity and lower modal identification accuracy.

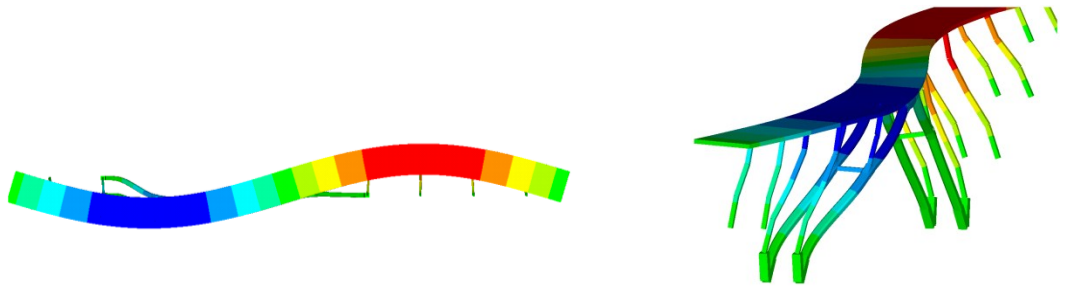
Fig. 10 shows a comparison between measured and calculated mode shapes. Measured mode shapes display only the deck, while calculated show the whole bridge. Shapes are displayed from two view angles. The first is either horizontal or vertical, the second is axonometric. Below the picture is description of the mode, mean of frequency and damping from the two identified modal parameter sets, and the calculated frequency.



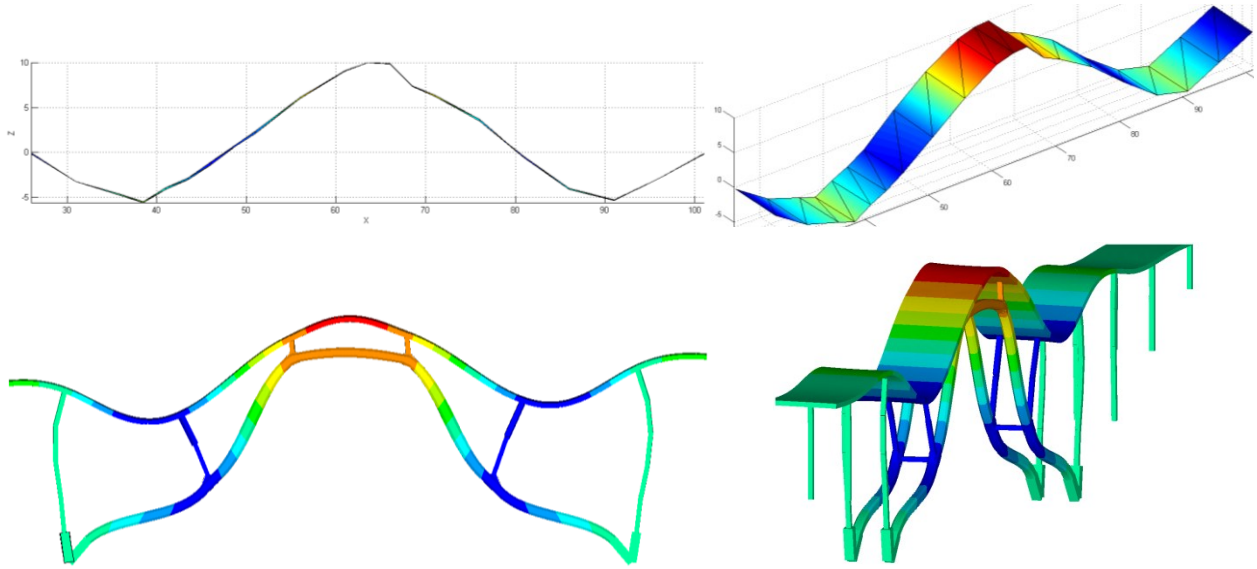
Mode 1, lateral, 0.5 wave. Measurement: 0.72 Hz, ξ=2.1%; FE-model: 0.74 Hz.



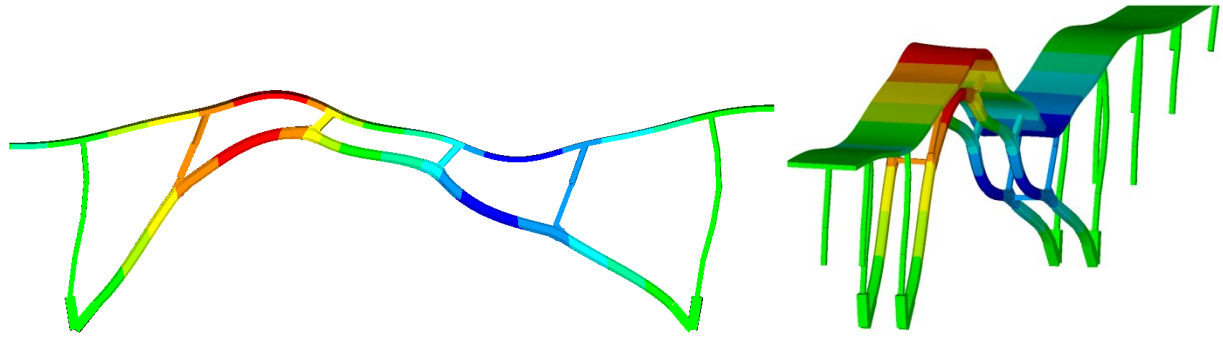
Mode 2, vertical, 1 wave. Measurement: 2.02 Hz, ξ=6.5%; FE-model: 1.14 Hz



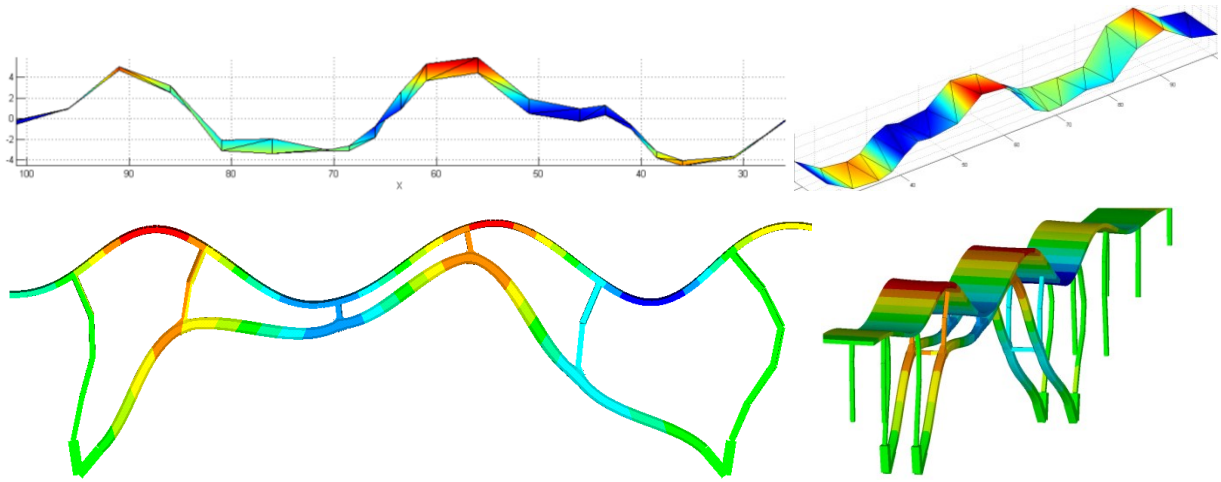
Mode 3, lateral, 1 wave. Measurement: - ; FE-model: 2.38 Hz



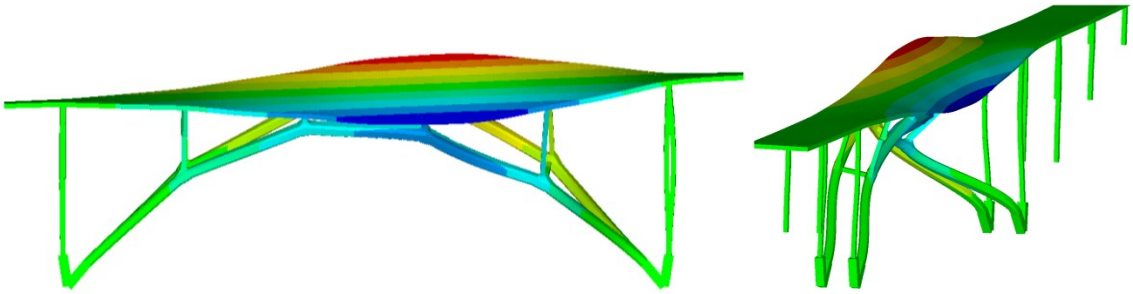
Mode 4, vertical, 1.5 wave. Measurement: 2.87 Hz, ξ=1.9%; FE-model: 2.79 Hz



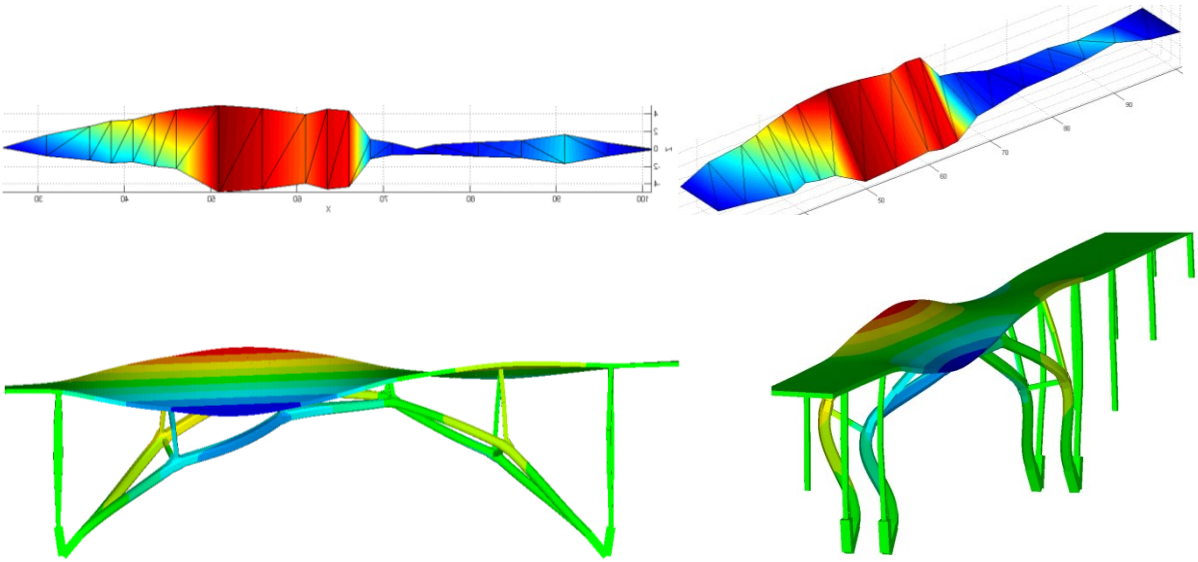
Mode 5, vertical. Measurement: - ; FE-model: 3.21 Hz



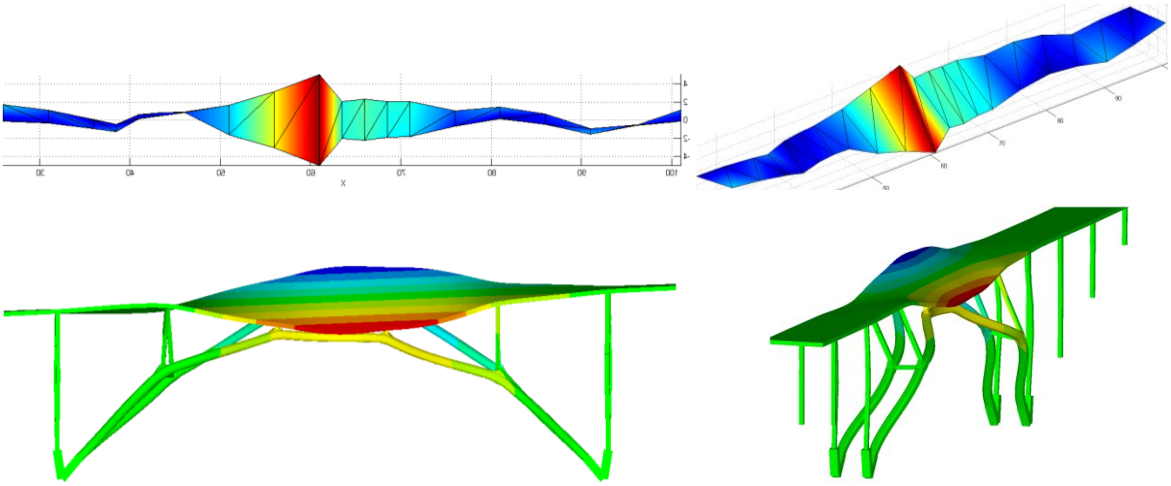
Mode 6, vertical, 2 waves. Measurement: 4.13 Hz, ξ=1.6%; FE-model: 3.93 Hz



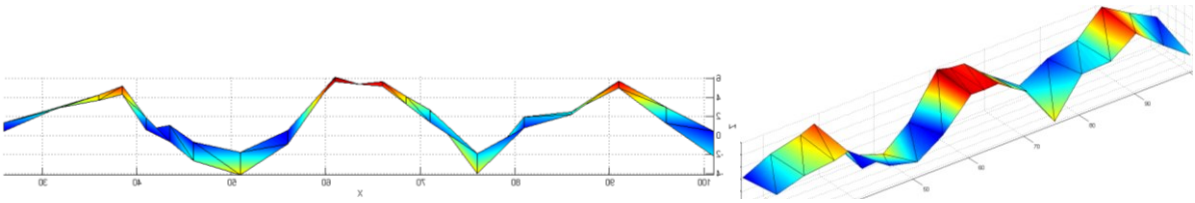
Mode 7, torsional, 1 wave. Measurement: - ; FE-model: 4.19 Hz

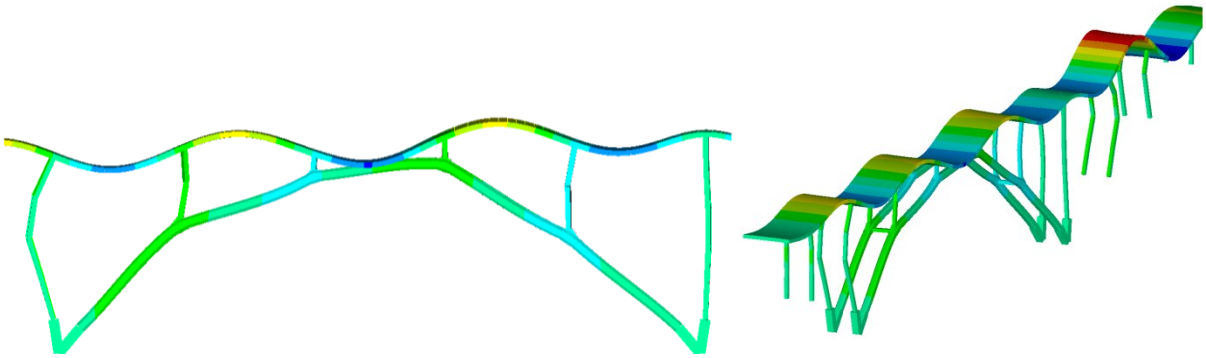


Mode 8, torsional, 1 wave. Measurement: 4.40 Hz, ξ=0.6%; FE-model: 4.61 Hz

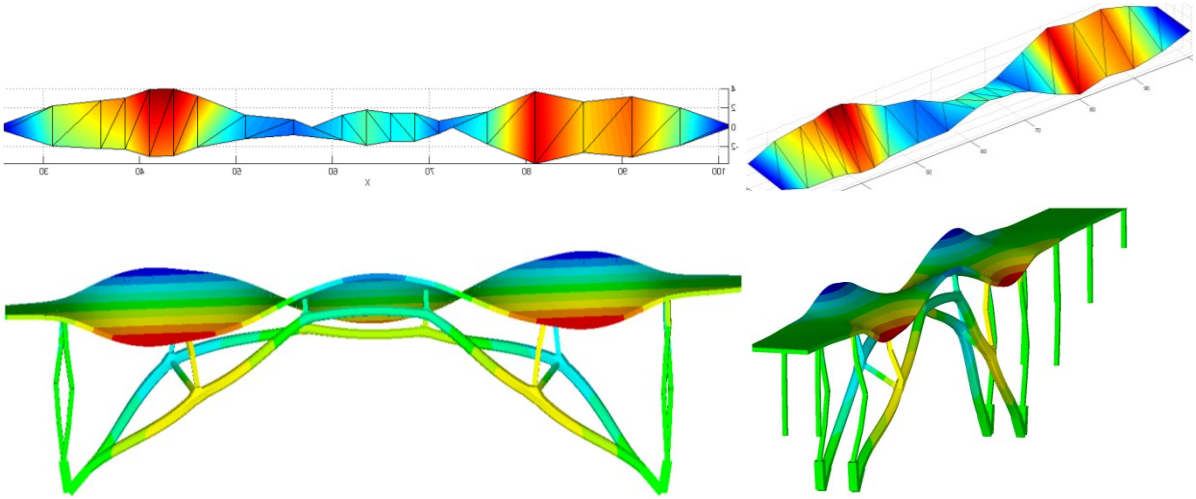


Mode 9, torsional, 1 wave. Measurement: 5.32 Hz, ξ=1.4%; FE-model: 5.61 Hz

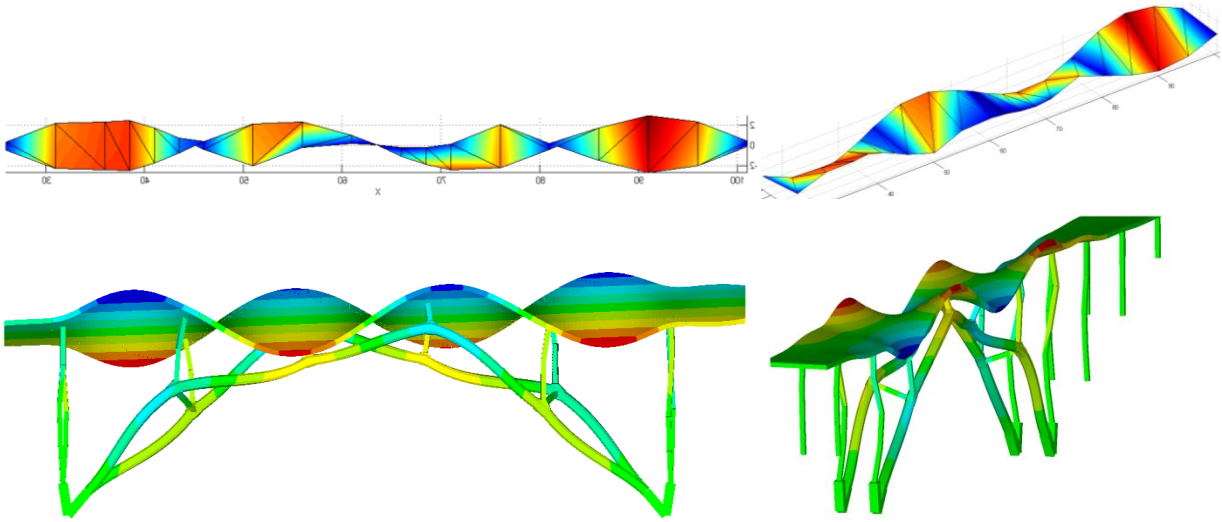




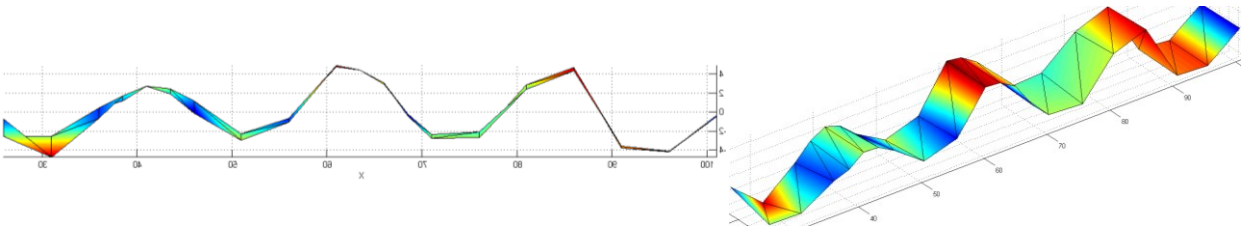
Mode 10, vertical, 2.5 waves. Measurement: 6.22 Hz, ξ=2.2%; FE-model: 5.36 Hz



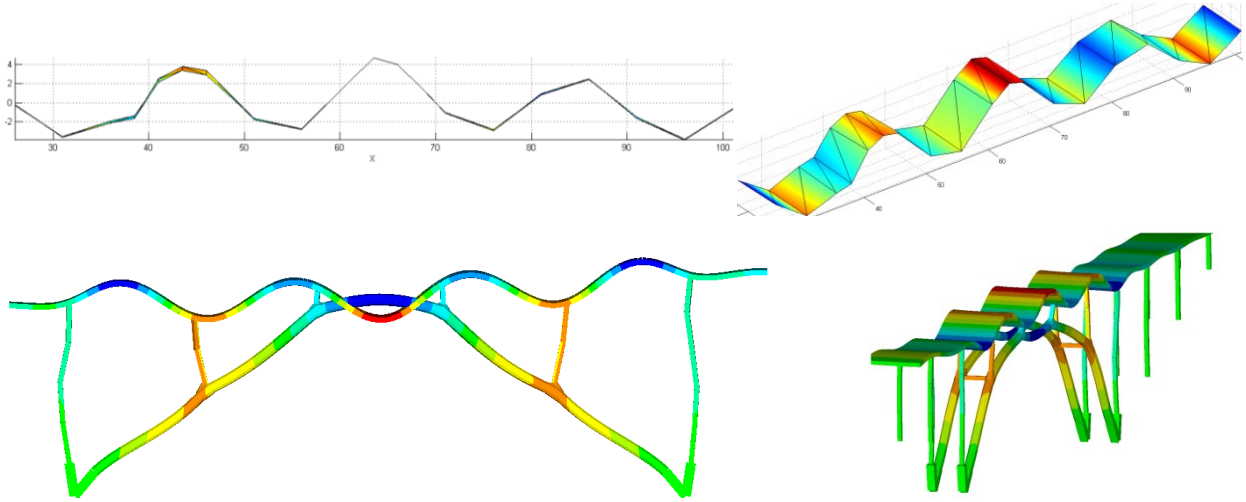
Mode 11, torsional, 1.5 waves. Measurement: 6.99 Hz, ξ=0.8%, FE-Model: 7.95 Hz



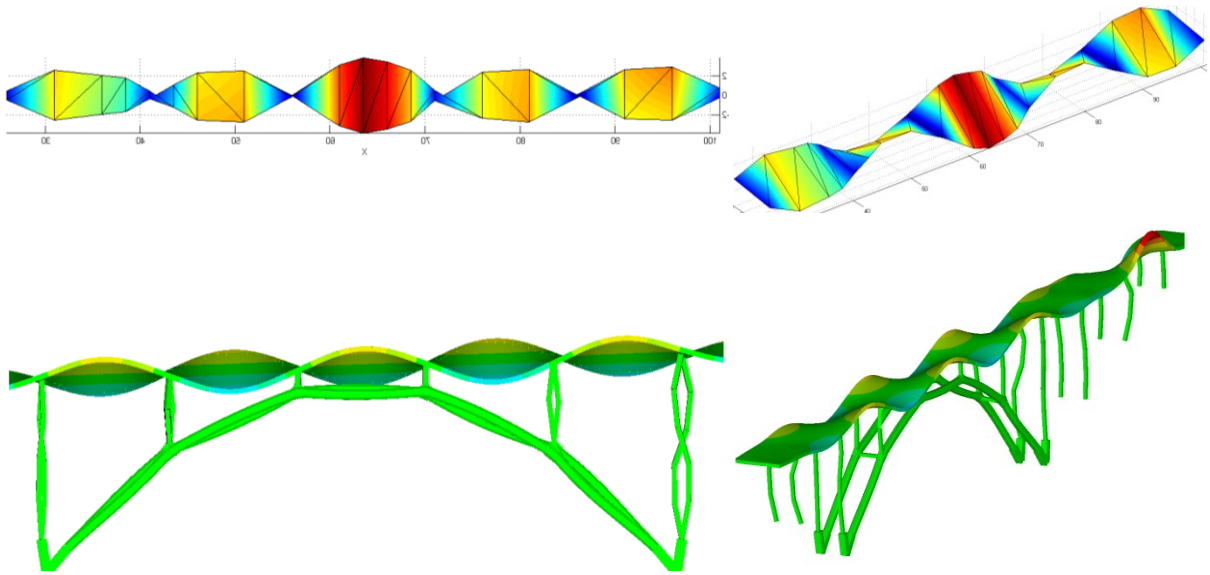
Mode 12, torsional, 2 waves. Measurement: 8.17 Hz, ξ=1.0%; FE-Model: 9.54 Hz



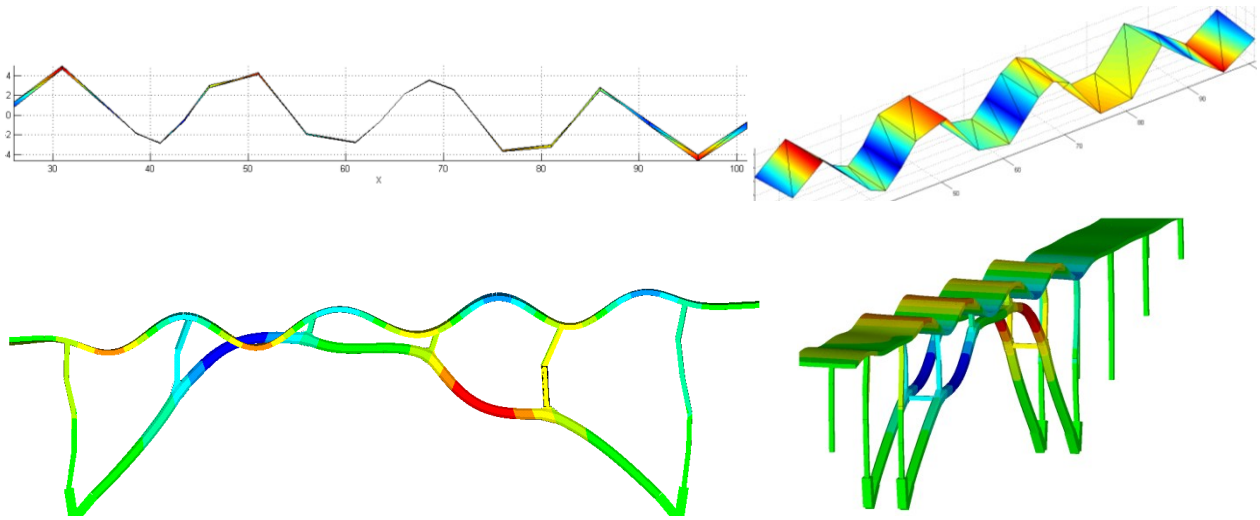
Mode 13, vertical, 3.5 waves. Measurement: 8.7 Hz, ξ=1.0%; FE-Model: -



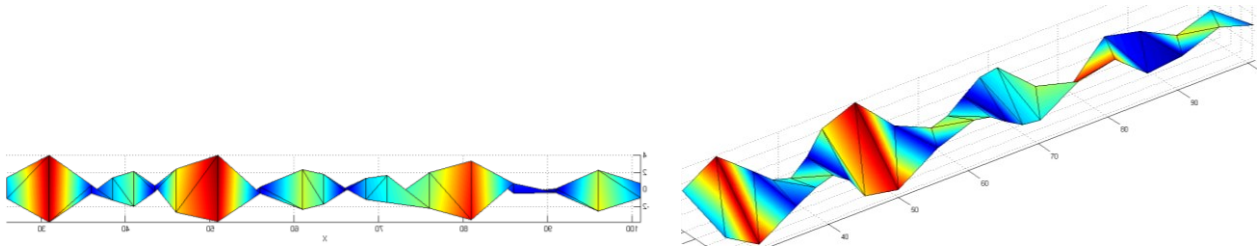
Mode 14, vertical, 3.5 waves. Measurement: 9.32 Hz, ξ=1.7%; FE-Model: 9.04 Hz



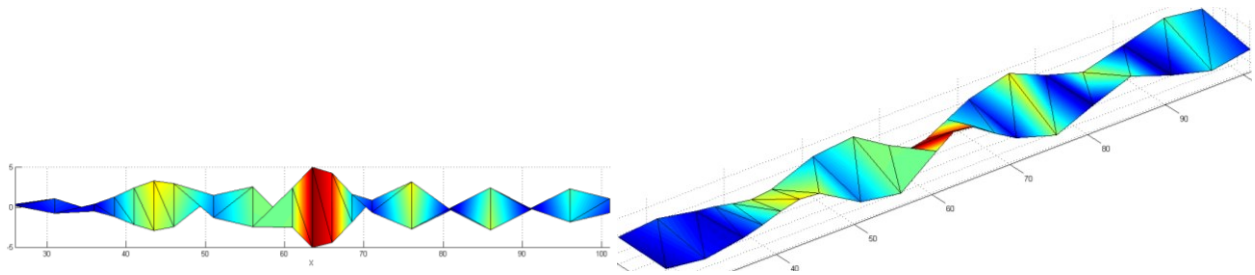
Mode 15, torsional, 2.5 waves. Measurement: 10.23 Hz, ξ=1.1%; FE-Model: 12.45 Hz



Mode 16, vertical, 4 waves. Measurement: 11.03 Hz, ξ=2.0%; FE-Model: 11.14 Hz



Mode 17, torsional. Measurement: 14.80 Hz, ξ=1.0%; FE-Model: -

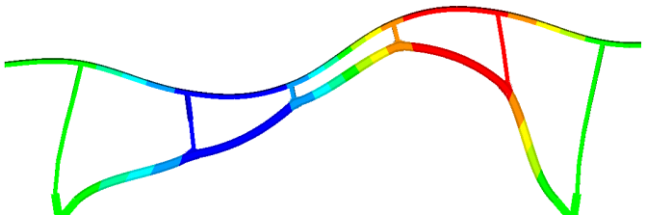


Mode 18, torsional. Measurement: 15.63 Hz, ξ=0.5%; FE-Model: -

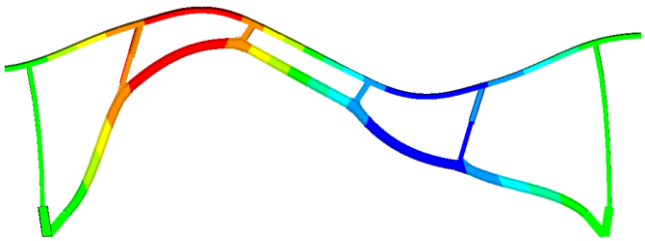
Fig. 10. Measured and calculated mode shapes

Measured and calculated modal parameters are overall in relatively good coincidence. However, there are some differences that may be used to update the original FE-model. The most significant discrepancy is in the frequency of the first vertical bending mode. The measured frequency (2.02 Hz) was almost double of what was predicted (1.14 Hz). The original FE-model is not constrained in longitudinal direction at dilatation joints of the deck or at bearings of abutment. The FE-model can be improved by adding springs in longitudinal (X) bridge direction at both ends of the deck. If the spring stiffness at the deck ends is 40 MN/m per unit length of dilatation, the frequency of the first vertical mode rises to 2.03 Hz, while the mode shape remains almost the same (Fig. 11).

Original FE-Model: 1.14 Hz



Updated FE-Model: 2.03 Hz



Measurement: 2.02 Hz

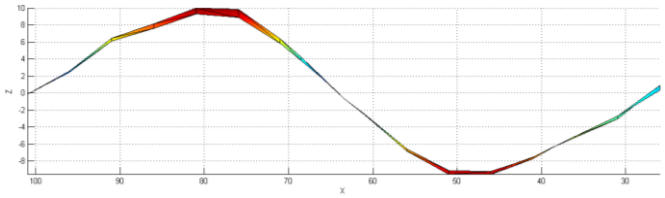


Fig. 11. First vertical mode of original and updated FE-model

This example just shows the principle and potential of model updating. Model updating has not been completed yet; and it will require considering all measured modes and several other FE-model parameters.

The improved (updated) FE-model can be used for different calculations of the bridge operating in comparable load levels to those during testing. The updated model boundary conditions may no longer apply at significantly different load levels, e.g. load levels at the Ultimate Limit State, unless capacity of the updated boundary conditions is thoroughly investigated.

### *5.1.7 Benefits of using field monitoring*

Forced vibration test has been used to acquire real dynamic properties of the UHPFRC arch bridge Wild. Fifteen mode shapes in range 0-16 Hz were identified and compared to computational model of the bridge. The measurement results will be used to improve the model that can then be used for static and dynamic calculations of the bridge under normal traffic conditions. Ultimately, after completion of further investigations, the updated model should be used to correct present safety assessment calculations and to aid in life-cycle management of this structure.

## *5.2 Industrial building – reinforced concrete structure AIT*

Vibration test of a RC industrial building is presented. Forced vibration test was carried out as primary test and ambient vibration test as complementary experiment.

### *5.2.1 Quick facts*

| | |
|---|---|
| Building type: | Reinforced concrete cylinder |
| Ultimate objective: | Earthquake assessment |
| Number of measurement points: | 33 |
| Number of excitation points: | 1 |
| Sensors / frequency range: | geophones / 1-315 Hz |
| Exciter / frequency range: | hydraulic / 0-80 Hz |
| Used excitation types: | sine-sweep, broadband random, ambient |
| System identification method: | frequency domain polyreference method |
| Reference: | Friedl et al. 2007, 2009 |

### *5.2.2 Building description*

The building is a reinforced concrete cylinder (Fig. 13) with a diameter of 26 m and height of 22 m (Fig. 12). The cylinder is 25 cm thick. Cylinder roof is a dome with spherical shape.

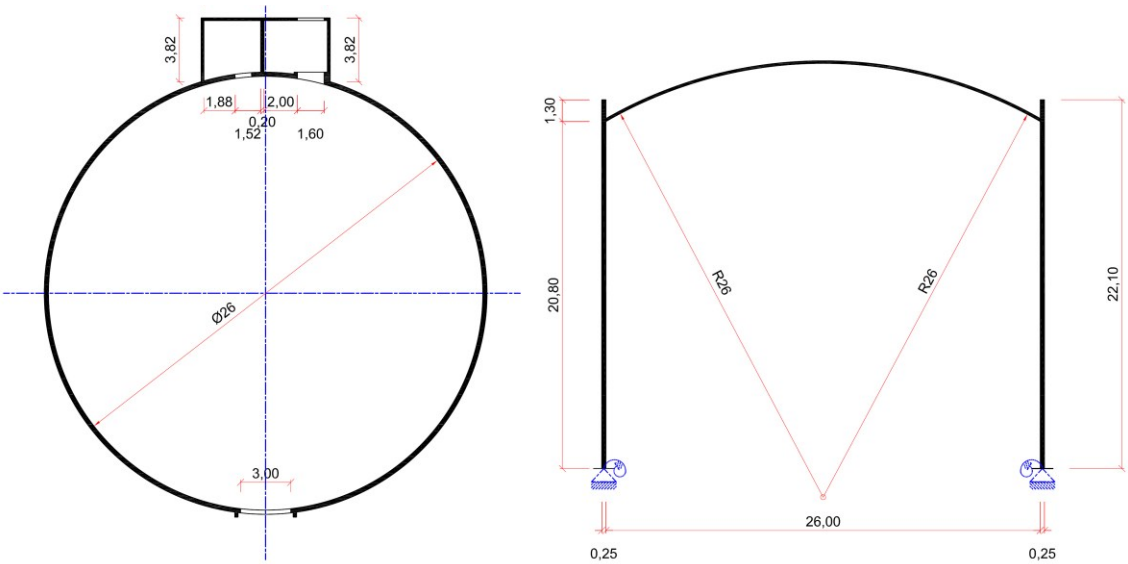



Fig. 12. Building geometry

Fig. 13. View of the building

### *5.2.3 Objective*

Aim of vibration testing was to acquire real dynamic structural properties for more accurate earthquake assessment. Modal properties identified from measurements should be used to validate or update a computational model of the building that would be used for earthquake assessment.

### *5.2.4 Methodologies of measurement*

The experiment was carried out as forced vibration test. The reasons for choosing forced over ambient vibration excitation were:

a) High excitation levels, especially in higher frequencies
b) Low amplitude of ambient vibration at this specific site
c) High excitation control

Planning of experiment was done in three steps:

1. Preliminary ambient vibration measurement with one triaxial sensor
2. Calculation of expected mode shapes using preliminary Finite-Element model
3. Design of excitation and measurement point scheme; making of time schedule

The preliminary ambient vibration test was an easy and inexpensive way to assess potential of ambient vibration testing at this structure and also to get a first approximation of structural dynamic behaviour. Having this information makes experiment planning easier regarding proper choice of measurement method and equipment to be used. The preliminary test took ca. 30 minutes and was done using one high-sensitivity (10 V/g) triaxial accelerometer with frequency range 0.05 – 500 Hz. From data acquired with this accelerometer it can be stated whether such high sensitivity accelerometers with frequency ranges below 1 Hz are needed for the final setup, or cheaper and smaller sensors would be sufficient.

The preliminary ambient vibration showed very low ambient vibration levels. The location of the building is a rural area with very few ambient vibration sources. Recorded excitation from the wind was very low. It was concluded, that extraction of modal properties from ambient vibration data would be difficult even using the high-sensitivity sensors. Therefore, forced vibration test was planned.

Mode shapes calculated with preliminary Finite-Element model showed 4 to 6 waves around the cylinder circumference and approximately half wave across the cylinder height (see Fig. 17).

*Measurement point grid*

To capture the modes in sufficient resolution, 15° circumference-spacing of measurement points was proposed. Sufficient for the purpose of this test it was considered to capture at least 4 points per major mode shape wave. Across the cylinder height, 4 measurement point levels were defined (Fig. 14). To reduce the number of measurement points and thus the expense of the test, the 15° circumference-spacing was used only at one level (middle of cylinder height) of the setup, and in that level at half of the circumference only (Fig. 15). The rest of the setup used 60° spacing. Some of the non-measured points of the original 15° spacing setup can be evaluated as "slave points" in visualization of identified mode shapes.

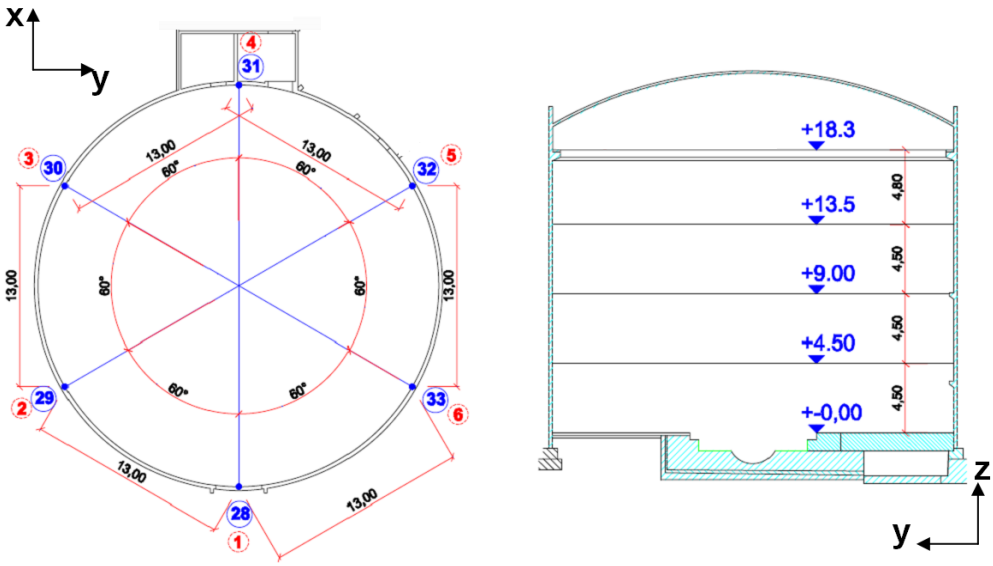



Fig. 14. Measurement point grid at levels +4.5, +13.5, +18.3

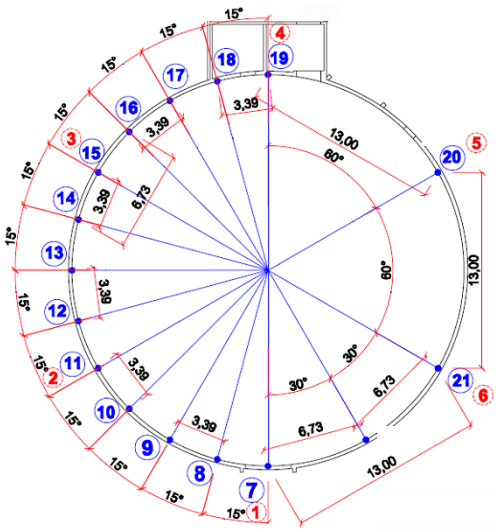



Fig. 15. Measurement point grid at level +9.0

*Excitation*

Ideal height of the excitation point would be in the middle of the cylinder, which was technically not possible in this case. The excitation point was fixed at height of 7.5 m.

Two types of excitation were introduced: sine sweep with 8-40 Hz range and broadband random excitation with 2-40 Hz range. Amplitude of dynamic force excitation used was approximately 1.5 kN.

*Equipment*

Sensors: 5 triaxial velocity transducers (geophones), frequency range 1-315 Hz, native sensitivity 25 V/m*s

Acquisition system: A/D conversion at 24bit, 1000 Hz sampling rate

Excitation: Hydraulic exciter, max. force 25 kN, controlled by target force signal as input

Sensors were mounted using screws and dowels (one per sensor). The hydraulic exciter was connected to the structure using a rod chain (Fig. 16) that was attached to a mounting plate fixed at the excitation point. The rod chain has an embedded force sensor. The hydraulic exciter is capable of using any input signal as target force or target piston displacement. Therefore any type of excitation (sine-sweep, random, other specific time-history signal) is possible using this device.

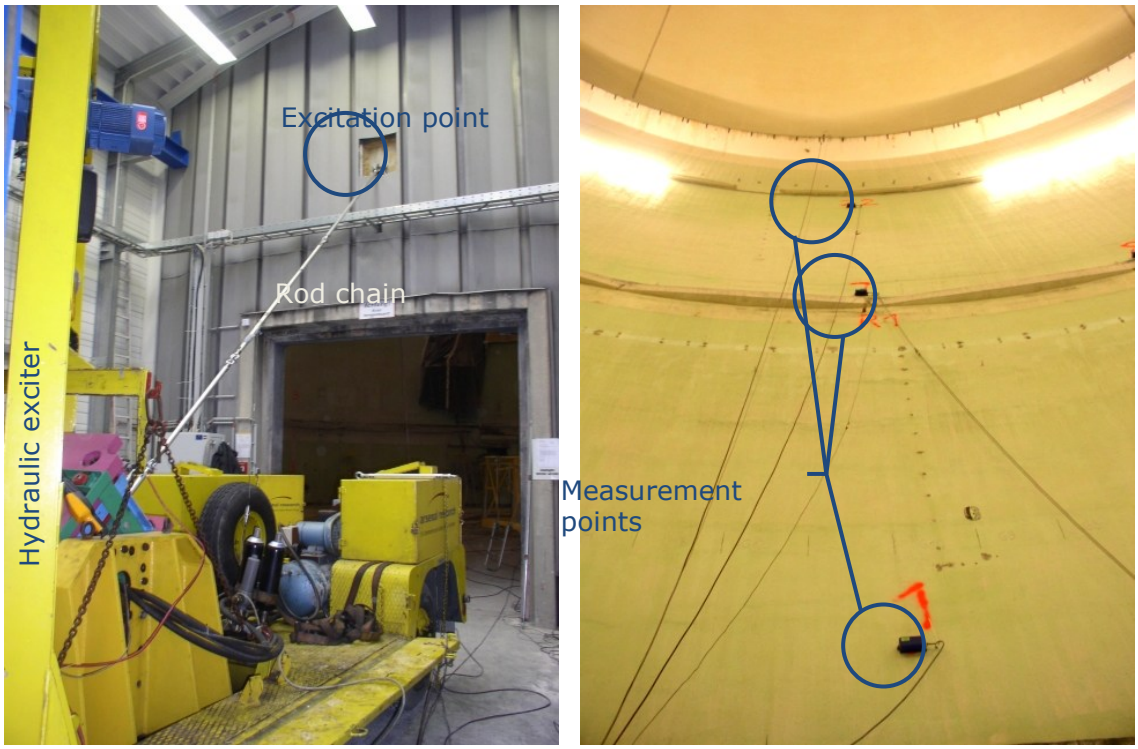



Fig. 16. Excitation via rod chain (left) and mounted sensors (right)

### *5.2.5 System Identification Methods*

Identification of modal properties from measured data was carried out using a commercial software package. The implemented algorithm is a frequency-domain polyreference method. Poles were calculated for a model order up to 25. Stable poles were selected from the stabilization plot in model orders 22-24.

**OROS Modal 2 (OM2)**

| | |
|---|---|
| Geometry | Geometry builder - import in UFF and IGES |
| Data import/export | UFF and Excel compatibility |
| Impact hammer acquisition | Sequencer - FRF H1/H2, coherence- force/response window - double impact rejection - manual accept/reject |
| Shakers acquisition | Multi excitation - sine/random/chirp excitation - hanning window |
| Modal Indicator Function | Based on Singular Value Decomposition - available in ODS, EMA and OMA modules |
| Stability diagram | Automatic detection of structural modes |
| ODS | In time and frequency domain |
| EMA SIMO method | Based on Rational Fraction Polynomial formulation of transfer function |
| EMA MIMO 1 method | Based on Frequency Domain Poly-Reference algorithm (FDPR) |
| EMA/OMA broadband method | Based on Polyreference Least Squares Complex Frequency algorithm (p-LSCF) |
| Validation | Modal Assurance Criterion |

### *5.2.6 Data Analysis*

Frequency response functions (FRFs) were calculated for both excitation types used (Fig. 17, Fig. 18). The FRFs showed very small differences. Data from random excitation was then used for modal parameter identification, since from theory is known that FRFs from sine-sweep contain some error as a result of nonzero sweep speed.

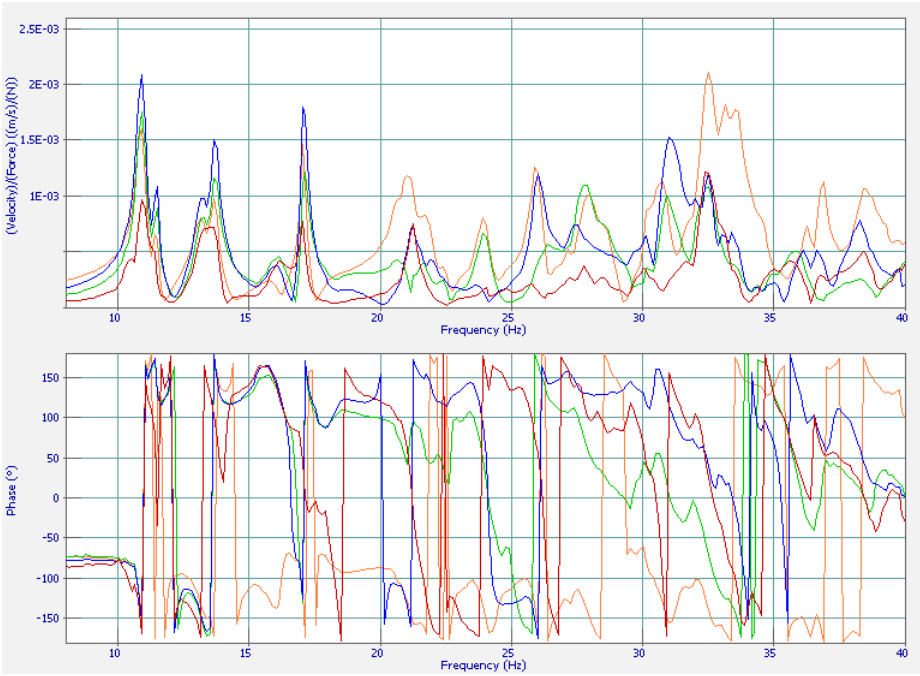



Fig. 17. Four frequency response functions from sine-sweep excitation

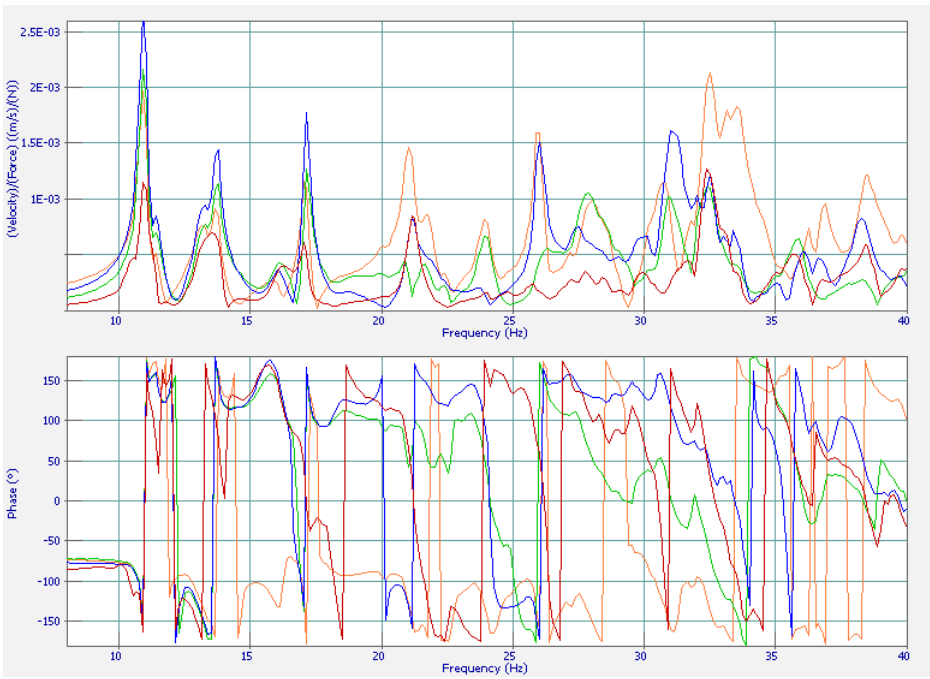



Fig. 18. Four frequency response functions from random excitation

Six modes in frequency range up to 20 Hz were selected from the stability plot (Fig. 19).

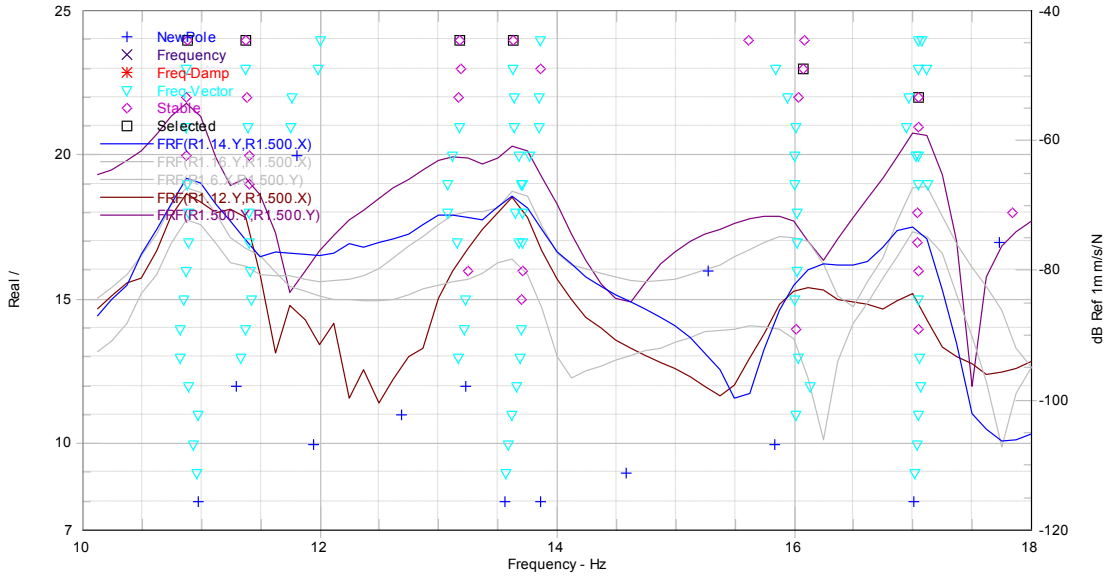


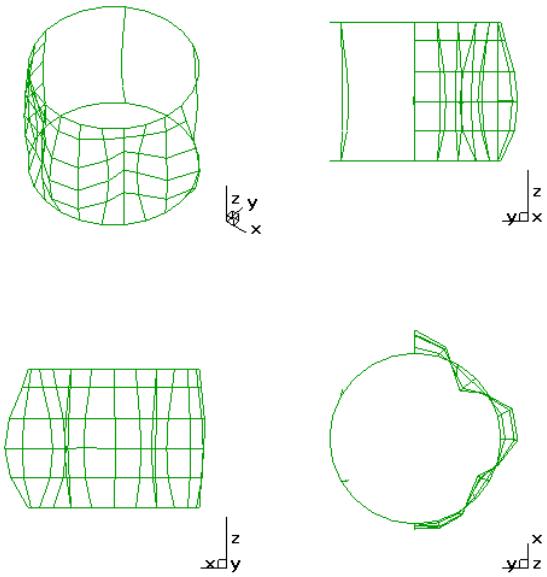

Fig. 19. Stability plot

Resulting frequencies and damping factors are listed in Table 4 and the identified mode shapes are shown in Fig. 20.

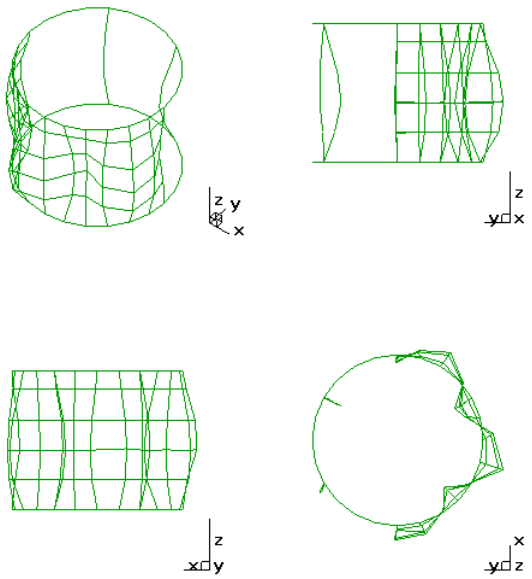
Table 4. Identified frequencies and damping factors

| Mode | Frequency [Hz] | Damping [%] |
|---|---|---|
| 1 | 10.88 | 1.22 |
| 2 | 11.41 | 1.09 |
| 3 | 13.18 | 2.47 |
| 4 | 13.64 | 1.01 |
| 5 | 16.04 | 2.00 |
| 6 | 17.05 | 0.65 |

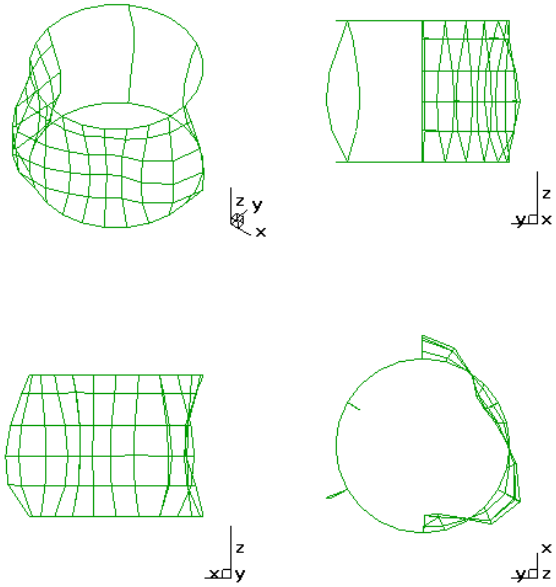
Name: Mode 1 Freq.: 10,88138 Damping: 0,0122233



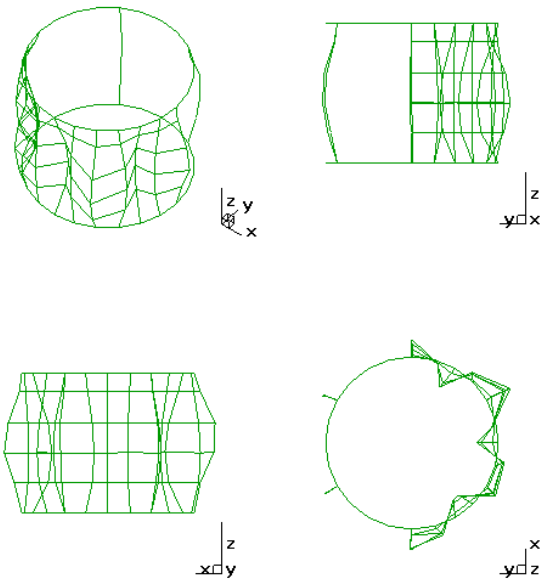
Name: Mode 2 Freq.: 11,40774 Damping: 0,0109068



Name: Mode 3 Freq.: 13,18437 Damping: 2,466392E-02



Name: Mode 4 Freq.: 13,63602 Damping: 1,007855E-02



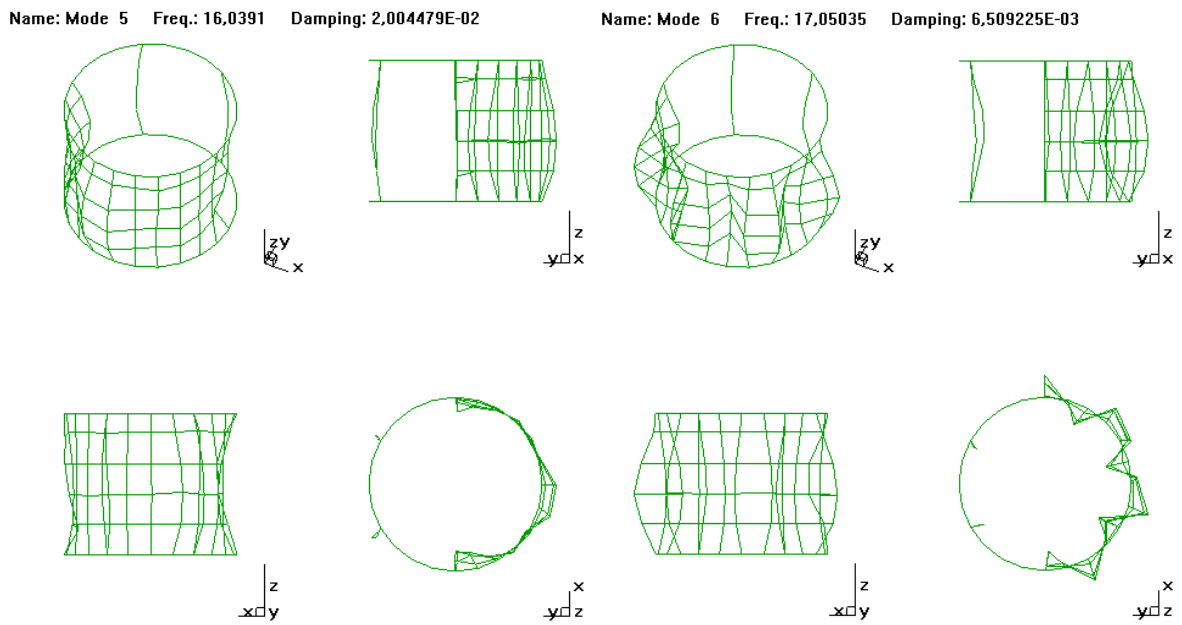
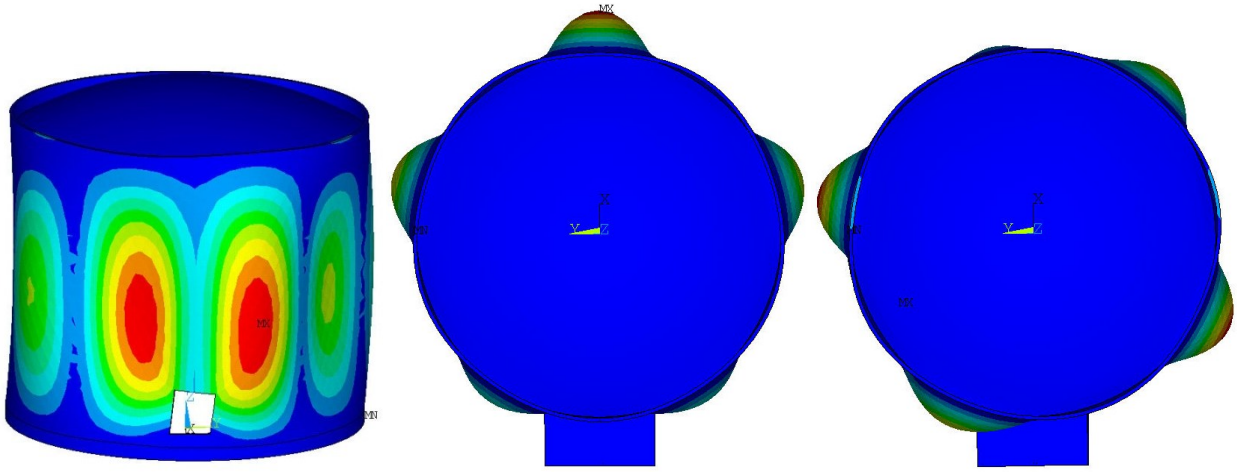



Fig. 20. Identified mode shapes

The first two measured modes were used to update the computational model. The updated model parameters were the Young's modulus of concrete and rotational stiffness of the foundation (Friedl et al. 2009). The Young's modulus of concrete was corrected from 28.8 GPa to 38.7 GPa. The modal parameters of the updated model are displayed in Fig. 21.



| | | |
|---|---|---|
| $f_{calc}$ = 10.10 Hz | $f_{calc}$ = 10.65 Hz | $f_{calc}$ = 11.43 Hz |
| $f_{meas}$ = — Hz | $f_{meas}$ = 10.88 Hz | $f_{meas}$ = 11.41 Hz |

Fig. 21. Calculated mode shapes

### *5.2.7 Benefits of using field monitoring*

Results from experimental modal analysis were used to update and verify present computational model of the structure.

## *5.3 SDOF system – Soil structure effect*

### *5.3.1 Construction*

In the framework of the European project SERIES (Grant agreement 227887,) a new prototype structure (Fig. 22) was built in the EUROSEISTEST site in order to study salient features of the SFSI mechanism. The model structure comprises of three concrete plates, representing the foundation (one plate) and structural mass (two removable plates), connected with four steel columns and braces between the columns on all four sides forming a totally symmetric structure. The prototype structure is founded on the free surface of the soil, with a minimal embedding of a few centimeters to ensure full and optimum contact of the foundation with the subsoil.

Fig. 22. Prototype structure in Euroseistest constructed in the framework of SERIES

More specific details on the structure are given below:

- Foundation plate with dimensions 3x3x0.4m, made of reinforced concrete C25/30. The foundation plate has a cylindrical hole in its geometrical center, for the monitoring devices to be inserted in the 30m deep borehole below. On the upper part of the foundation plate there is a bolt configuration ready to attach the eccentric mass vibrator for the forced vibration tests to be performed.
- Four steel columns of cross-section type QHS150x8 and free height 3.8m. The four columns are bolted on the foundation plate.
- Vertical braced frames, each brace of section L100x9 are bolted on the steel columns in both directions.
- Two reinforced concrete slabs of dimensions 3x3x0.4m each, representing the structural mass, are placed on top of the steel columns. The bottom one is bolted on the columns and the upper one is attached to the bottom one with a vertical bolt. On the upper part of the top slab there is a bolt configuration ready to attach the eccentric mass vibrator for the forced vibration tests to be performed.

- The structure is configurable and can attain a fixed-base resonant frequency of 5-15Hz based on the braced frame and mass configuration.

The structural design of the prototype structure conforms to the provisions of modern codes and regulations. It is designed to withstand pull-out and forced vibration, in order to better understand the wave propagation induced by the structure to the foundation soil and the SFSI effects.

Furthermore, considering the soil beneath the prototype structure, the subsurface stratigraphy at the site as well as the dynamic properties of soil formations are already very well known, derived from the past extended geotechnical and geophysical surveys. However, in order to define a detailed soil stratigraphy exactly below the prototype structure and to measure the specific soil properties under the framework of the present research program, a 30m deep borehole was drilled in the geometric center of the foundation plate. SPT tests were conducted in this hole and continuous specimens were taken according to EC7 regulations for undisturbed sampling. A second 15m deep borehole was drilled outside of the foundation slab perimeter in order to place vertically instruments for the measurement of the soil response due to the oscillation of the structure by the pull-out and the forced vibration tests.

Split-spoon and undisturbed samples retrieved for laboratory index testing, soil classification, strength and compressibility characteristics. Triaxial and resonant column tests will be performed at representative soil specimens. A complementary survey of array measurement of microtremors is also scheduled.

### *5.3.2 Numerical analyses*

Numerical simulation of the free and forced vibration tests were performed prior to any in-situ large scale experiment. The aim was to define numerically the resonant frequencies of the system and its response to given dynamic loading. Two finite element software programs were used, namely ANSYS and ABAQUS. The two software applications were validated against each other, as well as against theoretical solutions and regarding their accuracy (Fig. 23). For the validation and the numerical analyses, the soil was simplified to a two-layer homogeneous profile and the structure as a single degree of freedom structure. The fundamental frequency of the two-layer soil is at $f_{o,soil}$=2.9Hz. The frequency at which the ratio of the superstructure to free field horizontal displacement is maximized was computed to determine the effective natural frequency of the SSI system. The results revealed an effective frequency $f_{SSI}$ at 5.7Hz indicating a 50% decrease with respect to the fixed base case ($f_{str.fixed}$=8.7Hz) due to foundation compliance, thus quantifying soil-structure interaction in terms of fundamental dynamics considerations.

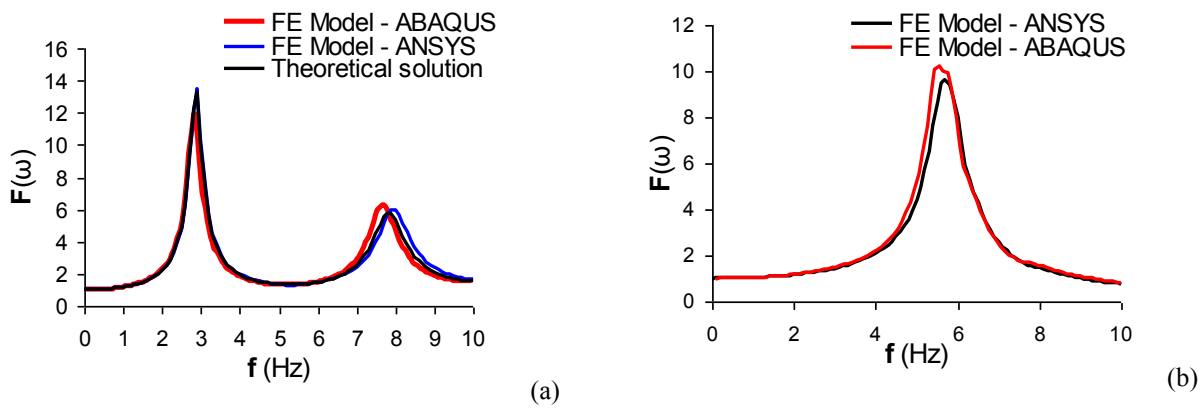



Fig. 23. Transfer function of the two-layer soil (b) and Free field-to-superstructure transfer function (a)

### *5.3.3 Numerical simulation of pullout tests*

In order to simulate pull-out loading, a constant force was applied at the superstructure mass for a small time interval and was then removed allowing free vibration of the structure. The effect of loading force was examined though different values of loading amplitude ($F_o$) (i.e. 2, 5, 10, 20, 30, 40 and 50kN). It is noted that 50kN corresponds to the maximum horizontal force that can be produced by the eccentric mass vibrator mounted at the top of the roof slab, representing an extreme loading scenario.

Pull-out loading was examined for flexible base conditions utilizing the coupled soil-structure FE models mentioned above. Comparison of the fixed- and flexible-base response, as computed from ANSYS FE model, is shown in Fig. 24(a) and (b), referring to the horizontal displacement of the structure under 10kN and 50kN amplitude of the imposed force. It is observed that soil compliance increases substantially the absolute displacement of the structure with respect to the fixed-base case. Furthermore, natural period elongation and damping increase due to soil-structure interaction is clearly demonstrated in time domain as well. Similar results came out from ABAQUS FE model, as is shown in Fig. 25(a) and (b), for the 50 kN loading scenario.

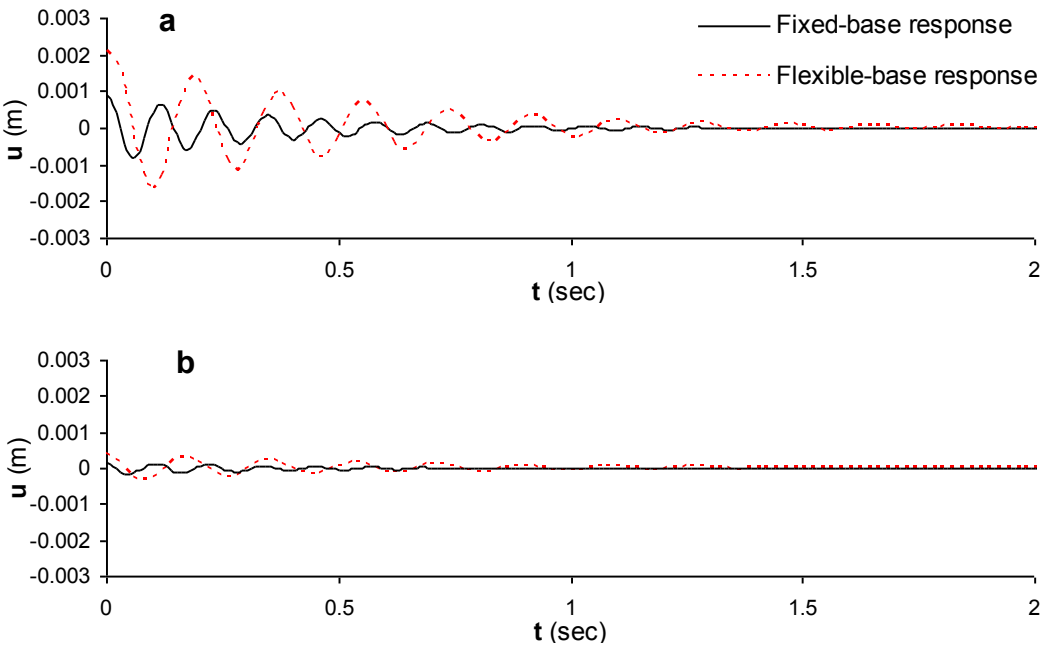



Fig. 24. Comparison of fixed base to flexible base response of the model structure under free vibration, as computed from ANSYS FE model for (a) 50kN force amplitude and (b) 10kN force amplitude

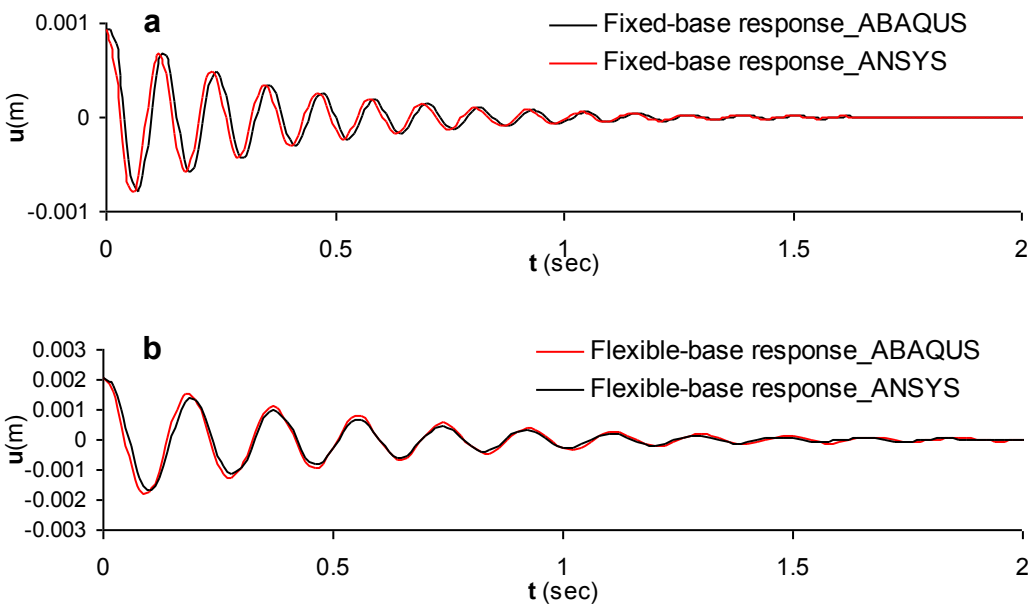



Fig. 25. Comparison of the (a) fixed base and (b) flexible base response of the model structure under free vibration, as computed from ANSYS and ABAQUS FE models for 50kN force amplitude

### *5.3.4 Numerical simulation of forced vibration tests*

Numerical analyses of the simplified system were performed applying harmonic horizontal force at the top of the structure. Of particular interest was to examine the dynamic response of the system when the loading frequency is equal to the fundamental frequency of the two-layer soil (i.e. 2.9Hz), the effective natural frequency of the SFSI system (i.e 5.7Hz) and the natural frequency of the structure fixed at its base (i.e. 8.7Hz) respectively, generating resonance effects. The latter are demonstrated in Fig. 26 where fixed and flexible base response, computed by ANSYS, are compared in terms of structural absolute displacements and accelerations. As expected, SFSI effects are strongly controlled by the loading frequency. When the excitation frequency is equal to $f_{str.fixed}$ (Fig. 26c) the well-known elastodynamic response of a fixed-base SDOF structure in resonance is observed. Conversely, the response of the flexible-base structure is significantly amplified at the effective natural frequency of the system (Fig. 26b). The fact that $f_{o,soil}$ is closer to $f_{SSI}$ than to $f_{str.fixed}$ explains the larger amplification pattern observed for the flexible-base system with respect to the fixed-base case (Fig. 26a).

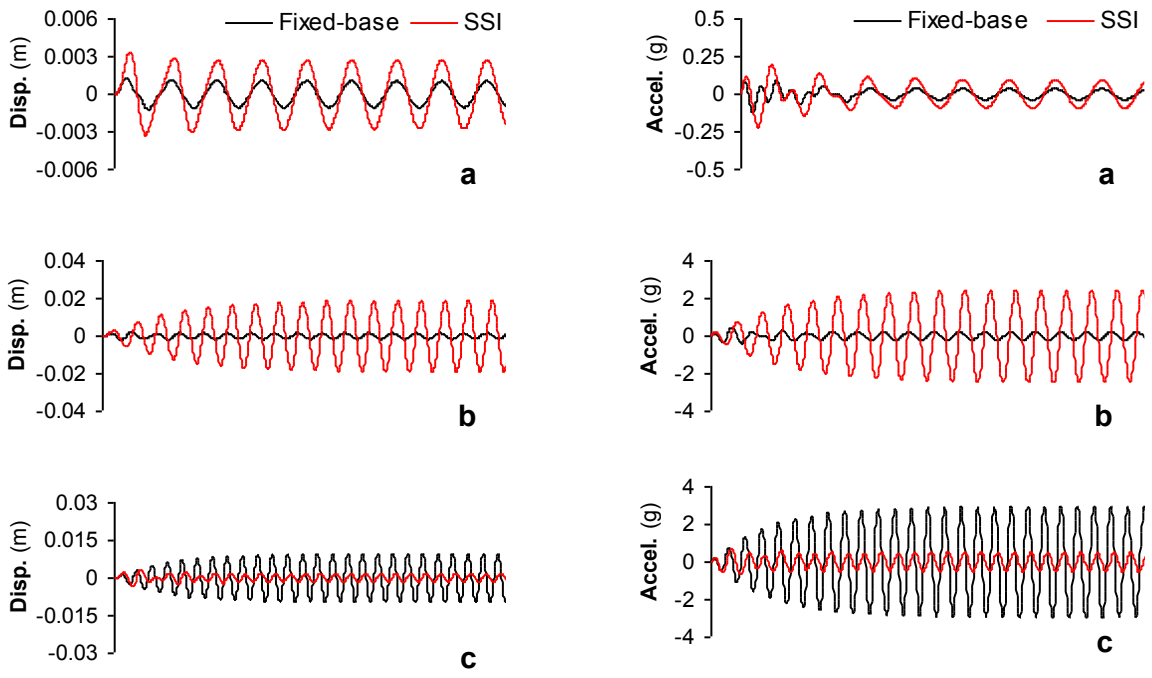



Fig. 26. Comparison of fixed-base to flexible-base response of the structure under forced vibration, as computed from ANSYS. Absolute displacements (left) and accelerations (right) generated for excitation frequency at: (a) 2.9Hz (=$f_{o,soil}$) (b) 5.7Hz (=$f_{SSI}$) and (c) 8.7Hz (=$f_{str.fixed}$)

## *5.4 SDOF System – Benchmarking of different test methods AUTH*

### *5.4.1 Large-scale experiments in Euroseis*

The prototype structure was designed to be excited by "Pull-out" and "Forced Vibration" tests. The latter are going to be performed by a shaker mounted on the roof slab, with different oscillation orientations in order to promote swaying and/or rocking. The level of the shaking – needed to promote the SFSI - is still to be determined from parametric numerical analyses performed by AUTH and the characteristics of the available shakers to be provided by ITSAK.

Pullout experiments were performed in Euroseis in February 2011 and July 2011. The soil and the structure were instrumented in order to measure the dynamic response of the system to the free vibration of the structure. In order to capture the 3D wave field emanating from the oscillation of the structure, the prototype structure and the surrounding ground were instrumented with an important number of classical and new sensors (i.e. SAA – ShapeAccelArrays, Measurand). The newly purchased SAA instrumentation was placed horizontally and vertically, underneath or in the vicinity of the foundation, in order to capture the 3D propagation of the emanating wave field from the vibrating foundation to the soil More specifically, apart from the structure, boreholes were drilled and trenches were excavated in the vicinity in order to instrument and monitor the pull-out (free vibration) and forced vibration experiments planned in the context of SERIES. The boreholes and trenches were designed to accommodate the two 12m-long SAA sensing devices as well as the temporary array of seismometers and accelerometers in Euroseis.

The planned 3D configuration of the monitoring array was designed to capture the in plane and out of plane ground motion during the different tests. In total more than 50 individual sensors were deployed on and around the prototype structure, to study wave propagation characteristics and SSI.

The soil was instrumented with 13 seismometers, 1 down-hole accelerometer and 2 SAAs, covering the two horizontal and the vertical directions. The structure was instrumented with 6 accelerometers and 2 SAAs. Part of the instrumentation in the soil and on the structure can be seen in Fig. 27.

Fig. 27. Part of the instrumentation in the soil (left) and on the structure (right). In the soil the seismometers and the horizontal SAA (in the trench) can be distinguished.

The pull-out force in these experiments ranged from 1.8kN to 11.2kN and was applied on the structure by a wire rope. Some preliminary recordings from the seismometers can be seen in Fig. 28.

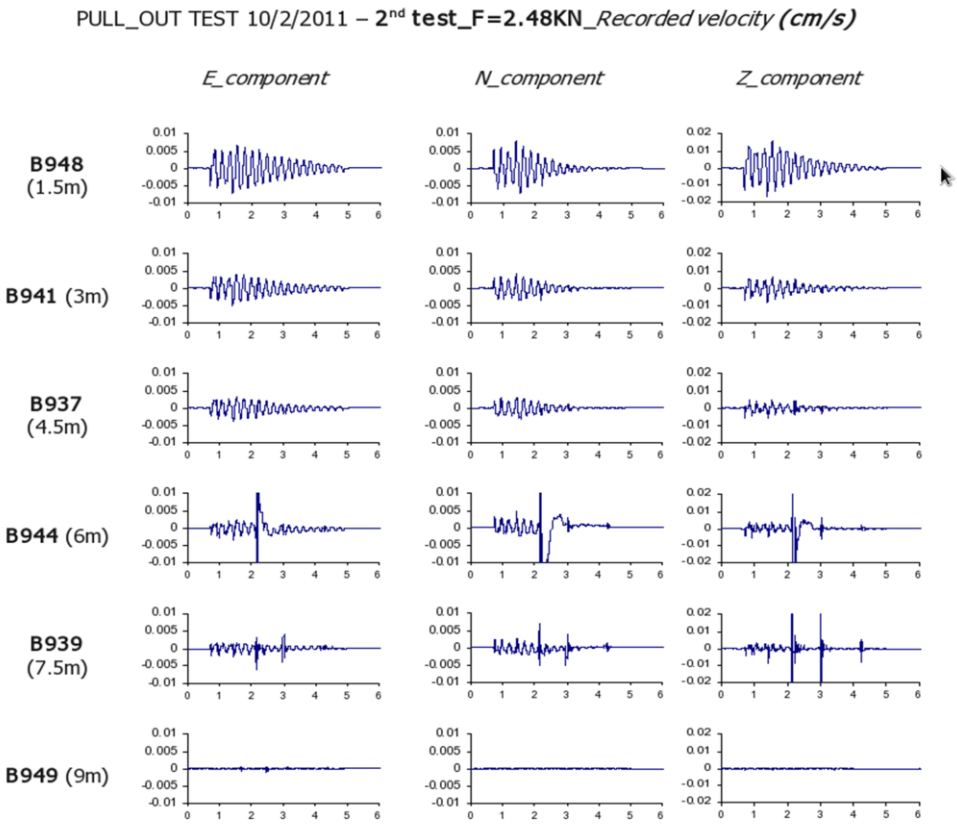



Fig. 28. Recordings based on an applied force of 2.48kN; in three directions (north, east and vertical); distances vary from 1.5m to 9m from the foundation

In Fig. 29 is shown the decay of the recorded velocity with distance from the foundation, for different levels of pull-out force.

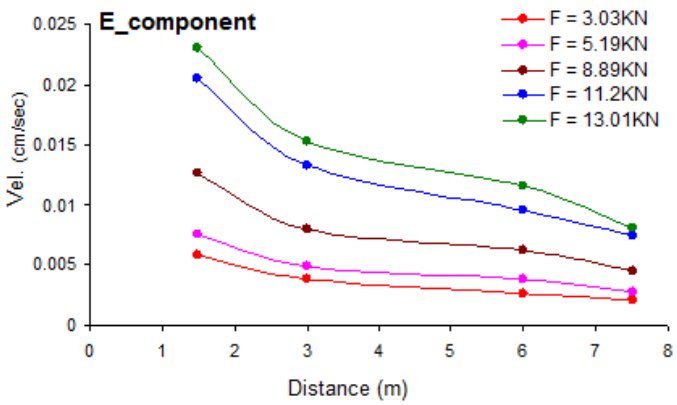



Fig. 29. Recorded velocity decrease with distance from foundation for different pull-out force levels.

## *5.5 Sanctuary of Vicoforte*

### *5.5.1 Quick facts*

The Sanctuary of Vicoforte (Fig. 30) is a monumental church located in Vicoforte, a small town close to Cuneo in the north-west of Italy. It is known for having the largest elliptical dome in Europe. Recently, the Sanctuary has been undergoing a delicate renovation process including interventions on the main dome and on some other structural parts. Dynamic identification tests were used to characterise the structure, its behaviour under wind excitation and to have a benchmark to compare to after the execution of the retrofit interventions.
Detailed information on this study can be found in the Eucentre Test Report (2008).

### *5.5.2 Building description*

The masonry structure under investigation is a monumental church with typical characteristics of the late Italian Renaissance whose construction began in 1596 by the architect Ascanio Vitozzi. In 1710, the architect Francesco Gallo started the construction of the upper part of the structure and in 1731 the dome was built. The dome is currently the biggest in Europe being 74 m in height and having diameters equal to 24.90 m and 37.15 m. The church was completed in the second half of the 19th century with the construction of the four bell towers.

Fig. 30. Sanctuary of Vicoforte

### *5.5.3 Objective*

Objective of the testing campaign was the determination of the main natural frequency of the structure. Particular attention was given to the dome and to the counterforts trying to evaluate their dynamic characteristics including natural periods of vibration and approximate mode shapes. Final target was the assessment of the structural behaviour with respect to external vibration sources such as traffic and wind.

### *5.5.4 Methodology of measurement*

Different instruments were used for the characterisation of the different parts of the structure. In particular, both velocimeters and piezo-electronic accelerometers were installed in the upper face of the dome, just below the roof. The initial phase of acquisition was performed using the tri-axial velocimeter with the objective of finding the best possible

configuration of the instruments. After this, the piezo-electronic accelerometers were installed and connected to a permanent monitoring system. The following Fig. 31 shows the tri-axial geophones and the accelerometers installed on the top of the dome.

Fig. 31. Geophone and accelerometers used for the dynamic identification of the dome

The lower part of the church, including the drum and the inner columns, was identified using the tri-axial geophones. Again, different instruments configurations were adopted to characterise the whole structure. Multiple recordings were needed to find the optimal instruments configuration and to cover all the structural elements under investigation. The following Fig. 32 shows two of the adopted arrangements of the instruments respectively for the identification of the dome and of the drum.

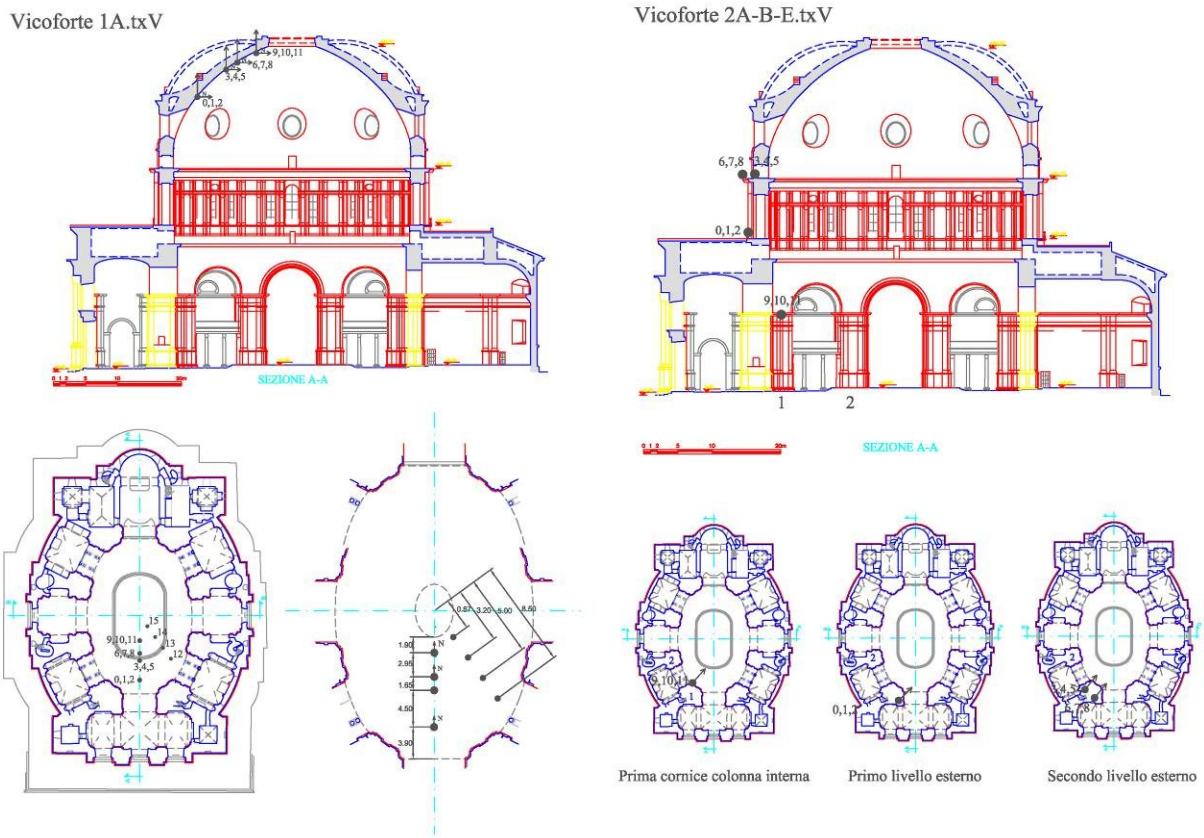



Fig. 32. Position of geophone and accelerometers used for the dynamic identification

### *5.5.5 System identification method*

Because of the importance of the structure that belongs to the historic wealth of Italy, ambient vibrations produced by wind and surrounding traffic were the only possible acceptable input for the characterisation of the dome. A number of acquisitions were performed changing the position of the instruments since the best configuration was not known a priori. This was due to the fact that the structure under investigation was very complex and the state of preservation of the materials was uncertain, hence leading to a quite unreliable numerical model of the structure. Once an effective instruments configuration was found, a permanent monitoring system was installed with an automatic trigger based on the wind intensity measured by an anemometer installed in the upper part of the dome (see Fig. 33).

On the other hand, for the dynamic characterisation of the columns inside the church, since the wind induced vibrations were too weak to produce meaningful signals, it was chosen to use forced vibrations. As previously mentioned, it was not possible to install a vibrodyne nor to use impulsive excitation: the solution was to use the bells of the church that were found to be able to produce sufficiently strong vibrations.

Fig. 33. Anemometer installed in the upper part of the church

### *5.5.6 Data analysis*

The acquired signals were analysed to estimate the natural vibration periods and the approximate modal shapes. First of all, a low-pass filter at 50 Hz was applied to the signal to cut all the high frequency noise that could jeopardise the dynamic identification analysis process.

Data processing encompassed signal filtering and Fast Fourier Transformation (FFT), followed by computations of the Power Spectral Density (PSD) and all the cross-spectra. These analyses were used to indentify the first natural frequencies: the dome and the lower part of the church were found to have a natural frequency at 2.13 Hz and 1.94 Hz respectively in the longitudinal and transversal direction. For the dome alone, it was also possible to determine two natural vibration frequencies at 5.25 Hz and 6.13 Hz. Also the first fundamental periods of the counterforts were evaluated (see Table 5).

Table 5. Identified natural frequencies of the counterforts

| **Counterfort #1** | **Counterfort #2** |
|---|---|
| 1.63 Hz | 1.63 Hz |
| 3.00 Hz | 3.00 Hz |
| 3.75 Hz | 3.56 Hz |

As an example of the analyses performed on the acquired data, the FFT, some of the cross-spectra and some frequency-time-PSD amplitude plots are displayed in Fig. 34, Fig. 35 and Fig. 36 (plots are relative to instrument configuration showed on the right-hand side of Fig. 32).

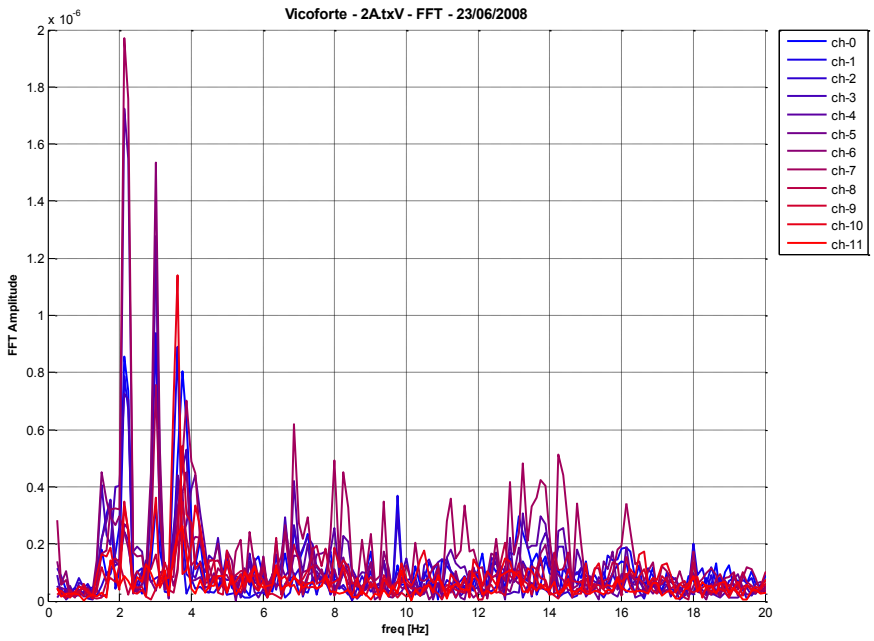



Fig. 34. FFT of the acquired signals

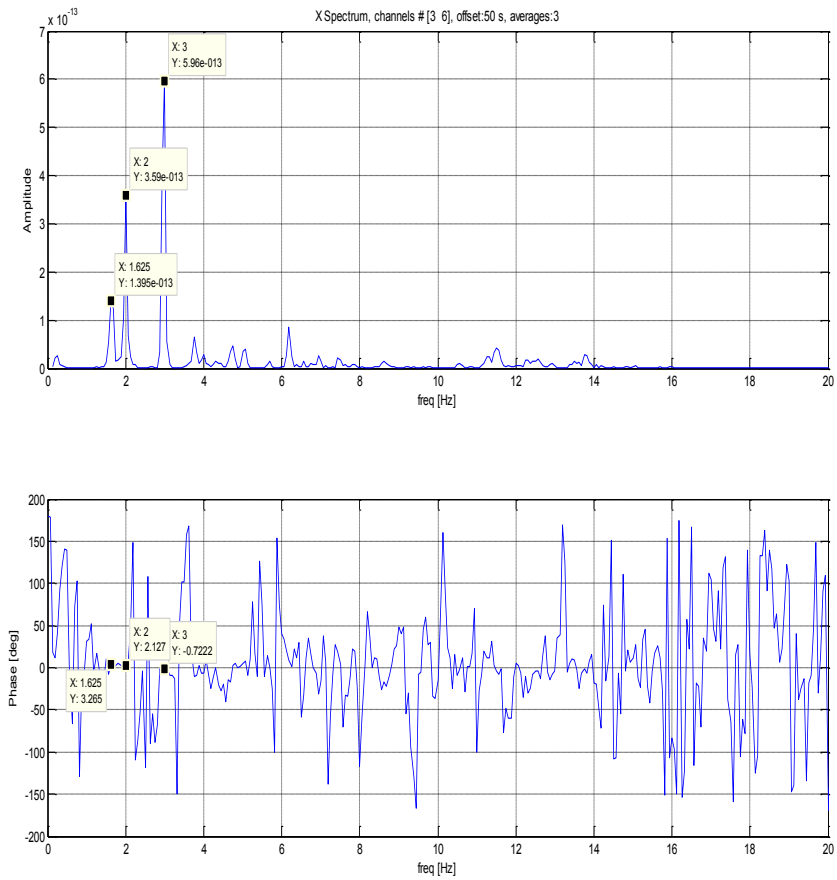



Fig. 35. Cross-spectra (amplitude and phase) of channels 3 and 6

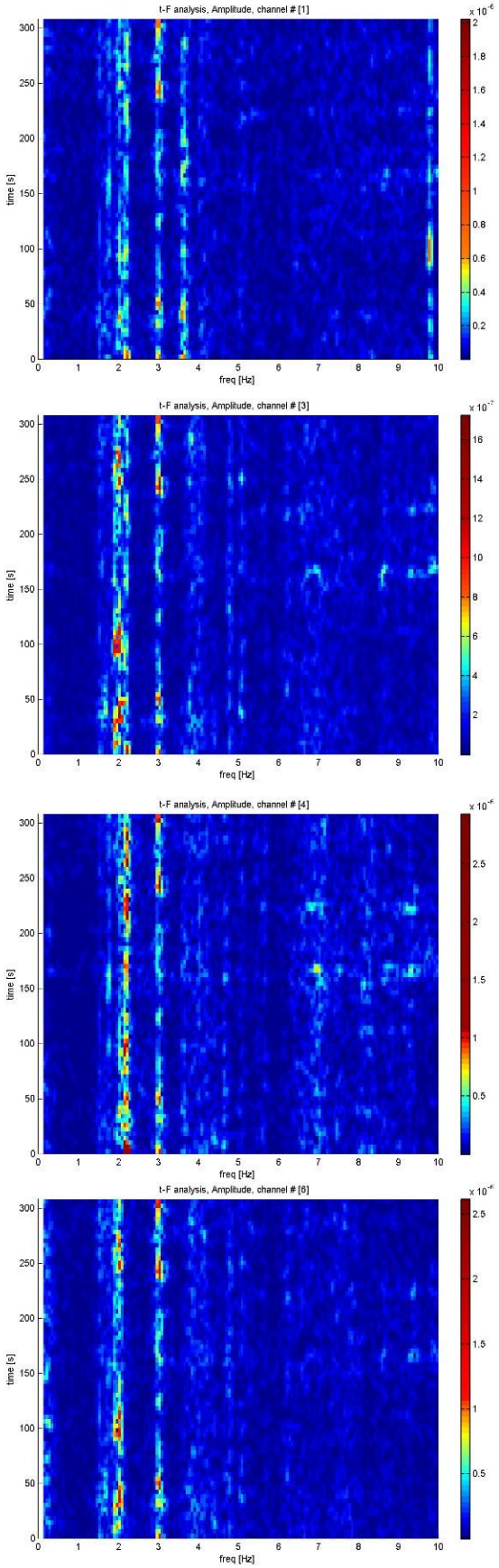



Fig. 36. Frequency-Time-PSD amplitude plots of channels 1-3-4-6

### *5.5.7 Benefits of using field monitoring*

In most of the cases of intervention aiming at the preservation or structural retrofit of historic structures, dynamic identification tests and non-destructive tests, such as those based on ultrasonic devices and georadar technique, are one of the few source of information for the designers. Also in this case, these kinds of scientific investigations were the only acceptable ones. The cross-evaluation of all the test results led to the determination of the most critical elements in the sanctuary. Furthermore, it was possible to recognise which of the several cracks in the domes were actually a sign of a potential problem connected to the structural stability of the dome itself.

## *5.6 Pedestrian tunnel at Malpensa Airport*

### *5.6.1 Quick facts*

The Malpensa Airport, the main airport of Milan (Italy), is currently undergoing an enlargement phase including the construction of new buildings and connection paths between them. In this framework, the designers asked for the execution of dynamic identification tests on different structure for the validation of the finite element models used for the design. The experimental activities were performed using different excitations, ambient and forced vibrations, for the structures under investigation.
Detailed information on this study can be found in the Eucentre Test Report (2011).

### *5.6.2 Building description*

The structure considered within this case study is an elevated pedestrian tunnel connecting different parts of the airport. The total length of the construction equals 100 m while its square cross-section is about 6 m by 6 m featuring a two stories walking path. The following Fig. 37 shows the analysed structure, it is worth to mention that the tunnel is actually rectilinear despite the effect of the wide angle lens used to photograph the structure.

Fig. 37. Pedestrian steel-glass tunnel at Malpensa Airport

As clear from the picture, the tunnel has a steel-glass structure as well as most of the Malpensa Airport buildings, while the slabs have a mixed steel-concrete structure. Furthermore, it has to be noticed that the two white lines crossing vertically the glass side of the tunnel are two expansion joints cutting the structure at about 25 m from both its ends.

### *5.6.3 Objective*

Objective of the experimental campaign was the determination of the dynamic characteristics of the structure, aiming in particular at the evaluation of the first modes of vibration and natural frequencies. These data have then been used for the validation of a finite element model used for the design. The designers wanted to use a calibrated model for the evaluation of the effects induced by the people walking inside the tunnel in the attempt of avoiding problems related to vibrations induced by the normal use of the structure.

### *5.6.4 Methodology of measurement*

The investigated structure was excited using different vibration sources including ambient vibrations, impulsive vibrations and forced vibrations generated by means of a vibrodyne. In particular the unbalanced rotating masses of the vibrodyne generated sinusoidal waves at changing frequencies within the expected range of interest. The following Fig. 38 shows the vibrodyne installed inside the pedestrian tunnel and a detail of its fixing realised through bolts crossing one of the tunnel concrete-steel slabs.

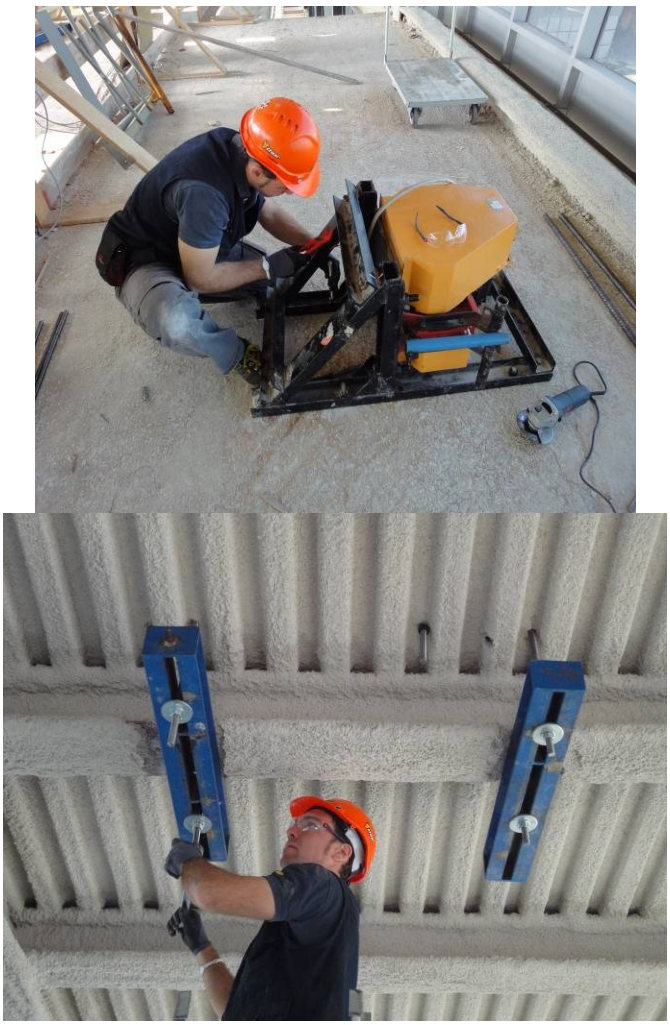


Fig. 38. Vibrodyne installed inside the tunnel and connection detail

The tunnel has then been instrumented installing a series of seven tri-axial seismometers (Lennartz LE3D-5s) as reported in the following Fig. 39 showing a scheme of the instruments location. For the determination of such positions, an approximate finite element model was used to located the tunnel zones with the maximum vibration amplitude due to the natural resonance of the structure.

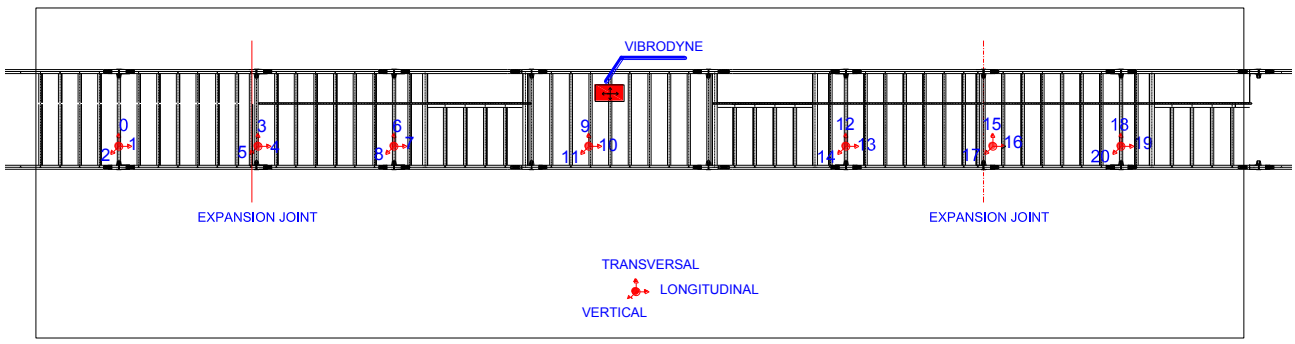



Fig. 39. Location of the tri-axial seismometers

The following Fig. 40 shows a picture of the instruments placed inside the tunnel and a screenshot of the acquisition software during one of the tests.


Fig. 40. Installed instrument chain and acquisition software screenshot

The geophones were then connected to the acquisition system composed of a signal conditioner board, an analog-digital converter and the personal computer recording the signals. The data was acquired at 256 Hz with 16 bit samples.

### *5.6.5 System identification method*

The identification was performed in three different phases essentially changing the type of excitation. First of all, the movements caused by ambient vibrations were acquired, then in a second phase people walking back and forth randomly was used to introduce structural vibrations. Finally, once the natural periods of the structure were roughly estimated, a vibrodyne was used to generate forced vibrations. The rotating masses of the vibrodyne were used to generate sinusoidal waves: the direction of excitation was set along the three principal directions in which the structure was supposed to move (i.e. longitudinal, transversal and vertical) and the frequency of the excitation was varied within the range of interest with 0.2 Hz steps.

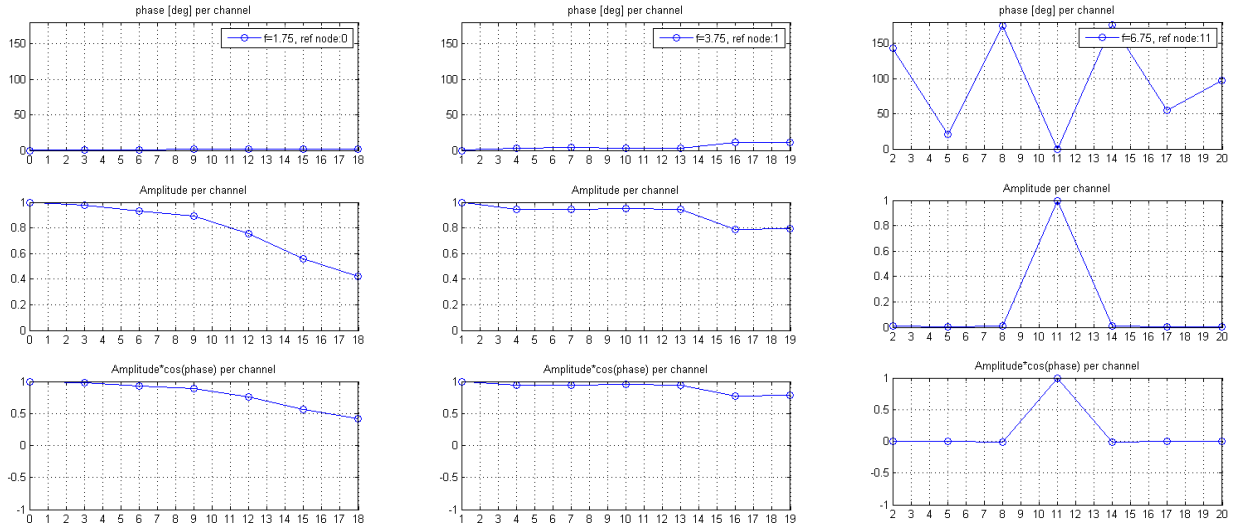
### *5.6.6 Data analysis*

The acquired signals were analysed to allow the estimation of the parameters of interest. In a first procedure a low-pass filter at 50 Hz was applied in order to cut all high frequency noise that could jeopardise the dynamic identification analysis process.
After signal filtering, FFT was applied followed by PSD and cross-spectra computations. These analyses were used to indentify the first two natural frequencies along the three principal directions (i.e. longitudinal, transversal and vertical) of the structure (see Table 6).

Table 6. Identified natural frequencies

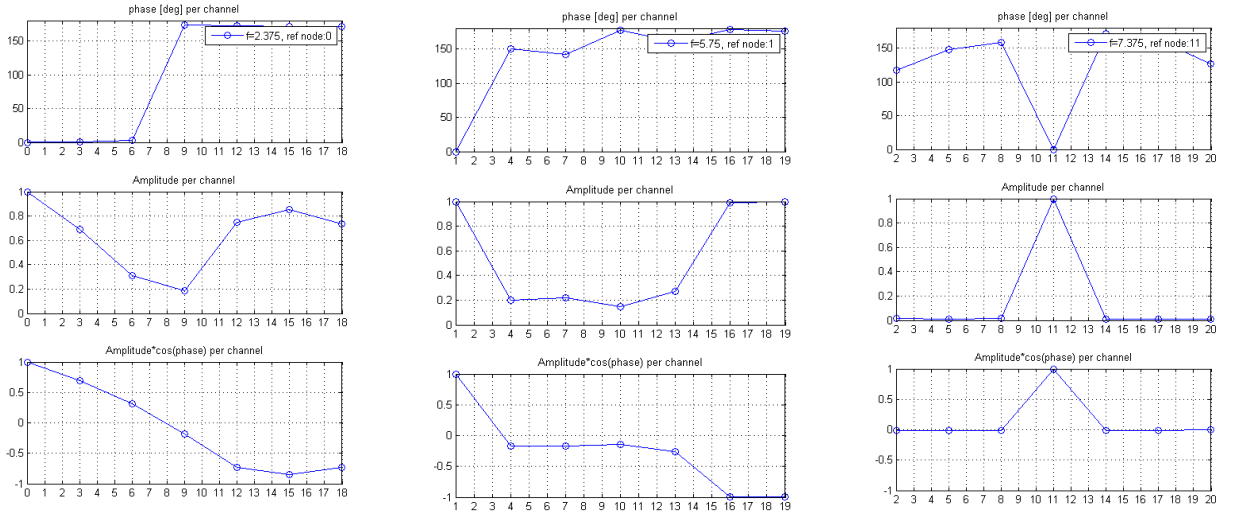
| **Direction** | **1st Natural Frequency** | **2nd Natural Frequency** |
|---|---|---|
| Longitudinal | 3.75 Hz | 5.75 Hz |
| Transversal | 1.75 Hz | 2.37 Hz |
| Vertical | 6.75 Hz | 7.37 Hz |

The results are presented in Fig. 41 and Fig. 42 displaying the signal phases/amplitudes for the first and second natural frequencies respectively.




**(a)** **(b)** **(c)**

Fig. 41. Data derived by the cross-spectra:
(a) transversal channels at 1.75 Hz; (b) longitudinal channels at 3.75 Hz; (c) Vertical channels at 6.75 Hz




**(a)** **(b)** **(c)**

Fig. 42. Data derived by the cross-spectra:
(a) transversal channels at 2.37 Hz; (b) longitudinal channels at 5.75 Hz; (c) Vertical channels at 7.37 Hz

Talking about the results of the investigation on the vertical modes, it has to be noted that the signals of the different channels are not in phase as one could expect at least for the first mode. This was probably due to the effect of the expansion joints located along the pedestrian tunnel: these joints, although they were not seismic ones and just allowed very small relative movements, were probably able to decouple the vibration of the different part of the structure. Furthermore, it has to be underlined that, for reason of structural safety, it was not possible to use the vibrodyne to vertically excite the structure above 7 Hz.

The following Fig. 43 shows two examples of a frequency-time-amplitude plot for two transversal channels (channels 0 and 3 were the transversal ones towards the left end of the structure). From these two plots, relative to an acquisition with ambient vibrations, it is possible to guess the first transversal natural vibration frequency identified by the red spots at about 2 Hz.

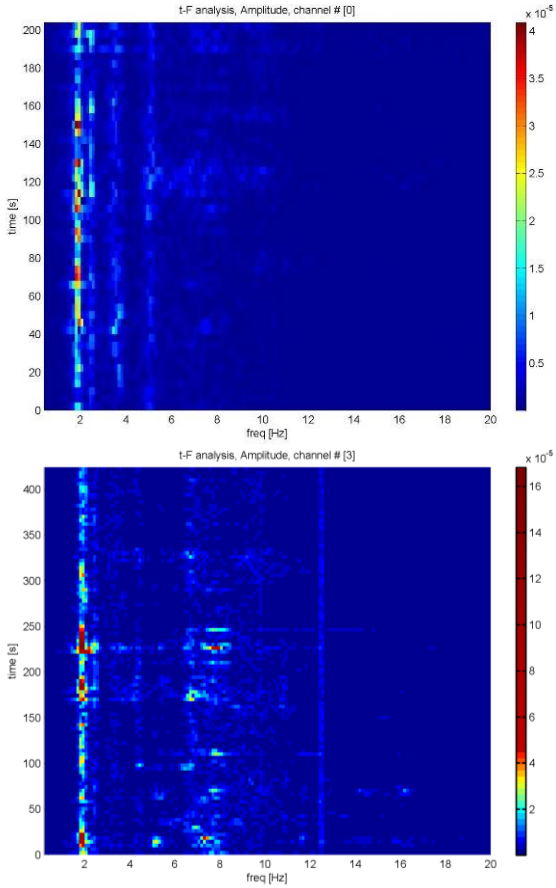
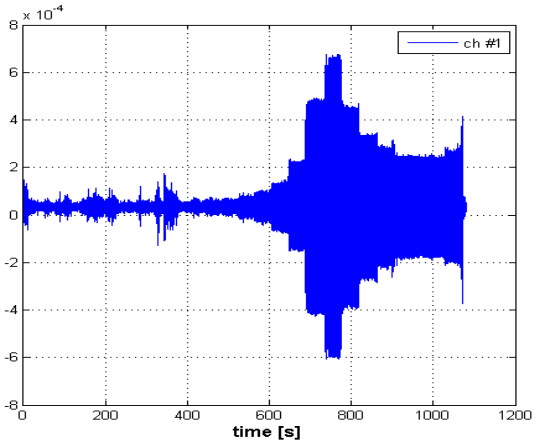
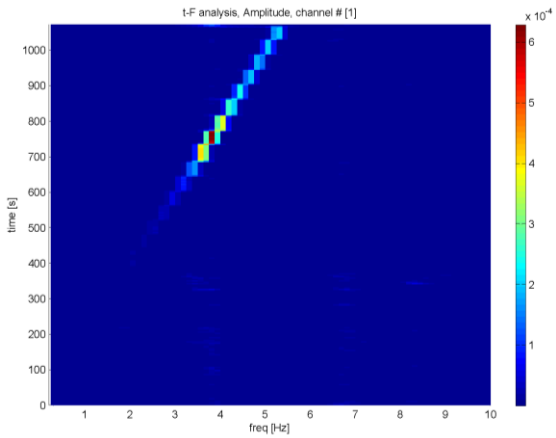



Fig. 43. Frequency-Time-Amplitude plots of channel 0 and 3 (transversal direction)

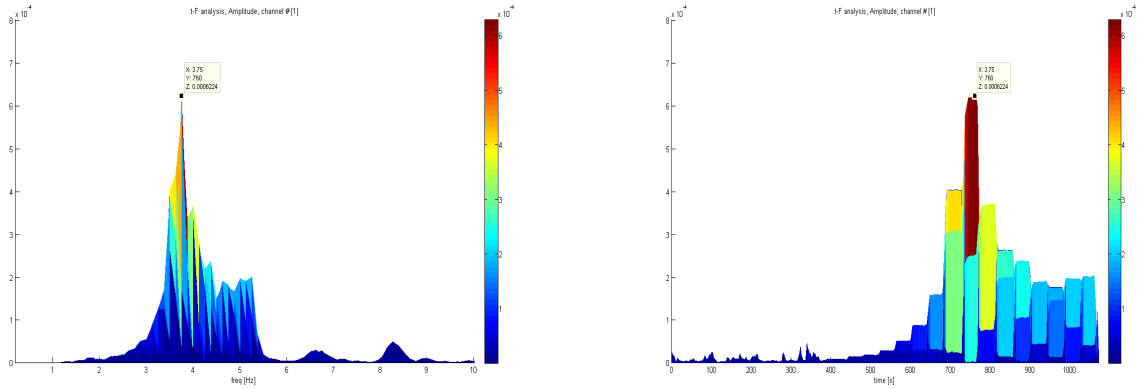
Finally, acquisition results based on longitudinal forced vibrations are displayed in the following Fig. 44.




(a)




(b)




(c) (d)

Fig. 44. Longitudinal forced vibrations: (a) acquired signal; (b) frequency-time-PSD amplitude;
(c) Amplitude of the PSD vs. frequency; (d) Amplitude of the PSD vs. time

From Fig. 44b, it is interesting to see a region with higher PSD amplitude identified by the diagonal trace on the plot. This is a typical output of a test performed using a vibrodyne when the frequency of the induced vibrations (i.e. the excited frequency) is increased in time during the test execution: the plot shows that the frequency of the induced vibration was about 2 Hz at the beginning of the test and it was increased up to about 5.5 Hz. In the same plot, a peak on the amplitude can be noticed at about 4 Hz. Plotting the amplitude against the frequency, the dominant peak at the resonant frequency (3.75 Hz) is well evident.

### *5.6.7 Benefits of using field monitoring*

Dynamic identification led to the estimation of the first natural frequencies and damping of the pedestrian tunnel under investigation. The results helped to validate and further improve the numerical model before performing the final checks required by the design code. Furthermore, it was possible to test the structure against the vibration input due to pedestrians moving along the walking paths. This test led to the conclusion that such input was not inducing resonant effects on the structure and no additional damping or structural modifications were needed.

## *5.7 Building structure*

### *5.7.1 Measurement design for buildings*

The strategy to design a measurement grid and to rove sensors for multi-storey buildings might follow a two-step approach: (i) to measure the (horizontal) dynamic behavior of the global structure, i.e. the columns and associated beams that form the principal structural framing of the building; (ii) to measure the (vertical) vibration of the floors. For a conventional floor with primary and secondary beams as in Fig. 45, the measurement grid is often based on the layout of the beams. For example, if the primary beams are considered very stiff compared to the secondary beams, the measurement line strategy can be adopted in which the measurement lines are based on the secondary beams and rove along the primary beams. Further information can be found in two guidelines (Pavic et al. 2008a; Pavic et al. 2008b; Smith et al. 2009).

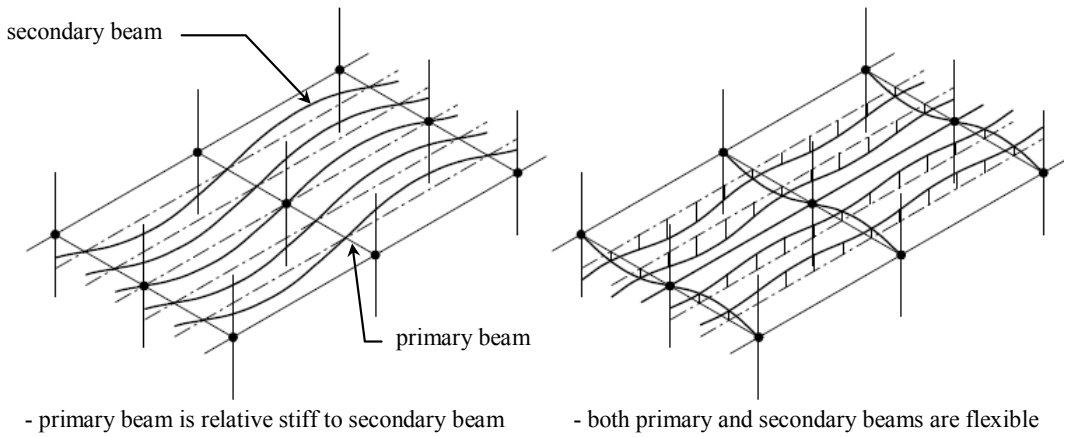



Fig. 45. Typical assumption for a beam column system

### *5.7.2 Ambient vibration testing of the data center – K.U.Leuven*

Modal identification of the floor under both forced and ambient excitation is the main focus of this investigation. The measurement campaign is planned in two phases. The first phase is to test the floor while the building is under construction. The second phase starts when the construction is completed. The main objective is to study the changes of natural frequencies and damping values.

The floor under investigation of the data center – K.U.Leuven is made of precast concrete panels. The pre-stressed panels have a length of 15.15m, width of 1.2m and height of 0.4m. The overlay is 0.06m making the total thickness of the slab 0.46m.

The floor has two concrete cores which are elevator and staircase shafts. The zone in the middle, apart from the cores is constructed as reinforced concrete which is thinner than the total pre-stressed slab. The left and right span are symmetric and identical. Both spans have a width of 15 m and a length of 36 m. Dimensions of the squared symmetric floor are shown in Fig. 46.

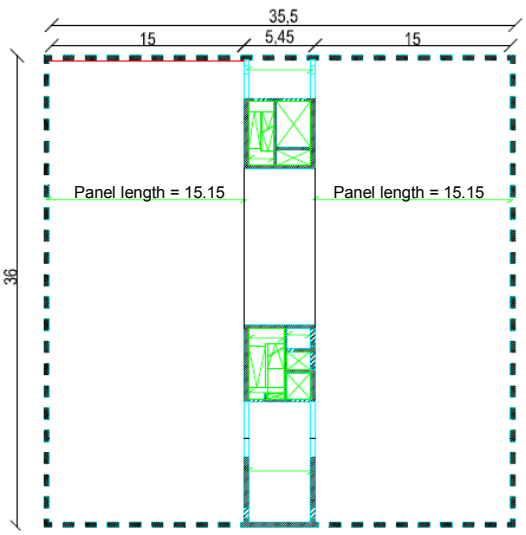



Fig. 46. Structural arrangement of the floor of the data center (dimensions are in meters)

In order to plan the test, a finite element model was built and the first twenty modes were extracted. The floor is modeled with shell elements. A fixed boundary condition is adapted on the outside circumference as well as at locations of the elevator shafts. Fig. 47 shows the first three mode shapes of the floor. Taking into account the symmetry, a dense measurement was designed with three reference nodes which are common to all setups (Fig. 48).

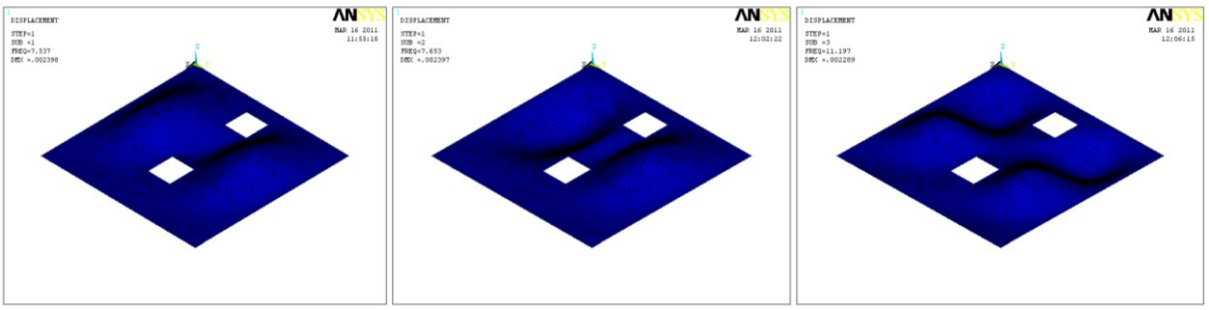


Fig. 47. The first three modes of the finite element model

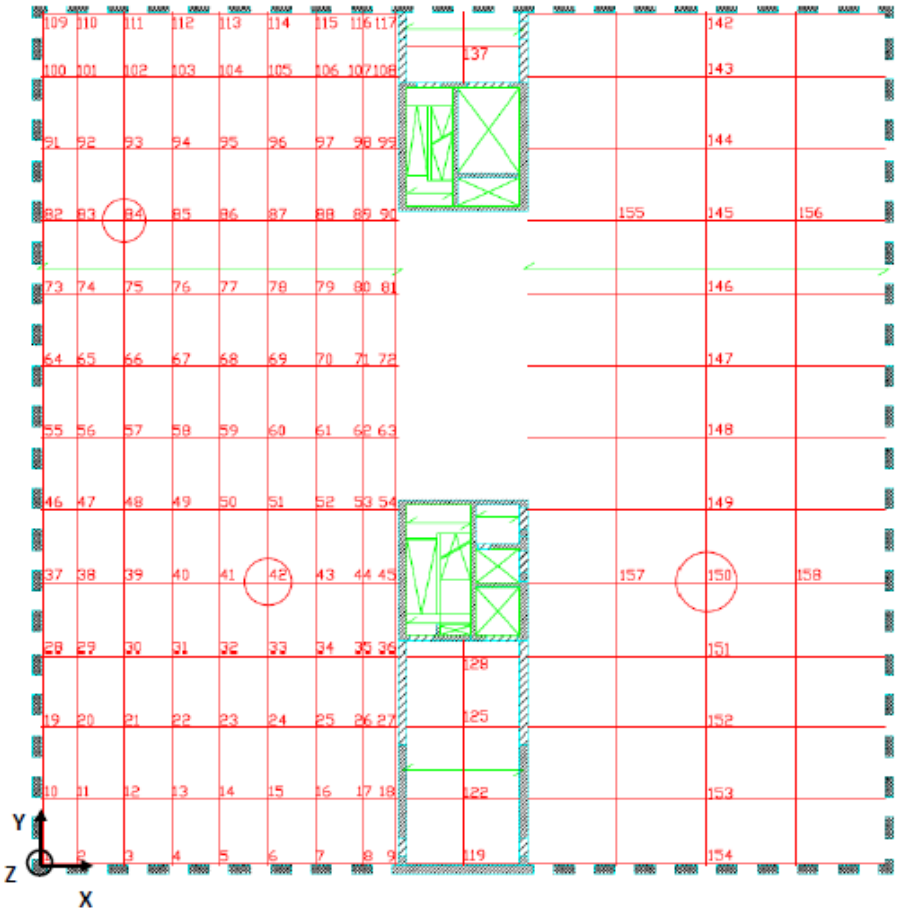
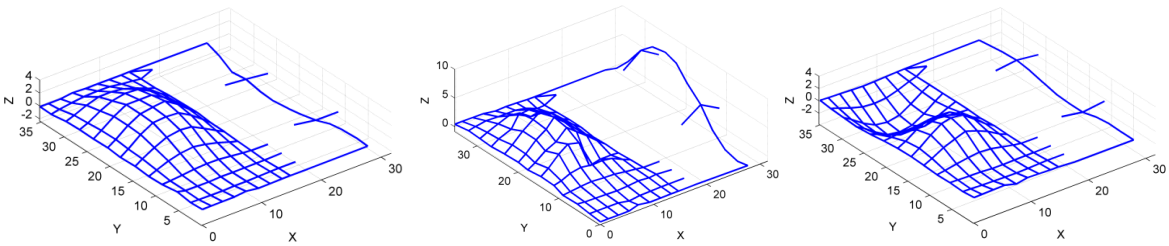



Fig. 48. The measurement grid

*First results*

Since the building is still under construction, limited test results are available from the first phase of the measurement campaign. Twelve modes have been found in a frequency range from 8.26 to 27.67 Hz. The first three modes are given in Fig. 49.




Mode 1 [8.26 Hz, 4.32%] Mode 2 [8.56 Hz, 2.39%] Mode 3 [11.09 Hz, 2.18%]

Fig. 49. The first three identified modes of the floor

### *5.8 Bridge structure*

An ambient vibration test was performed on the Guadalquivir bridge near the town of Lora del Rio in Andalusia, Spain. It is a rail bridge of the line between Seville and Alcázar. The focus of this test was to identify natural frequencies, mode shapes and other modal parameters.

The Guadalquivir bridge (Fig. 50) – "PUENTE DEL GUADALQUIVIR" in Spanish – is a twin steel truss bridge with one rail track in each direction. The bridge consists of five continuous truss spans of almost equal length: 50.48 + 50.94 + 50.94 + 50.94 + 50.61 (m). The abutment on the Alcázar side is referred to as E-1 (Fig. 51). The four piers are numbered as P-1, P-2, P-3, P-4, starting from the E-1 side. The last (fifth) span goes from P-4 to the abutment E-2 on the Seville side. The fixed bearings of the bridge are located on top of pier P-2.

The bridge was built in 1929 and has been strengthened in several occasions by adding reinforcements to its members. Visual inspection shows that the condition of the bridge is reasonably good in spite of several spot defects in the structural members and connection bolts.

Fig. 50: The Guadalquivir river rail bridge

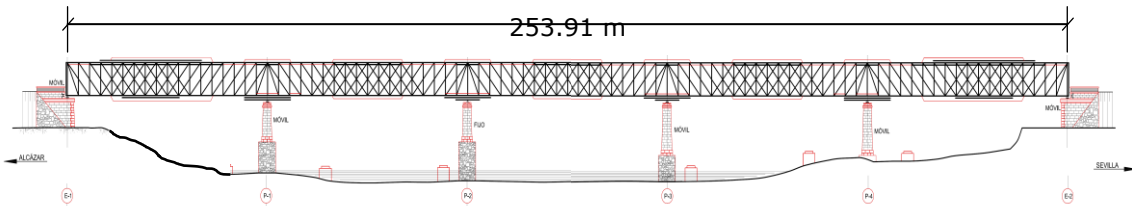



Fig. 51. Side view of the Guadalquivir river rail bridge

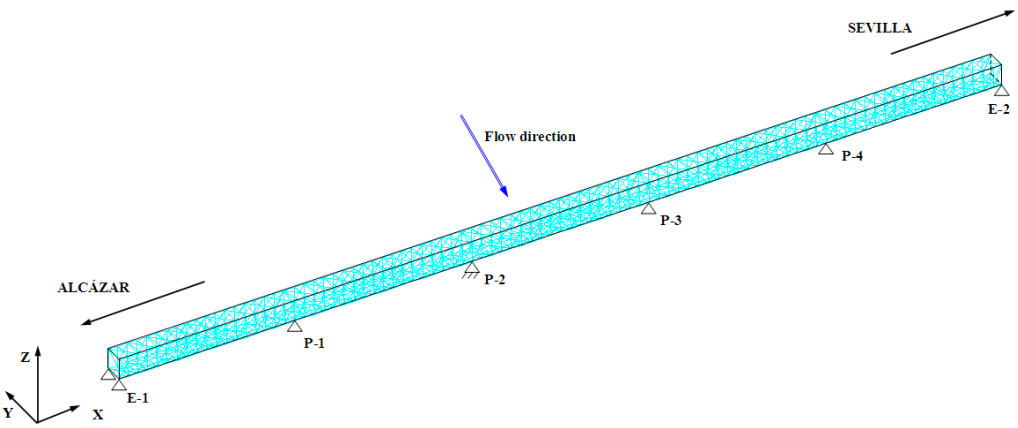



Fig. 52. Schematic view of the structural arrangement of the bridge

### *5.8.1 A pre-test finite element model*

In order to plan and design the test setup, a finite element (FE) model was built based on the geometry from site visit and inspection. This model is to conceptualize the vibration characteristics of the bridge. Based on that, a measurement grid is designed for the actual field test. Main structural members (top chords, bottom chords and verticals) are modeled using beam elements. The transverse beams (of the deck system) are modeled using a special tapered beam element type. The diagonals (of the main truss), the bracing members and the contra-braking truss are modeled using bar elements. The rail track and non-structural components, i.e. power lines, hand rail, maintenance paths, are included in the model as added mass. The contra-braking truss is connected to the main truss by translational constraints to the nodal points. In the longitudinal direction, two supports on the pier P-2 are fixed, while in the transverse direction, all supports are fixed.

The global X-axis is in the longitudinal direction of the bridge; the Y-axis is in the transverse direction – opposite to the river flow direction and the Z-axis is in the vertical direction (see Fig. 52). The left main truss is the one on the upstream side. Likewise, the right main truss is the one on the downstream side. Both trusses represent the side view of the bridge.

Table 7 shows the summary of the first twenty extracted mode shapes, featuring a quite high modal density within the frequency range from 3.06 Hz to 8.19 Hz. Some modes have very close frequencies, which might pose a challenge to separate them from the identification results in the later modal analysis step. Within this frequency interval, all modes are global.

Table 7. Summary of the first twenty extracted mode shapes from the FE model

| Nr. | Frequency (Hz) | Mode type | Note |
|---|---|---|---|
| 1 | 3.06 | First transverse mode | |
| 2 | 3.09 | Second transverse mode | Close to the previous mode |
| 3 | 3.25 | Third transverse mode | |
| 4 | 3.52 | Fourth transverse mode | |
| 5 | 3.92 | Fifth transverse mode | |
| *6* | *4.49* | *First vertical mode* | |
| *7* | *5.10* | *Second vertical mode* | |
| *8* | *5.25* | *Third vertical mode* | |
| 9 | 5.47 | Sixth transverse mode | |
| 10 | 5.53 | Seventh transverse mode | Close to the previous mode |
| *11* | *5.88* | *Forth vertical mode* | |
| 12 | 5.89 | Eighth transverse mode | Close to the previous mode |
| 13 | 6.03 | Ninth transverse mode | |
| 14 | 6.09 | Tenth transverse mode | Close to the previous mode |
| 15 | 6.15 | Eleventh transverse mode | Close to the previous mode |
| *16* | *6.86* | *Fifth vertical mode* | |
| *17* | *7.06* | *Sixth vertical mode* | |
| 18 | 7.66 | First torsional mode | |
| *19* | *7.79* | *Seventh vertical mode* | |
| 20 | 8.19 | Second torsional mode | |

The finite element results show that for many modes modal displacements occur in all three directions. The first extracted vertical mode has a natural frequency of 4.49 Hz. In total, seven vertical modes have been found up to a frequency of 7.79 Hz (Fig. 53). These are important modes because they are in the main load direction of the bridge and also well separated from others (except for the fourth vertical mode). It can be seen that the vertical modal displacements are largest in the fourth span. The third and the fifth spans also have significant modal deflections.

Fig. 54 is an overview of several mode shapes in the considered frequency range. Mode shape 1, f = 3.06 Hz, is a transverse mode. In this mode, the truss mainly deforms in the transverse direction (transverse "bending"). The top bracing truss is supported by portal frames, whereas the bottom bracing truss is supported directly by the sub-structure, which is much stiffer. Therefore this mode looks like a sway of the truss in the transverse direction. Mode shape 6, f = 4.49 Hz, is the first vertical mode. In this mode, the truss is mainly moving in the vertical direction (Z-axis). It is often associated with a longitudinal movement

(X-axis) because only the support on pier P-2 is fixed. A vertical mode can be seen as a "bending" mode, conceptually. Mode shape 18, f = 7.66 Hz, is a torsional mode. In this mode, the cross section rotates and the truss vibrates in both vertical and transverse directions. This mode may also be referred to as a combination mode.

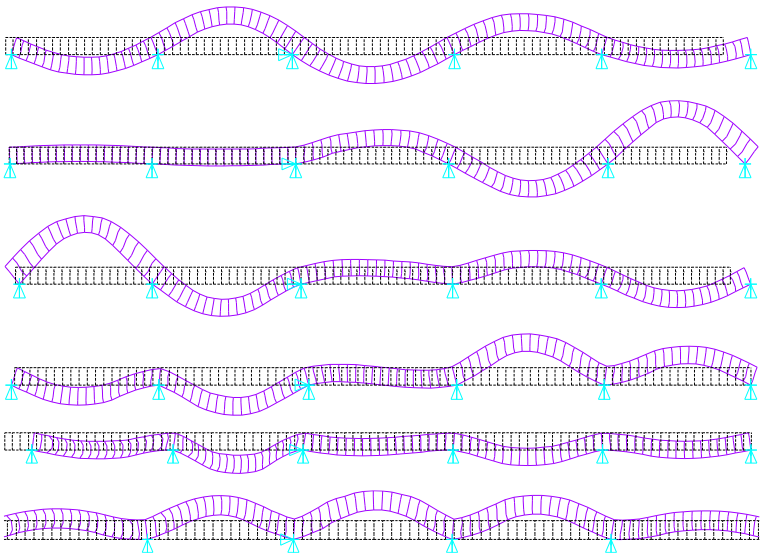
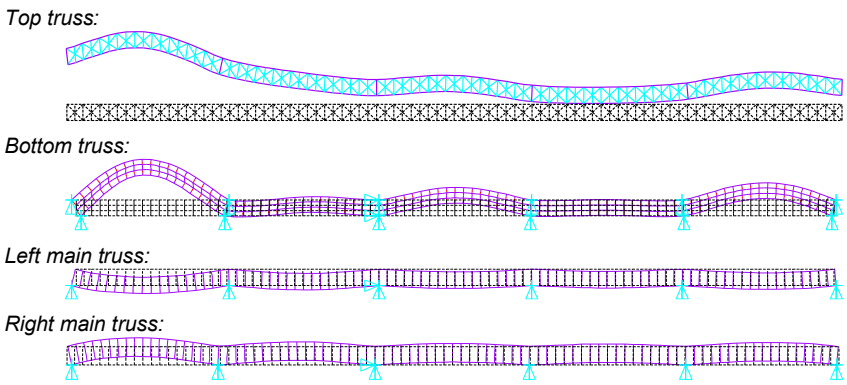
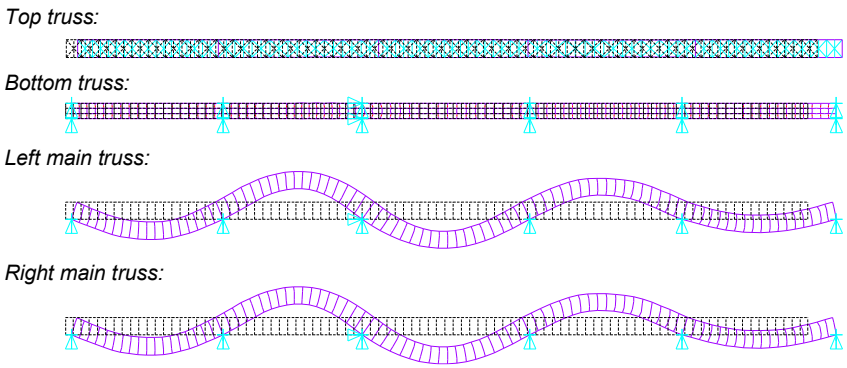


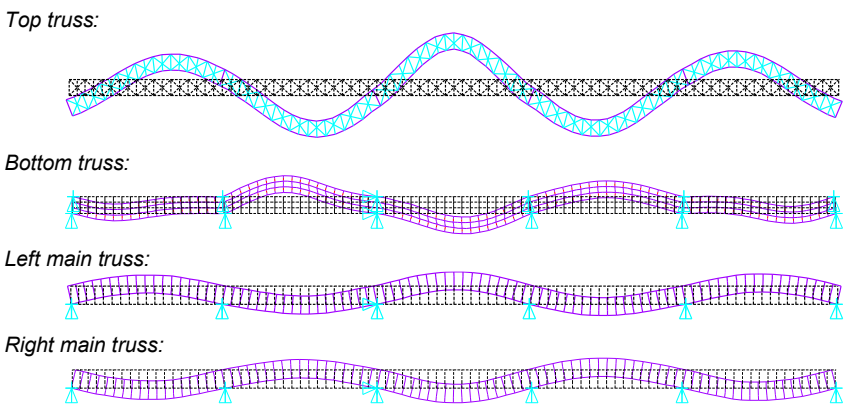
Fig. 53. The first six bending mode shapes extracted from the FE model




(a) Mode shape 1, f = 3.06 Hz.




(b) Mode shape 6, f = 4.49 Hz.




(c) Mode shape 18, f = 7.66 Hz.

Fig. 54. Overview of some typical extracted mode shapes from the FE model

### *5.8.2 The ambient vibration test*

The ambient vibration test was performed on one of the twin bridges at the upstream side of the river (for trains in the Seville – Alcázar direction). Excitation sources are ambient forces, e.g. wind, or the free vibration of the bridge after train passage. Ideally, for this kind of bridge structure, the measurement grid would be coincident with all nodes at the bottom and at the top of the truss. However due to safety reasons (there is no hand rail on the top chords), the measurement nodes were mainly on the bottom chords of the truss. Only a limited number of nodes in the top chords were measured. Fig. 55 shows the measurement grid of the bridge with all joints on the bottom chords measured.

Twelve three-directional acceleration wireless sensors (GEOSIG) were used. Therefore 12 x 3 = 36 degrees-of-freedom can be simultaneously measured. The sampling frequency is 200 Hz. The measurement was divided into different setups with five fixed reference sensors distributed on the first, second, third and fourth span. These reference sensors are in the locations of significant modal displacements of many modes. The other seven sensors were roved in "modules" to cover all the remaining measurement grid points. Fig. 56 shows the placement of sensor units for setup 1. There were twenty-seven setups in total. The last setup (setup 27, Fig. 57) was to measure the vibration of the portal frame and also the torsional modes. Therefore, four sensors were installed on the accessible top chord nodes. Measurement time was about fifteen to twenty minutes per setup. Fig. 58 shows the placement of sensor units in a top node and in a bottom node of the bridge.

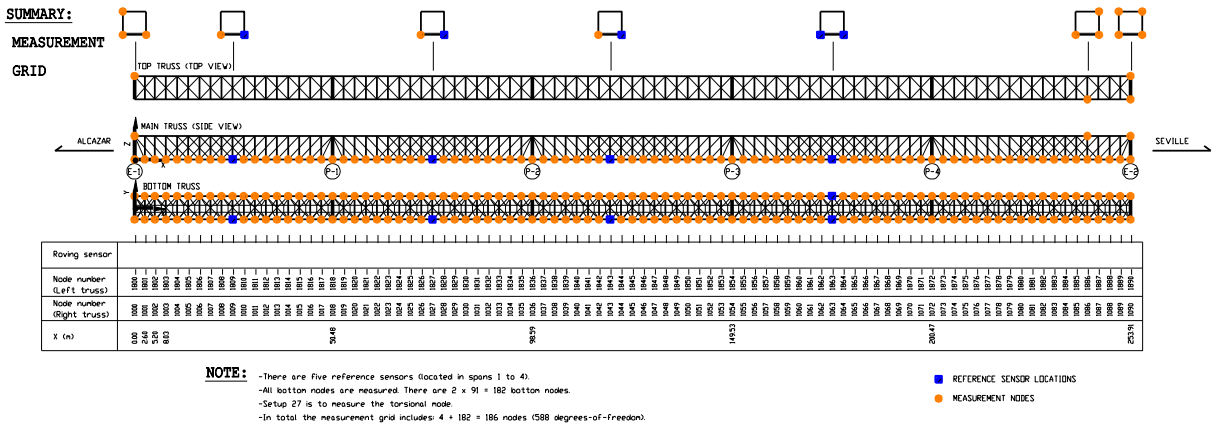



Fig. 55. The measurement grid with all the bottom nodes and 4 top nodes

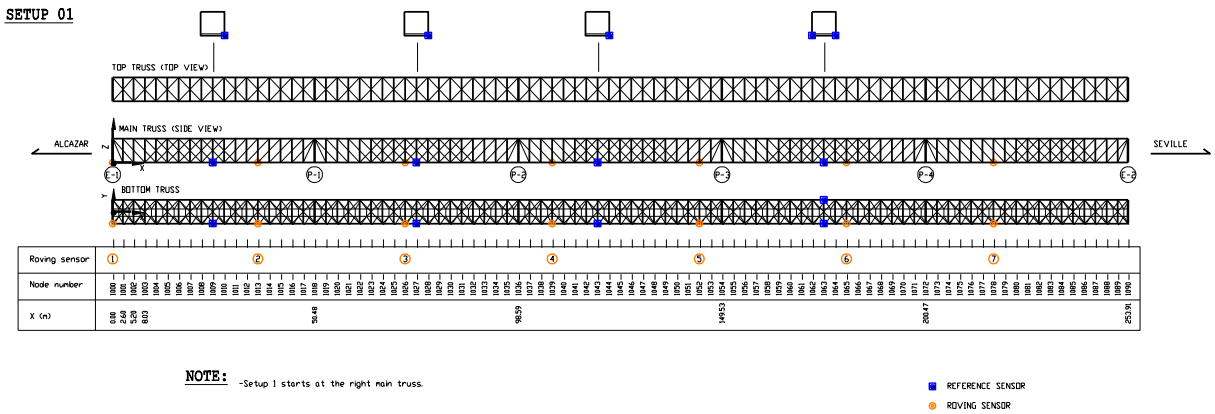



Fig. 56. Arrangement of sensor for setup 1

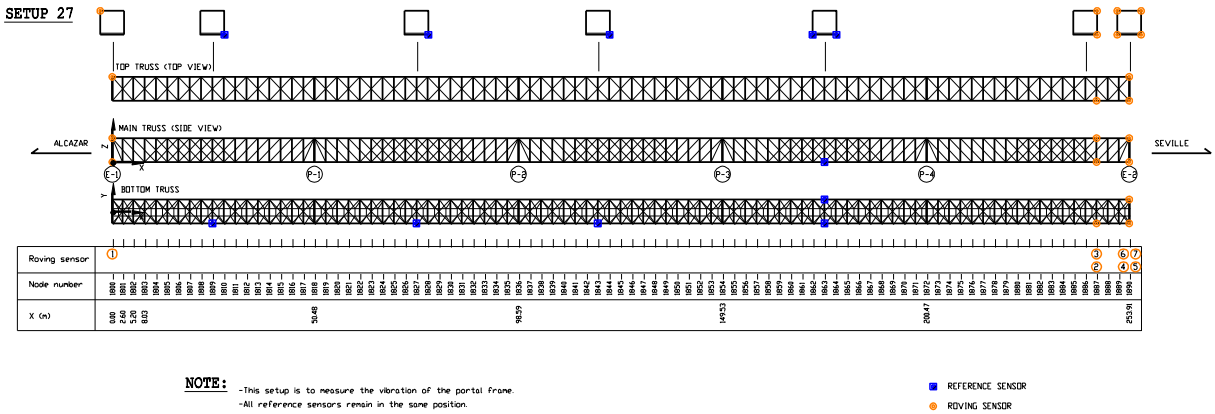



Fig. 57. Arrangement of sensor for setup 27

Fig. 58. Placement of sensor units at the top and bottom nodal joints of the bridge

### *5.8.3 System identification*

Data acquired from all setups are treated in a systematic manner. A pure ambient measurement period of about fifteen minutes is extracted. The time histories of the data are processed by first removing all offsets. Then high pass filters are applied with a cutoff frequency of 0.1 Hz. It is used because there are slow "drifts" in the data that suggest a low frequency disturbance (blurring) or noise. Next, zero-phase digital filtering of the input data is employed in both the forward and reverse directions. Finally, all the time histories of the data are down-sampled to 33 Hz for system identification processing. Since vibrations in the longitudinal direction are hardly excited and longitudinal modal displacements associated with vertical modes are small, all the channels in that direction are removed from the data set to speed up computation and to reduce noise disturbance.

After pre-processing the data, a model of the structure is identified from the data in the system identification process using the toolbox MACEC 3.2 developed by the Structural Mechanics Division, K.U.Leuven. The reference-based implementation of the covariance-driven stochastic subspace identification method (SSI-cov) is used for dynamic system identification. By using this, it is possible to identify the uncertainty bounds in the identification results. Reference signals are the pre-processed time histories from the five reference sensors plus one roving sensor, mostly in the middle region of the fifth span. In short, there are twelve reference signals out of twenty-four. Half the number of block rows is 120 and the considered model order range is from 4 to 200 in steps of 4.

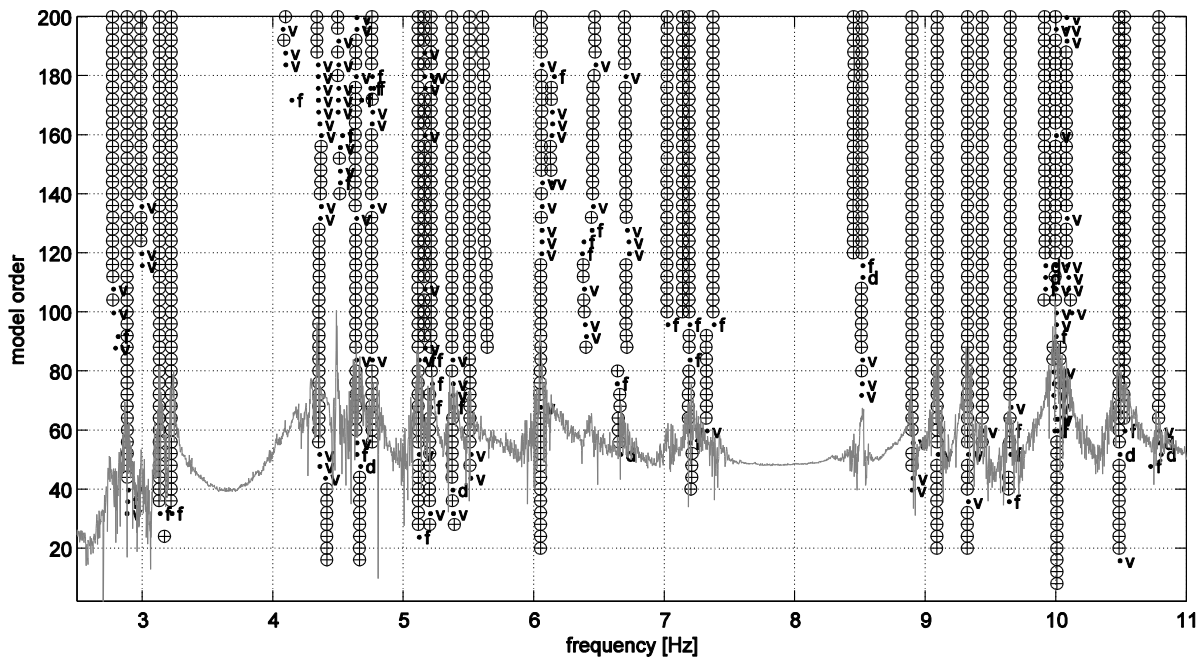



Fig. 59. The stabilization diagram in the frequency interval between 2.5 and 11 Hz

Fig. 59 shows the stabilization diagram in one of the setups (setup number one). The power spectral density (PSD) of the signals is also superimposed on the diagram. The diagram is very clear with many stable poles. The poles that come along in vertical lines are candidates for picking modes in the later modal analysis step. It can also be seen that stable poles systematically appear in certain frequency intervals, i.e. from 2.5 to 3.5 Hz; 4 to 7.5 Hz and 8 to 11 Hz.

### *5.8.4 Modal analysis*

The first twenty-three modes have been initially identified within a frequency range from 2.78 Hz to 9.65 Hz. The identified modal parameters are given in Table 8. The modal phase collinearity (MPC) value [0;1] indicates the realness of a mode. It should be close to 1 for a real mode. The mean phase (MP) value [0;90] should be close to 0 degree for a real mode. The MPD value is the deviation of the mean phase. As seen from the table, the quality of the identified modes is quite good (except for mode 3). These modes are almost real with low damping ratio (xi), high modal phase collinearity, low mean phase and low mean phase deviation. The lower quality of mode 3 is expected as it is very close to mode 2 and it might also be not well excited.

The first frequency range (2.78 to 3.37 Hz) contains transverse modes. Most of the modes found in the second frequency range (4.34 to 6.70 Hz) are vertical modes, except for mode 15 [5.51 Hz, 0.24%]. From mode 26 onward, all modes are torsional modes (of increasing mode shape order) with a frequency range from 8.44 to 9.65 Hz. The first torsional mode is found at a frequency of 8.90 Hz (mode 26). The frequencies of the identified modes are in general smaller than those extracted from the FE model.

Table 8. Summary of the identified modes (sym. – symmetric; asym. – asymmetric)

| Mode No. | f [Hz] | xi [%] | MPC [-] | MP [°] | MPD [°] | Mode type | Note |
|---|---|---|---|---|---|---|---|
| 1 | 2.78 | 0.72 | 0.975 | -2.77 | 4.74 | Transverse | Not found in FE model |
| 2 | 2.90 | 0.67 | 0.982 | -0.64 | 4.82 | Transverse | FE mode 1, 3.06 Hz |
| 3 | 2.98 | 0.57 | 0.796 | -0.05 | 16.37 | Transverse | FE mode 2, 3.09 Hz |
| 4 | 3.13 | 0.70 | 0.957 | -5.45 | 5.86 | Transverse | Not found in FE model |
| 5 | 3.24 | 0.47 | 0.988 | -0.59 | 3.35 | Transverse | FE mode 5, 3.92 Hz |
| 6 | 3.37 | 0.55 | 0.926 | -3.63 | 10.16 | Transverse | |
| 7 | 4.34 | 0.31 | 0.979 | -1.25 | 3.67 | Vertical, sym. | FE mode 6, 4.49 Hz |
| 9 | 4.48 | 0.47 | 0.974 | -6.44 | 4.22 | Vertical | FE mode 7, 5.10 Hz |
| 10 | 4.64 | 0.63 | 0.918 | -11.26 | 8.40 | Vertical | FE mode 8, 5.25 Hz |
| 12 | 5.16 | 0.38 | 0.956 | -0.50 | 8.87 | Vertical | FE mode 9, 5.47 Hz |
| 13 | 5.22 | 0.38 | 0.977 | -1.11 | 6.23 | Vertical | FE mode 10, 5.53 Hz |
| 14 | 5.37 | 0.71 | 0.971 | -0.81 | 4.80 | Vertical, sym. | FE mode 11, 5.88 Hz |
| 15 | 5.51 | 0.24 | 0.930 | -2.72 | 8.69 | Combination | Not found in FE model |
| 16 | 5.59 | 0.88 | 0.893 | -0.87 | 9.82 | Combination | FE mode 13, 6.03 Hz |
| 18 | 6.05 | 0.46 | 0.964 | -2.45 | 5.72 | Vertical, asym. | FE mode 16, 6.86 Hz |
| 20 | 6.70 | 1.06 | 0.950 | -4.87 | 7.03 | Vertical, sym. | FE mode 17, 7.06 Hz |
| 24 | 8.44 | 0.70 | 0.941 | 5.56 | 7.30 | Combination | |
| 25 | 8.52 | 0.30 | 0.940 | -13.15 | 7.22 | Combination | |
| 26 | 8.90 | 0.19 | 0.947 | -1.06 | 5.72 | Torsional | |
| 27 | 9.11 | 0.24 | 0.960 | -4.32 | 5.19 | Torsional | |
| 28 | 9.33 | 0.26 | 0.940 | -1.50 | 6.57 | Torsional | |
| 29 | 9.44 | 0.29 | 0.955 | -3.21 | 6.23 | Torsional | |
| 30 | 9.65 | 0.32 | 0.952 | 5.23 | 6.57 | Torsional | |

Fig. 60 to Fig. 63 are selected identified mode shapes of the bridge. In each figure, the upper graph is the isometric view of the mode shape displacement, and the lower graph(s) are the top view or side view (or both) depending on the mode type. All mode shapes are normalized to unity. Meanwhile, in some of those figures (for combination modes and torsional modes of the bridge), the isometric plot is the modal displacement of the truss' bottom "plane" for better visualization.

In setup 27, four sensors were put on the top chords, which is not enough to plot the mode shape of the upper bracing truss. Nevertheless, this setup is very helpful in identifying the modal displacements of the portal frames. Thanks to this, some of the measured modes could be related to known modes from the finite element model (Fig. 61).

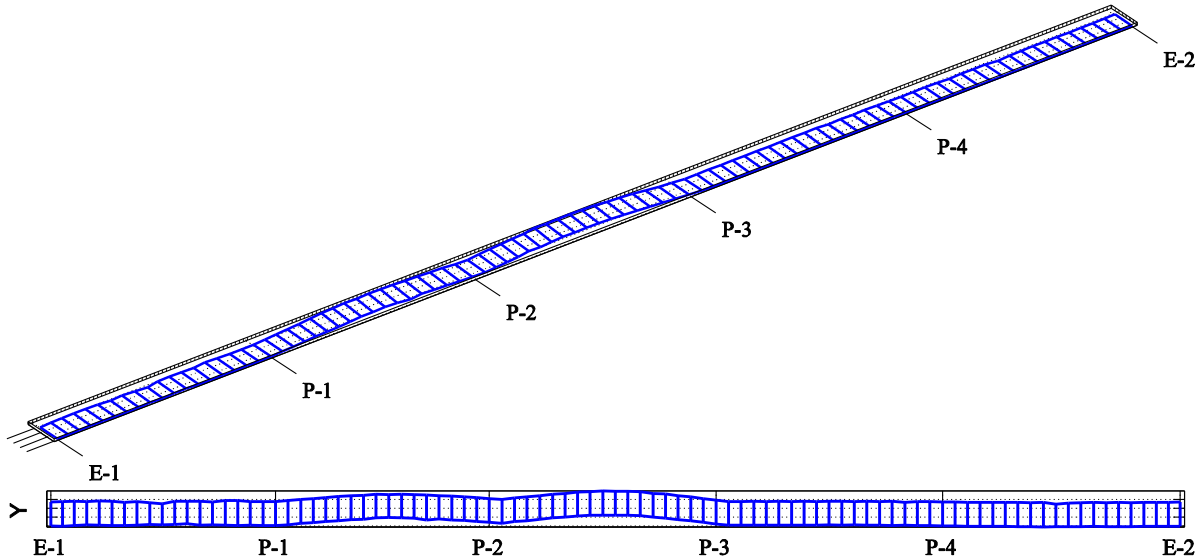



Fig. 60. Identified mode 1 – transverse mode [2.78 Hz, 0.72%]

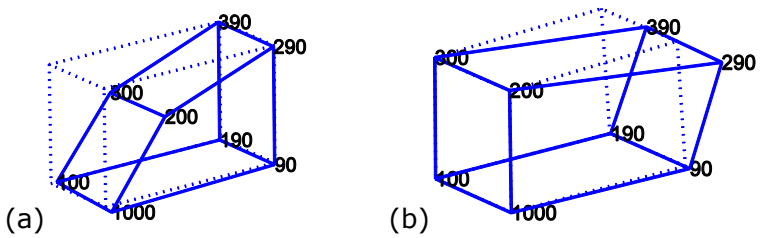



Fig. 61. The modal displacement of the two end portal frames in mode 2 (a) and mode 3 (b)

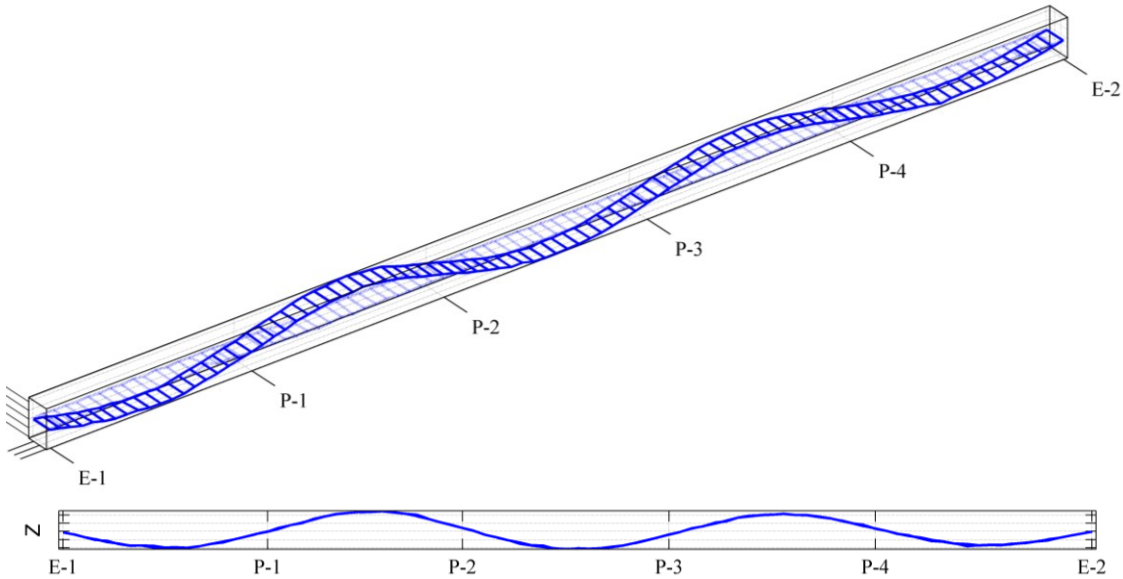



Fig. 62. Identified mode 7 – vertical mode [4.34 Hz, 0.31%]

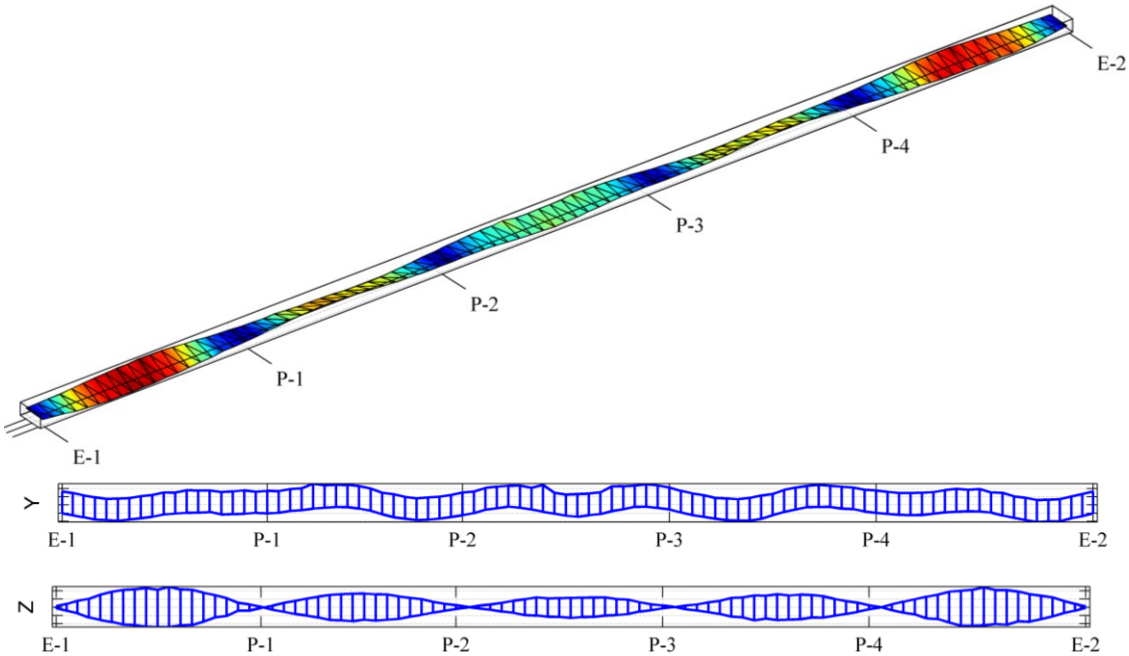



Fig. 63. Identified mode 27 – torsional [9.11 Hz, 0.24%]

### *5.8.5 Conclusions*

From the ambient vibration test, in total, 23 natural modes have been identified from all twenty-seven measurement setups. Many modes are well connected to the calculated modes from the finite element analysis. Some measured modes are not found in the finite element model, however. The quality of the identified modes is quite good, the modes are almost real with low damping ratio. Some of the closely spaced modes could be very well separated.

The ambient vibration test of the Guadalquivir river bridge is one of the most comprehensive measurement campaigns on a civil engineering structure so far with 588 degrees-of-freedom covered. This could only be done with wireless sensor instrumentation. Otherwise, it would have taken a very long time with a conventional wired system.

## *5.9 FSM Suspension Bridge*

### *5.9.1 Quick facts*

The Fatih Sultan Mehmet (FSM) Suspension Bridge is one of the two suspension bridges connecting the European and Asian continents on the Bosporus strait and also known as Second Bosphorus Bridge. It is located on the route of Trans-European Motorway (TEM) which is between Edirne and Ankara. The FSM bridge is considered as one of the vital links not only for the trans-national transportation but also for the metropolitan city transportation since it is located on one of the two-peripheral motorways crossing the city from west end to east. The bridge's structural behavior and dynamic characteristics have been studied by several researchers in recent years. The permanent monitoring instrumentation has been installed on the bridge by the Highways General Directorate (KGM) and KOERI after the 1999 earthquakes and in 2011 the monitoring system has been upgraded by KOERI and real-time moritoring works are conducted. In June 2008, an ambient vibration test has been carried out on the bridge with Wireless Sensing Units and the results of this test comparing with the analytical results have been studied in this report.
This report refers to studies by Apaydin, 2002; Dumanoglu et al. 1992; Katsikaridis, 2011; Kaya, 2009; Picozzi et al. 2009.

### *5.9.2 Bridge description*

From the structural point of view, the FSM bridge is a gravity-anchored suspension bridge with no side spans, steel pylons and double vertical hangers. The aerodynamic deck is hanging on double vertical steel cables. It is 1,510 m long (210m + 1.090m + 210m) with a deck width of 39 m. The distance between the towers (main span) is 1,090 m and their height over road level is 105 m. The clearance of the bridge from sea level is 64 m. Fatih Sultan Mehmet Bridge had the 6th longest suspension bridge span in the world when it was completed in 1988. At present, it has the 11th longest suspension bridge span in the world. Owner of the FSM Bridge is the Ministry of Public Works and Settlement, Turkey. In Fig. 64 the location of the FSM bridge is shown.


Fig. 64. Location map of the Fatih Sultan Mehmet (FSM) Bridge, Istanbul

The general structural characteristic of the bridge is shown in Fig. 65 and given in Table 9.

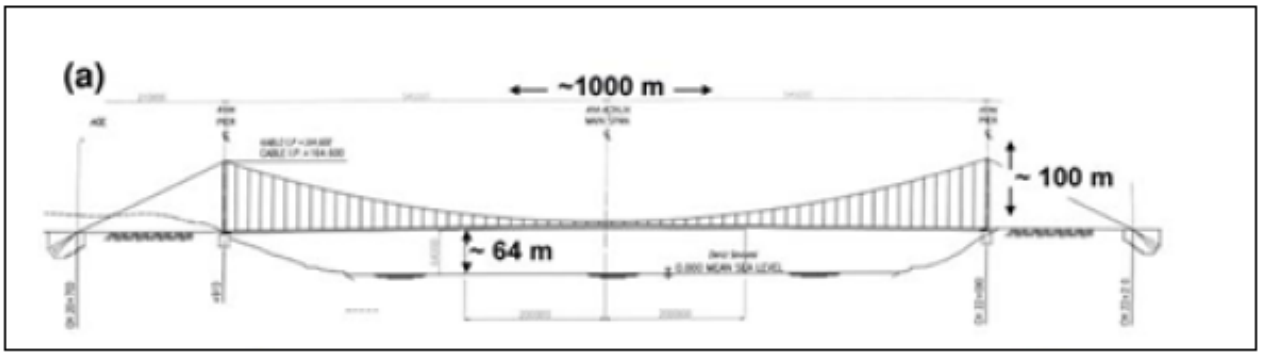



Fig. 65. The general characteristics of the FSM Suspension Bridge

Table 9. The structural characteristics of the FSM Suspension Bridge

| | |
|---|---|
| Span | Main span 1090 m |
| Deck width | 33.8 m (39.4 m total) |
| Substructure | 2 steel towers (107 m) |
| | 2 anchorages |
| | Spread foundation |
| Dead load per unit length | 216 kN/m |
| Area per cable | 0.365 $m^2$ |
| Gross x-sectional area | 1.26 $m^2$ |
| Deck moment of inertia | Ixx = 1.73 $m^4$ |
| | Iyy = 129.2 $m^4$ |
| | Torsional = 4.7 $m^4$ |
| Modulus of elasticity of cables | 205 $kN/mm^2$ |
| Sag | 91 m + 6.3 m |
| Hanger geometry | Vertical (double) |
| Clearance from the sea level | 64 m |

### *5.9.3 Objective*

The ambient vibration test has been carried out by new low-cost Wireless Sensing Units (WSU). The purpose of the test was to compensate the reduced sensitivity of these low-cost sensors by the deployment of the high-dense self-organizing networks in real-time data acquisition and analysis. Such characteristics make systems of WSU of great interest for the monitoring of strategic civil infrastructure.

### *5.9.4 Methodology of measurement*

On the 27th June 2008, 24 sensors were used to perform ambient vibration measurements at the bridge during the day-time under normal traffic conditions. The location of the sensors on the bridge is shown in Fig. 66. The acquisition scheme consisted of installation of 4 reference sensors placed outside of the bridge's deck, 8 sensors along each side, 2 sensors on the lowest part of the vertical cable at the midpoint of the bridge, and 4 sensors on top of the bridge's towers. In Fig. 67 some pictures from sensor installations on the bridge are shown. The test was performed for a few hours, and about 1-1/2 h of contemporary recordings at all sensors are available. The sampling rate was fixed to 100 Hz and at each node there were three channels in order to detect the bridge vibration on the vertical (V), the longitudinal (L) and the transversal (T) component.

Chanel 1 = Vertical component

Chanel 2 = Longitudinal component

Chanel 3 = Transversal component

The unit measurement of the accelerometers was m/sec2 and the data were stored in an ASCII format.

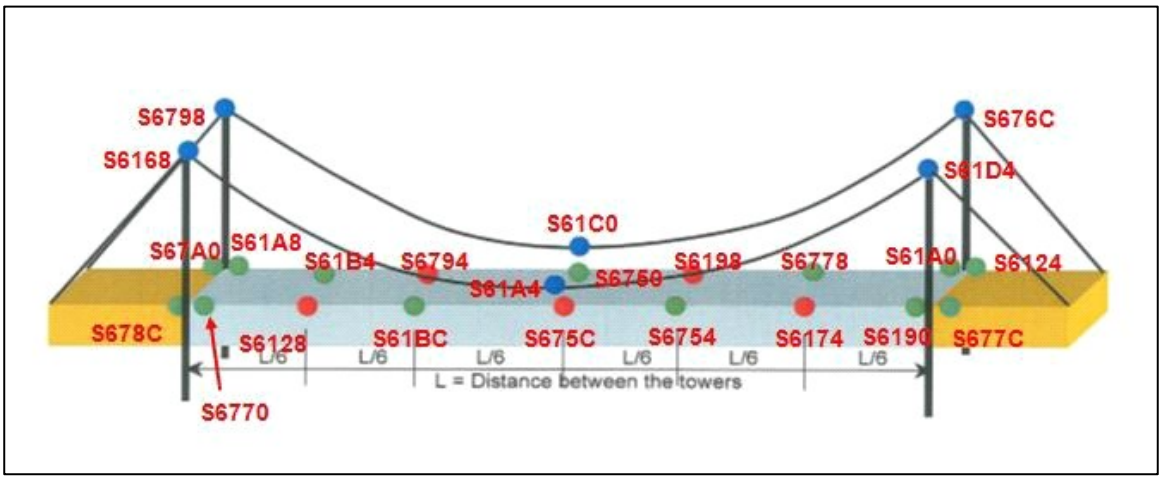



Fig. 66. Detailed view of the location of the sensors in FSM Bridge

Fig. 67. Types of sensor installation during the test measurements in June 2008

### *5.9.5 Data analysis*

The data recorded during the ambient vibration test has been analysed in a MATLAB-based code which is the part of the KOERI-MIDS (Kandilli Observatory and Earthquake Research Institute - Modal Identification Software) developed for real-time modal identification. The first 4 modes of the natural frequencies and the mode shapes of the bridge have been identified.

Fig. 68 to Fig. 70 show the time-history of the accelerations recorded in the first channel, for the vertical, longitudinal and the transversal direction.

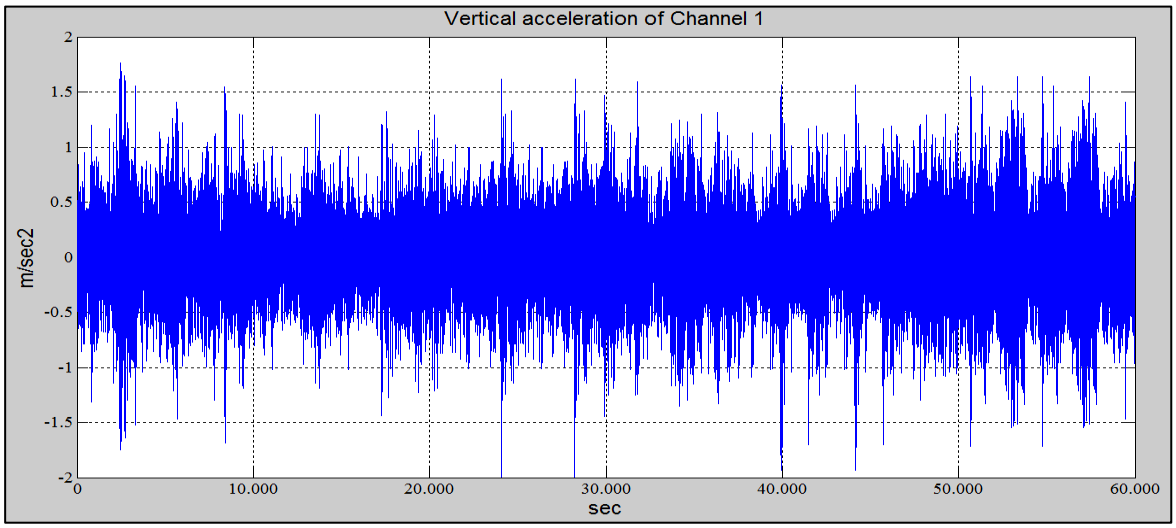



Fig. 68. Vertical acceleration recorded in node 1 (S6770)

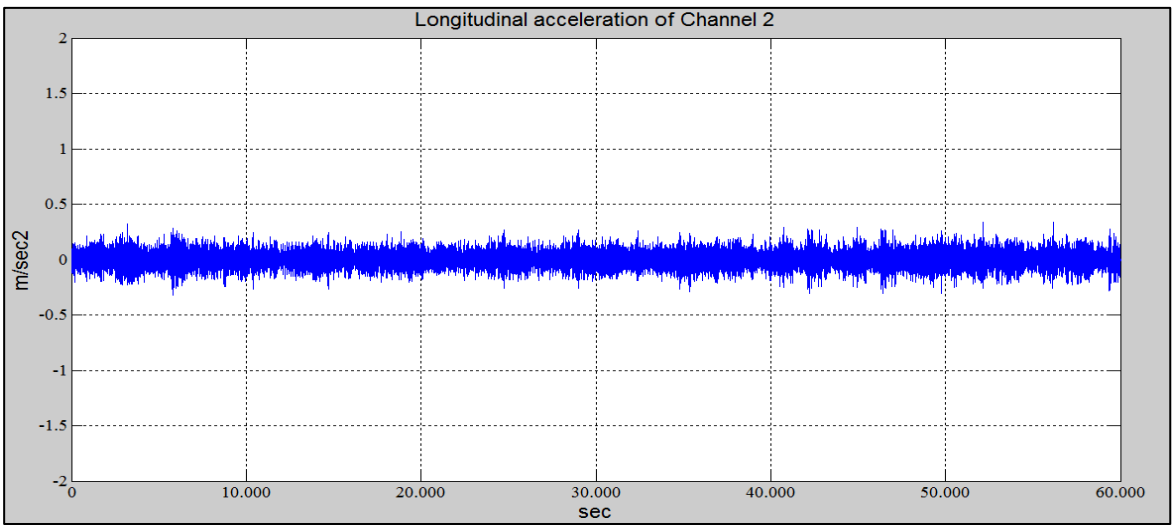



Fig. 69. Longitudinal acceleration recorded in node 1 (S6770)

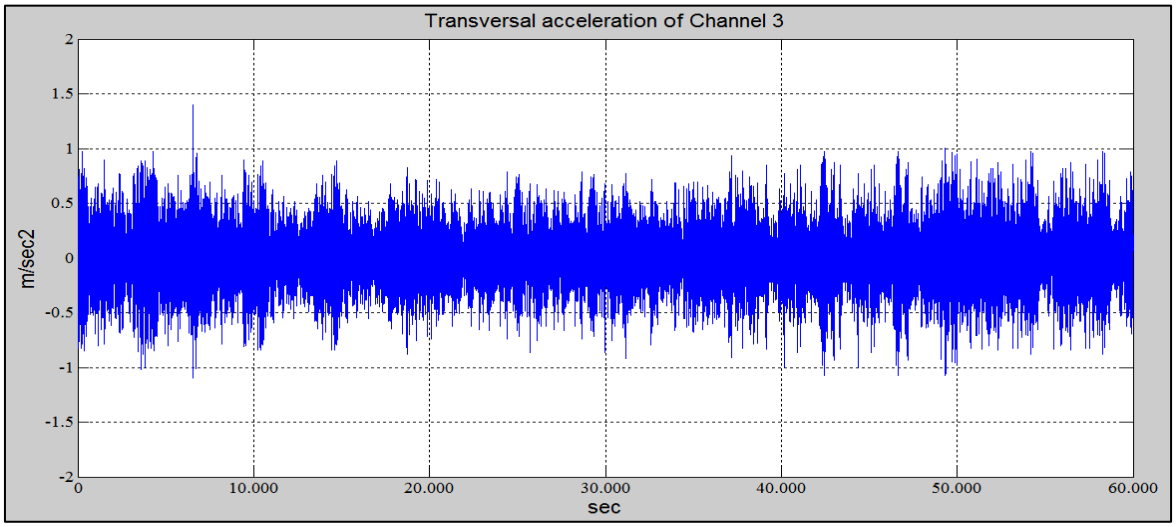



Fig. 70. Transversal acceleration recorded in node 1 (S6770)

In order to identify the modal frequencies of the bridge smoothed Fourier amplitude spectrums (SFAS) of all measuring stations are calculated. SFAS for each direction is estimated as an average of all the 14 stations which correspond to each direction as shown in Fig. 71 to Fig. 73. Since the collected data set refers to ambient vibration data, only a few modes of the structure is excited. Therefore, only the five fundamental modal frequencies for the vertical and the transversal direction are identified and the first three frequencies for the longitudinal. Two frequency bands are defined around the identified modal frequencies for the modal identification of the structure. The frequency band limits are defined at frequencies where the amplitude of the modal frequency attains its minimum.

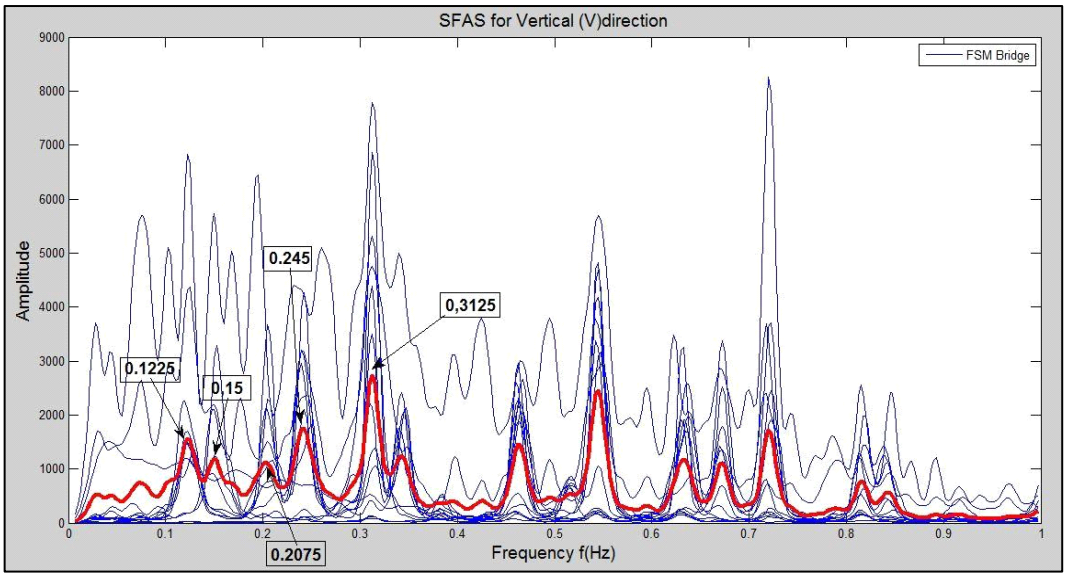



Fig. 71. SFAS for vertical component

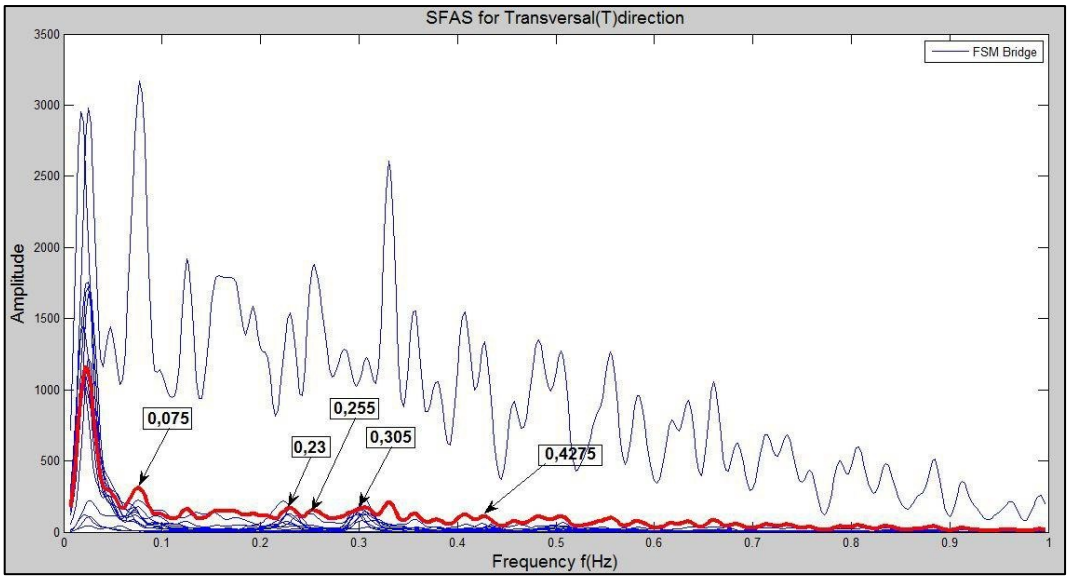



Fig. 72. SFAS for transversal component

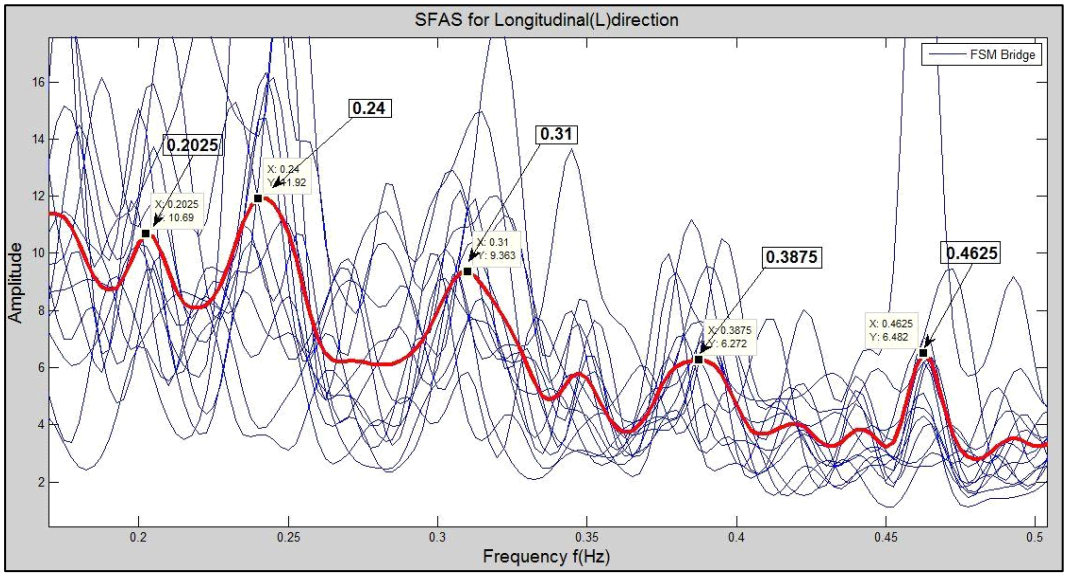



Fig. 73. SFAS for longitudinal component

The first five experimental SFAS peaks for the vertical, longitudinal and transversal directions are given in Table 10.

Table 10. First five experimental SFAS peaks for the vertical, longitudinal and transversal directions

| Mode Number | Vertical | Longitudinal | Transversal |
|---|---|---|---|
| 1st | 0.1225 | 0.2025 | 0.0775 |
| 2nd | 0.15 | 0.24 | 0.23 |
| 3rd | 0.2075 | 0.31 | 0.305 |
| 4th | 0.245 | 0.3875 | - |
| 5th | 0.3125 | 0.4625 | 0.427 |

The comparison of test results with the analytical results is shown in Table 11.

Table 11. Comparison of the ambient vibration test results with the analytical results

| | Vertical | | Longitudinal | | Transversal | |
|---|---|---|---|---|---|---|
| **Mode Number** | **Matlab Code** | **Brownjohn et all, 1992** | **Matlab Code** | **Picozzi et all, 2009** | **Matlab Code** | **Brownjohn et all, 1992** |
| **1st** | 0.1225 | 0.125 | 0.2025 | 0.204 | 0.0775 | 0.073 |
| **2nd** | 0.15 | 0.155 | 0.24 | 0.247 | 0.23 | 0.232 |
| **3rd** | 0.2075 | 0.208 | 0.31 | 0.311 | 0.25 | 0.288 |
| **4th** | 0.245 | 0.244 | 0.3875 | 0.387 | 0.305 | 0.303 |
| **5th** | 0.3125 | 0.317 | 0.4625 | 0.467 | 0.427 | 0.421 |

### *5.9.6 Outcomes of the Ambient Vibration Test Analysis*

The test showed that using Wireless Sensing Units is very practical and convenient for the temporary field monitoring due to its wireless communication, light weight and easy installation. The results given in Table 11 show that the ambient vibration test results coincide with the analytical results. Although a peak around 0.025Hz can be seen in transversal (Fig. 72) and longitudinal components (Fig. 73), this feature does not appear in the analytical results. It is considered as noise due to the installation of sensors on the bridge fences as shown in Fig. 74.

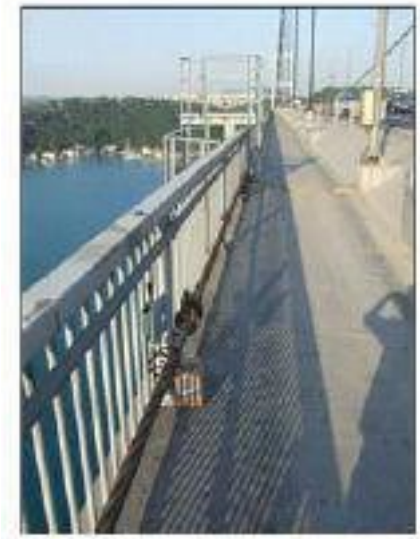


Fig. 74. Installation of the sensors in the deck of FSM Bridge

## *5.10 Residential Apartment Buildings*

### *5.10.1 Quick facts*

Ambient Vibration Tests (AVT) were carried out using Wireless Sensing Units (WSU) at two typical building structures (with and without infill walls) in Istanbul, Turkey. The dynamic characteristics of these structures have been studied experimentally and analytically in order to understand the effect of the infill walls. The structures were located at the same residential complex having four floors of RC-Moment Resisting Frame with the same structural plan. The first building comprised of one basement, the second one of three basements. Thus, the effects that non-structural elements and basements have on modal properties will be discussed too.

During the AVT, the structures were still under construction.

### *5.10.2 Description of the buildings*

The structural system and the geometry of the two buildings (Fig. 75) in this study are the same. The RC shear-walls were used in the perimeter and in the central core.

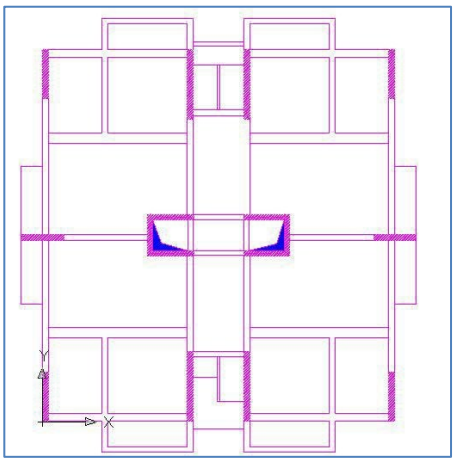



Fig. 75. View of the structural system of the buildings

The detailed information for each building is given as follows:

▪ *RC-frame building structure without infill-walls (Building 1)*

The first structure selected for the AVT experiment is a 4-storey residential building as shown in Fig. 76 which is a part of a residential complex in Istanbul, Turkey. The building was designed in 2010 according to Turkish Design Code 2007 with an RC-Moment Resisting Frame. As it can be seen from the structural plan of the building in Fig. 75, the structure is symmetric and mainly consists of shear walls at the core and RC columns and shear-walls around the perimeter. The total height is 14.20m and there is one basement. During the AVT, the building was under construction and no infill-walls were yet present (see Fig. 76).

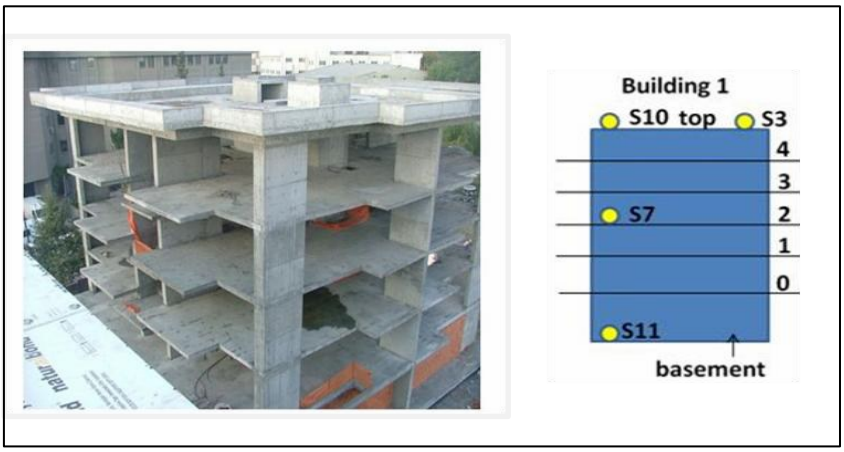



Fig. 76. Building 1 / Location of sensors

▪ *RC-frame building structure with infill-walls (Building 2)*

The second structure selected for the AVT is a 4-storey residential building (Fig. 77) which is part of the same residential complex as Building 1 (Istanbul, Turkey). The building was designed in 2010 according to Turkish Design Code 2007 with an RC-Moment Resisting Frame. As shown in the structural plan in Fig. 75, the building mainly consists of shear walls as a core and concrete columns around the perimeter just as in Building 1. The total height is 14.20m and three basements are present. During the AVT, the building was under construction, however, construction of the infill walls had already been completed.

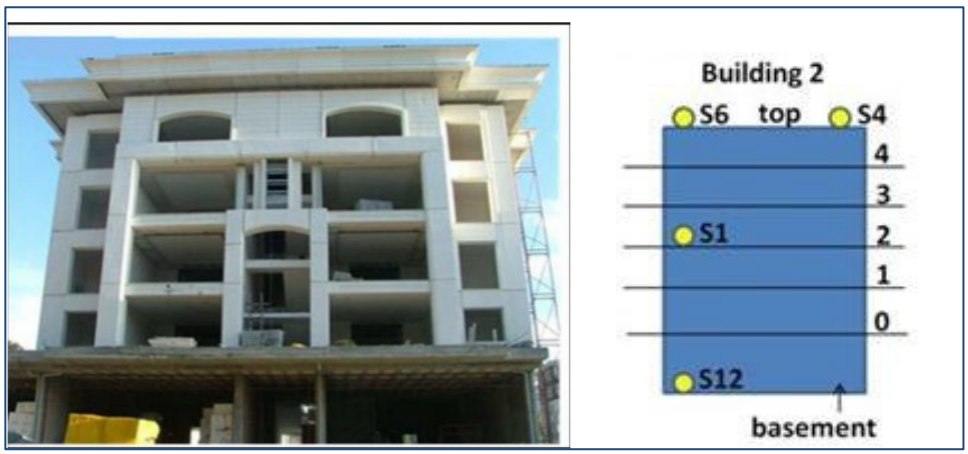



Fig. 77. Building 2 / Location of sensors

*5.10.3 Objective*

The purpose of the AVT was to compare the dynamic characteristics of the two buildings (with and without infill walls).

*5.10.4 Methodology of measurement*

The AVT was conducted on October 20, 2010 by a group of researchers from Bogazici University, KOERI, Istanbul, and GFZ Potsdam, Germany. The AVT set-up scheme (Fig. 78) consisted of installation of 4 sensors (velocity-meters) at the same levels in both buildings. 2 sensors were placed on the roof, 1 sensor on the second floor and 1 sensor on the basement of each building. All the sensors were installed following the same spatial convention, in order to detect the vibrations on the vertical (V), the north (N), and the east (E) components. Each sensor has been aligned with respect to the front of the building (north direction is relative). The test was performed between 8am and 1pm with a sampling rate of 100 Hz.

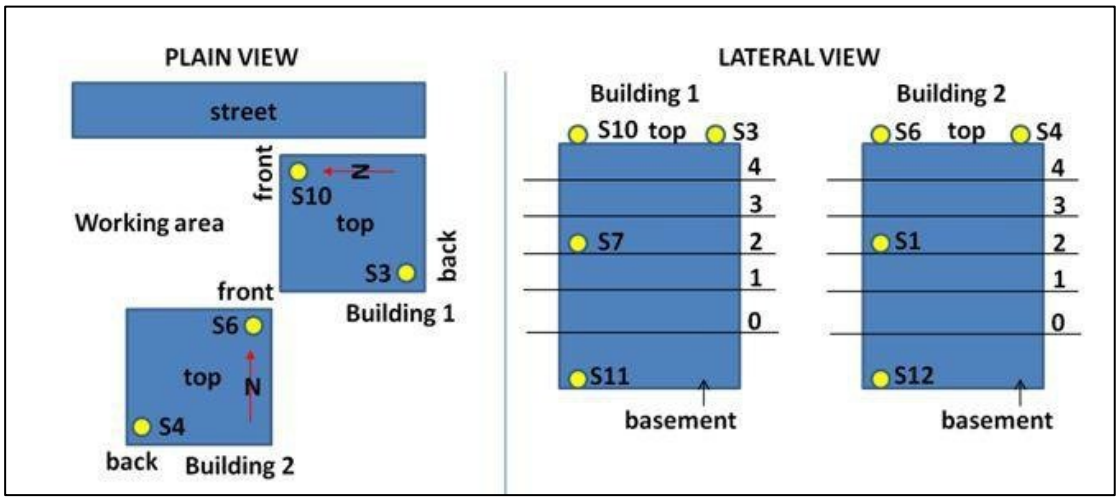



Fig. 78. Detailed view of the location of the sensors in the two buildings

Two sets of data have been recorded during the test.

*5.10.5 Data analysis*

Data recorded during the AVT have been analysed both in ARTEMIS and in a MATLAB-based code which is part of the KOERI-MIDS (Kandilli Observatory and Earthquake Research Institute - Modal Identification Software) developed for real-time modal identification. The AVT has had a total duration of 4 hours. Since the test was performed on a typical building structure, data recordings with a duration of 1 hour are considered to be enough for the modal identification of the structure. For this reason, two data sets with durations of 1 hour were created in order to check consistency in the results. A sample of the 1-hour data set from sensor 'S11' on Building 1 is shown in Fig. 79.

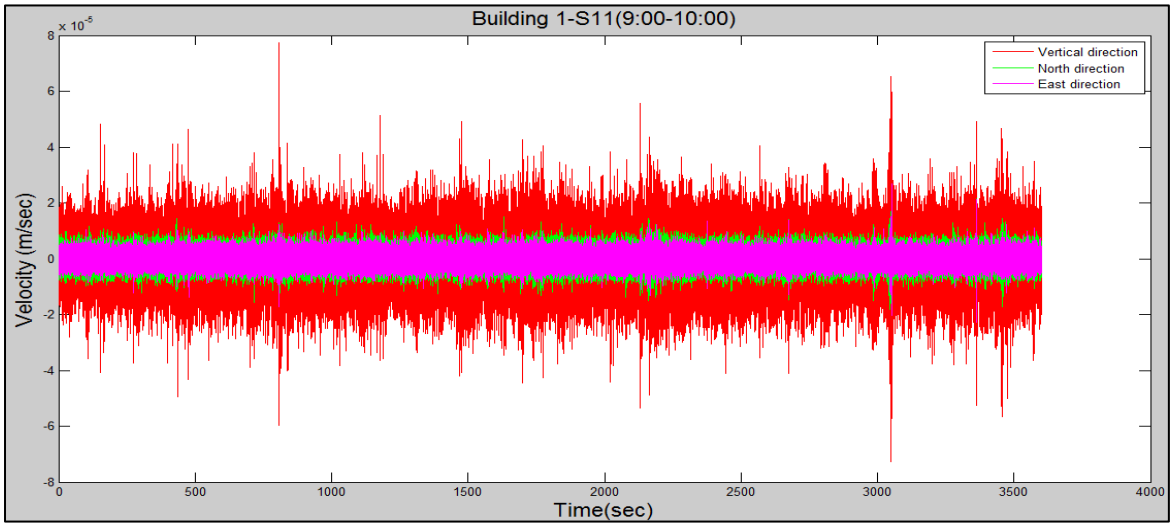



Fig. 79. Recorded velocity of Building 1, Station 11

Using the Matlab code, after segmenting the data recordings into running windows, each running window was baseline corrected and then band-pass filtered. The forth order butterworth band-pass filter has been used with corner frequencies of 0.01 Hz and 20 Hz. The Fourier Amplitude Spectrum (FAS) of each running window is smoothed in order to reduce the noise effect from FAS. The hamming type of smoothing window is used with an optimum length of ten. The optimum length was chosen by means of "trial and error"-method. After the FAS of each running window has been estimated and smoothed, the spectra were averaged to form the FAS of each station. Peaks of FAS are considered as the modal frequencies of the structure. A sample figure for the FAS & identified modes is shown in Fig. 80.

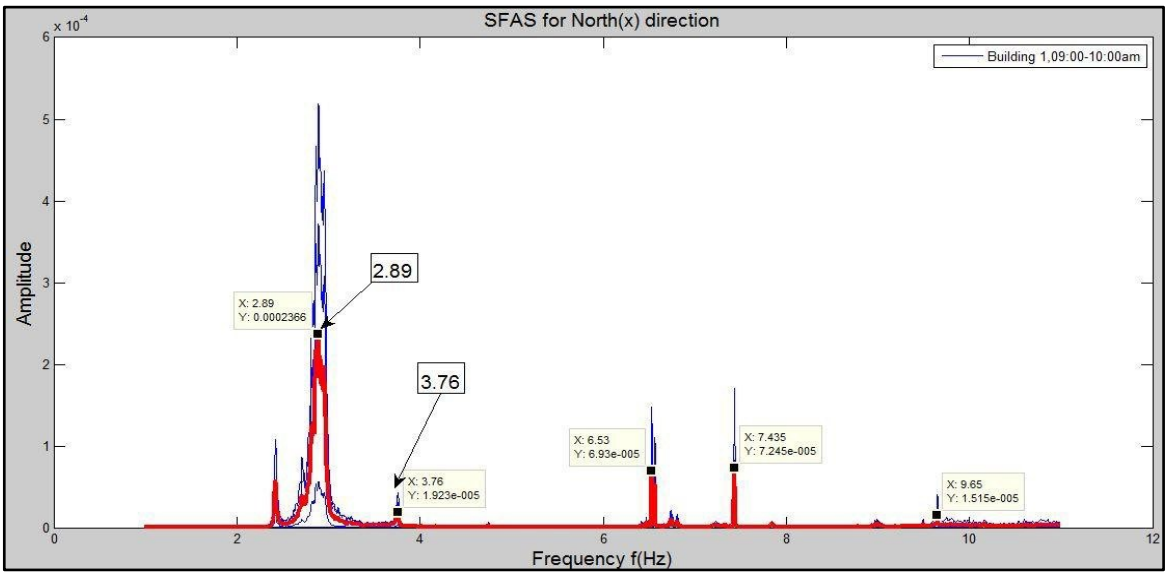



Fig. 80. FAS & identified modes for north(x) direction (Building 1, data set 1)

The animation of the first two modes identified for the north(x) direction is shown in Fig. 81 below.

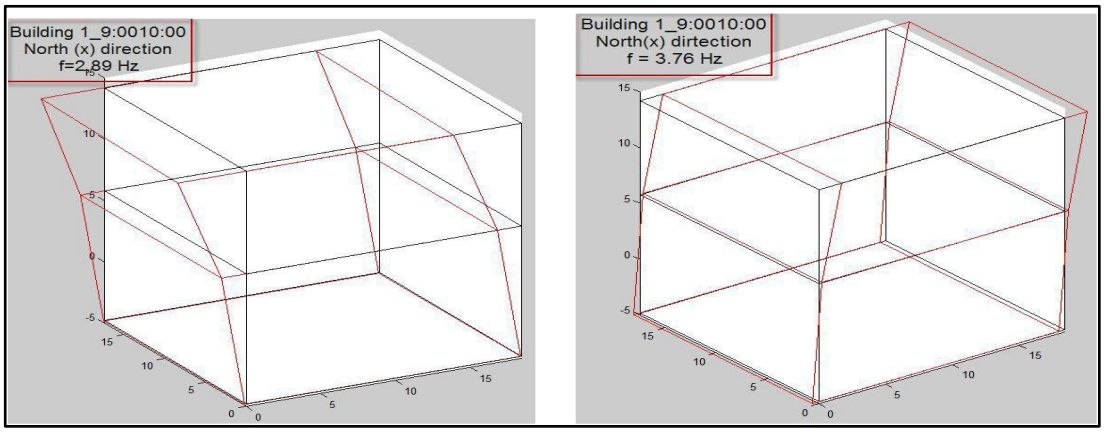



Fig. 81. Mode shapes in north(x) direction (first two modes), (Building 1, data set 1)

The identified modes in the data sets 1 and 2 for Building 1 are shown in Table 12 and Table 13 respectively:

Table 12. Identified modes in data set 1, Building 1

| Mode No | Data set 1 (09:00am-10:00am) |
|---|---|
| 1 | **2,716** (y) |
| 2 | **2,89** (x) |
| 3 | **3,76** (torsional) |
| 4 | **6,53** (z) |
| 5 | **7,433** (z) |
| 6 | **9,648** (y) |
| 7 | **10,16** (y) |

Table 13. Identified modes in data set 2, Building 1

| Mode No | Data set 2 (11:30am-12:30am) |
|---|---|
| 1 | **2,708** (y) |
| 2 | **2,888** (x) |
| 3 | **6,563** (z) |
| 4 | **7,425** (z) |
| 5 | **9,645** (y) |
| 6 | **10,15** (y) |

The two data sets show good consistency. Please notice that a torsional mode has been observed in data set 1.

A comparison of these two data sets is given in Table 14.

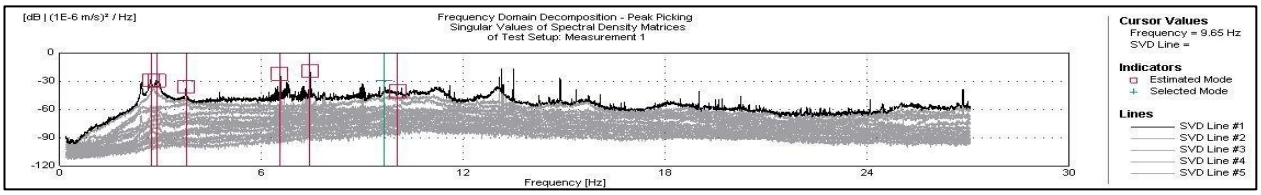
Table 14. Comparison between data sets, Building 1

| Mode No | 09:00-10:00am | 11:30-12:30pm | Difference (%) |
|---|---|---|---|
| **1** | **2,716** (y) | **2,708** (y) | *0,295* |
| **2** | **2,89** (x) | **2,888** (x) | *0,069* |
| **3** | **3,76** (tors) | - | - |
| **4** | **6,53** (z) | **6,563** (z) | *0,503* |
| **5** | **7,433** (z) | **7,425** (z) | *0,108* |
| **6** | **9,648** (y) | **9,645** (y) | *0,031* |
| **7** | **10,16** (y) | **10,15** (y) | *0,098* |

In ARTEMIS, Frequency Domain Decomposition (FDD) and Stochastic Subspace Identification (SSI) techniques have been applied to the two data sets for each building.

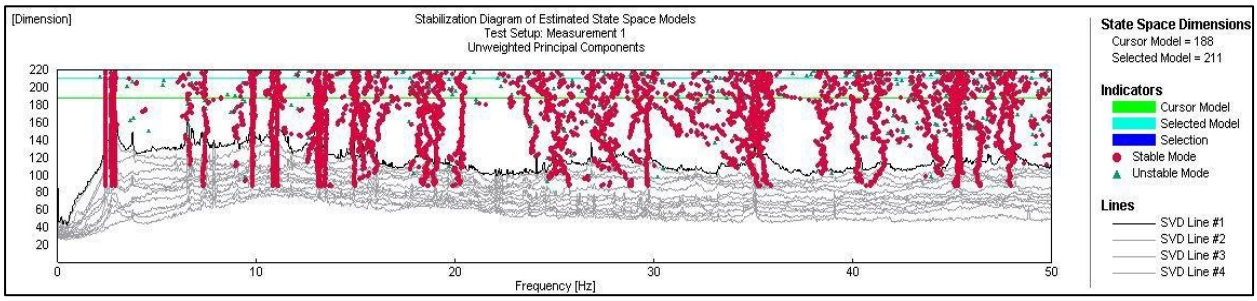
The response of the four-storey building was simulated using a lumped parameter system with twelve degrees of freedom. Only the floor slabs where the sensors were located were simulated in ARTEMIS. The slave node equations were used for definitions of rigid body motions and slave nodes. This is very helpful for the mode shape animation since the equations were used to make the complete geometry move.

The following Fig. 82 and Fig. 83 show the application of FDD and SSI to Building 1, data set 1.




| Mode | Frequency [Hz] | Std. Frequency [Hz] | Damping Ratio [%] | Std. Damping Ratio [%] | Comment | Creation Date & Time |
|---|---|---|---|---|---|---|
| FDD Mode 1 | 2.716 | 0 | 0 | 0 | Y-direction | 23-10-2011 11:58:06 |
| FDD Mode 2 | 2.89 | 0 | 0 | 0 | X-direction | 23-10-2011 11:55:04 |
| FDD Mode 3 | 3.76 | 0 | 0 | 0 | Rotational | 22-10-2011 11:46:47 |
| FDD Mode 4 | 6.531 | 0 | 0 | 0 | Z-direction | 23-10-2011 11:56:17 |
| FDD Mode 5 | 7.434 | 0 | 0 | 0 | Z-direction | 23-10-2011 11:56:19 |
| FDD Mode 6 | 9.65 | 0 | 0 | 0 | Y-direction | 23-10-2011 11:56:31 |
| FDD Mode 7 | 10.04 | 0 | 0 | 0 | Y-direction | 25-10-2011 18:20:31 |

Fig. 82. FDD peak picking technique (Building 1, data set 1)




| Mode | Frequency [Hz] | Std. Frequency [Hz] | Damping Ratio [%] | Std. Damping Ratio [%] | Comment | Creation Date & Time |
|---|---|---|---|---|---|---|
| SSI-UPC Mode 1 | 2.691 | 0 | 2.074 | 0 | Y-direction | 25-10-2011 17:10:18 |
| SSI-UPC Mode 2 | 2.957 | 0 | 1.875 | 0 | X-direction | 25-10-2011 17:10:23 |
| SSI-UPC Mode 3 | 3.608 | 0 | 1.232 | 0 | Rotational | 25-10-2011 17:10:26 |
| SSI-UPC Mode 4 | 6.422 | 0 | 3.737 | 0 | Z-direction | 25-10-2011 17:10:29 |
| SSI-UPC Mode 5 | 7.415 | 0 | 2.685 | 0 | Z-direction | 25-10-2011 17:10:31 |
| SSI-UPC Mode 6 | 9.835 | 0 | 2.913 | 0 | Y-direction | 25-10-2011 17:10:37 |
| SSI-UPC Mode 7 | 10.84 | 0 | 3.895 | 0 | Y-direction | 25-10-2011 17:10:47 |

Fig. 83. Stochastic Subspace Identification (SSI) technique, (Building 1, data set 1)

The comparison between FDD and SSI techniques based on Building 1/data set 1 is shown in Table 15.

Table 15. Comparison between FDD and SSI techniques, Building 1, data set 1

| Mode No | FDD | SS I | Difference (%) |
|---|---|---|---|
| **1** | **2,716**(y) | **2,691** (y) | ***0,929*** |
| **2** | **2,892** (x) | **2,957**(x) | ***2,248*** |
| **3** | **3,76**(tors.) | **3,608** (tors.) | ***4,213*** |
| **4** | **6,531** (z) | **6,422** (z) | ***1,697*** |
| **5** | **7,433** (z) | **7,415** (z) | ***0,243*** |
| **6** | **9,65** (y) | **9,835** (y) | ***1,881*** |
| **7** | **10,04** (y) | **10,39** | ***3,486*** |

Comparison of the results shows a good agreement between time domain and frequency domain methods. The small differences that exist in the results can be traced back to the different assumptions in FDD and SSI. In the case of SSI, since it is held in the time domain, we can avoid the disadvantages of frequency domain methods such as spectral leakage etc.

*5.10.6 Outcomes of the Ambient Vibration Test Analysis*

The AVT results for the two buildings are summerized in Table 16 and Table 17.

Table 16. Comparison of the 2 buildings, data set 1 (9:00am-10:00am)

| Mode No | Building 1 | Building 2 |
|---|---|---|
| 1 | **2,716** (y) | **2,82** (x) |
| 2 | | **2,958** (x) |
| 3 | **2,89**(x) | **3,17** (y) |
| 4 | **3,76**(tors) | **3,80** (tors.) |
| 5 | **6,53** (z) | **6,578** (z) |
| 6 | **7,433** (z) | **7,384** (z) |
| 7 | **9,648** (y) | **9,599** (x) |

Table 17. Comparison of the 2 buildings, data set 2 (11:30am-12:30pm)

| Mode No | Building 1 | Building 2 |
|---|---|---|
| 1 | **2,708** (y) | **2,82** (x) |
| 2 | | **2,958** (x) |
| 3 | **2,888** (x) | **3,17** (y) |
| 4 | **-** | **3,80** (tors.) |
| 5 | **6,563** (z) | **6,578** (z) |
| 6 | **7,425** (z) | **7,384** (z) |
| 7 | **9,645** (y) | **9,599** (x) |

As we can see from the results in both data sets, the presence of infill walls on the second building resulted in higher frequencies compared to the frequencies of the first building. Thus, the presence of the infill walls resulted in a more stiff structure. It is worth mentioning that the increase in the frequencies is less significant in the higher modes of the structures as the influence of these higher modes on the response of the structure is rather small. Furthermore, the modal properties for the first two modes were different. In Building 2 there are 2 closely spaced modes in x-direction, contrary to Building 1 where there are no such closely spaced modes in the respective direction. This observation was attributed to the presence of two additional basements in Building 2.

# 6 State of the Art

*Whang et al. 2004.* ***Integration of neesgrid into the NEES@UCLA field testing site****. Proceedings of 13th World Conference on Earthquake Engineering, Vancouver, Canada.*

For nees@UCLA mobile field laboratory a High Performance Mobile Network to support real-time telepresence through the NEESgrid architecture is developed. Within the mobile field laboratory for excitation eccentric mass shakers and linear inertial shakers are used. Measurements are carried out using above ground sensors, retrievable subsurface accelerometers. Data are collected via a wireless field data acquisition system and further transmitted by the high performance mobile network.

Building or bridge structural response/ performance studies. The equipment can be used to identify the modal responses of buildings (i.e. vibration periods, damping ratios, mode shapes), to evaluate the performance of non-structural elements within tested structures (i.e. HVAC, partition walls, equipment, etc.) and to evaluate the detailed response of structural components (e.g. beam-column connections, column-slab connections, etc.). Experiments can be performed at low levels of excitation from ambient vibration, micro-tremors, or over a range of excitation levels using the various shaker systems. For structures of small to modest size, the eccentric mass shakers can be utilized to excite structures into the nonlinear range. An important aspect of field testing is the ability to capture structural response and interactions without the shortcomings of scale and boundary conditions that commonly exist for laboratory testing. A unique feature of experiments performed using the *nees*@UCLA equipment relative to previous field testing programs is the potential for installation of dense instrumentation arrays that will provide more detailed insights into structural and non-structural performance characteristics. Potential applications might include forced-vibration studies of existing structures, slated for demolition, full-to moderate scale structures or sub-systems constructed specifically for testing, or use of the sensors and data acquisition system within structures during earthquake aftershocks.

Seismic health monitoring and sensor networks. A long-term vision for equipment use involves development of robust sensor networks for real-time seismic structural health monitoring by collaborating with other disciplines (e.g. computer science). Important issues to be addressed include: development of robust MEMS sensors, application of network time protocols in field sensor deployments, efficient transmission of data (e.g. multi-hopping or beam-forming), effective use of in-network processing, and development of efficient techniques for data management and interpretation.

Soil-foundation-structure interaction (SFSI) studies. The equipment can be used to apply forces and moments to foundation components, the response of which can be measured with acceleration and/or displacement sensors to evaluate SFSI effects. Load application to foundations is a natural consequence of vibration testing of buildings and bridges, so SFSI studies could be a component of any such experiment. Moreover, shakers can be directly installed on model foundations or simple structures mounted on model foundations to generate cyclic responses. Instrumentation would typically include an accelerometer array to record foundation motions and ground surface and below ground motions. Specific research objectives of such work could include the evaluation of frequency-dependent stiffness and damping terms for foundation systems, as well as foundation-soil-foundation interaction effects.

Response/ performance studies for geo-structures or soil deposits. As with buildings or bridge structures, geo-structures such as dams, embankments and retaining wall systems can be tested through forced vibration or seismic monitoring. Such studies would typically be performed to evaluate seismic response characteristics (i.e. vibration periods, damping ratios, topographic amplification effects). Excitation at amplitudes that could induce soil shear failure is expected to not generally be possible. Also internal response and deformations of geo-structures can be measured. Seismic monitoring of soil deposits is also possible, which might be of interest following a major earthquake, as data recorded from aftershocks could provide insight into wave propagation characteristics and soil pore water pressure generation.

*Aktan A.E., T. Kijewski-Correa, F.N. Catbas. 2011.* ***Structural Identification of Constructed Facilities. Approaches, Methods and Technologies for Effektive Practice of St-Id. A State-of-the-Art Report. ASCE SEI Committee on Structural Identification of Constructed Systems****. State of the Art report by ASCE & SEI.*

The original report consists of 281 pages. Within the framework of NERA Project a condensed overview is given within the following 21 pages.

***Chapter 1*** is on *Structural Identification of Constructed Systems*

Sub-headings:

1.1 Overview
1.2 Objectives
1.3 Historical development
1.4 Six steps for ST-ID of constructed systems
1.5 Outline of report

This report is written mainly for practicing engineers and is intended to provide an objective view of what sound applications of St-Id can and cannot offer. In this manner, this report aims to provide practicing engineers with the ability to decide whether St-Id may be appropriate in certain situations and to guide them as to what may be expected from such applications. It is emphasized that this report is not a "how to" manual or a "best practices" document, and was not assembled with the details or the intent to support actual applications.

Over the last decade, many have unsuccessfully attempted to apply St-Id approaches developed and proven in the case of manufactured systems directly to constructed systems. This is probably due to the fact, that St-Id approaches developed for manufactured systems implicitly ignore the many unique and confounding attributes of constructed systems (see Table 18).

The six steps for St-Id are:

Step 1 (par.1.4.1): *Objectives*, *Observations* and *Conceptualization*. Based on experience, there are several scenarios involving the construction, operation, maintenance or lifecycle asset management that may lead owners/ stewards to pursue a St-Id application, e.g.:

1. Load – capacity such as for different occupational conditions, wind and earthquake loadings for buildings or rating for inventory, operations or special permits for bridges
2. Design verification and construction quality control especially in case of challenging and/or ground-breaking new designs
3. A measurement-based delivery of a design-built contract in a performance-based framework
4. Documenting the as-is structural characteristics in order to serve as a baseline for assessing any future changes, due to aging and deterioration, following hazards, etc.
5. Evaluation of possible causes and development of mitigation strategies for deterioration, damage and/or types of performance deficiencies (e.g. vibrations, cracking, settlement, etc.
6. Designing structural modification, retrofit or hardening due to changes in use-modes, codes, aging, and/or for increasing system-reliability to more desirable levels.

Table 18. Uncertainties unique to constructed systems that influence their mechanical characteristics and performance (Aktan et al. 2011)

| | |
|---|---|
| Heterogeneity | Materials, member proportions, detailing, etc can vary considerably from member to member, and within a member. Deterioration and damage compounds these variations and makes discretization difficult and sometimes unmanageable without heuristics. |
| Boundaries | Constructed systems have unobservable soil-foundation interfaces that are often non-stationary in their contact properties. Soil and even rock properties change with pressure, moisture, temperature and time. |
| Continuity | Most constructed systems, and especially bridge systems are designed with movement systems and/or force releases. These systems are most often unobservable and behave differently under different levels of force and temperature. |
| Redundancy | Constructed systems have many types of local, regional and global/external redundancies. These redundancies are highly affected by temperature changes and temperature gradients (due to radiation), which results in intrinsic forces and changes in element properties. |
| Intrinsic Forces | Constructed systems maintain complex and non-stationary intrinsic forces due to dead weight, construction loads/staging, temperature effects, deterioration, damage, overloads, etc. These intrinsic forces are nearly impossible to measure in an absolute sense and their changes in many cases overwhelm the forces due to transient live loads. |
| Types of Nonlinearity | Element, connection and global behavior of real constructed systems exhibit many different types of nonlinearity that change at different limit-states. Cracking, material yielding, local instability, connection slip, interface friction, etc. are all associated with hardening/softening type behaviors. |
| Non-Stationarity | Constructed systems are non-stationary due to the non-stationary nature of their environment (e.g., temperature, radiation, etc) as well as their various loading-level and loading-type related nonlinearities. Temperature and humidity effects are highly complex: changes and rate of changes in ambient, regional and local temperatures and humidity of the structure and the soil may lead to intrinsic forces and also induce changes in boundary and continuity conditions. |
| Uniqueness | Nearly all constructed systems are custom-designed for specific applications and their mechanical characteristics are strongly affected by events during and immediately following their construction. While types of constructed systems may be grouped based on their primary structural system, size, materials, etc., applying results from a single structure to a larger population of structures is challenging due to their inherent uniqueness. |
| Geometric, Temporal Scale, Cost, Lifecycle | Constructed systems such as major highway bridges or combinations of several bridges and tunnels within regional transportation networks may be longer than several miles, cost several billions of dollars, and be expected to remain in service for over 100 years. The size and lifecycle impedes our ability to view such systems in a holistic manner over a sufficient span along their lifecycles and further compounds the natural variability and uncertainty in their mechanical characteristics. |
| Coupling | Most constructed systems feature coupling between sub-systems such as frames and walls, water, soil and foundation, substructure and superstructure, structural and nonstructural, etc. The coupling between sub-systems maybe highly complex, often nonlinear and nonstationary. Yet coupling may control how forces and displacements are transferred and the responses. |

Step 2 (par 1.4.2): *Measurement*, *Visualization* and *A-Priori Modeling*. Depending on the objectives of St-Id, investment into close range photogrammetry transformed into 3D CAD, and 3D imaging by laser-scanning is advised. In any case, an error-mitigation strategy for any errors in existing documentation and for inevitable human errors in building an a priori model from existing documentation is necessary before a priori modeling.

In some cases, idealized mechanical mass-spring models may be sufficient for a-priori modeling, whereas in other cases high resolution geometric-replica FE models may be justified.

Step 3 (par 1.4.3): *Controlled Experimentation*. In the past, the only linkage to actual constructed systems involved visual inspection, testing concrete cores and steel coupons retrieved from a structure, or, under unusual circumstances, straingaging. However, at the present time civil engineers have many dozens of options for measuring strain, linear distortion, normalized or relative deformation, tilt, velocity and acceleration among many other measurands including mechanical-thermal and electrochemical phenomena at the microscopic scales. Many additional options for nondestructive testing and evaluation (NDE) have become available which permit imaging and identification of discontinuities and faults within members and connections.

Step 4 (par 1.4.4): *Data Processing* and *Feature Extraction*. Data processing activities aim to make the acquired data more appropriate for interpretation. This is typically achieved through cleansing the data of blatant and subtle errors (e.g. spikes, malfunctioning sensors, statistical characterization), improving the quality of the data (e.g. averaging, filtering, windowing, etc.), and then compressing and/or transforming the data to better support interpretation (e.g. the extraction of modal parameters/flexibility, influence coefficients, etc.).
The second stage of Step 4, direct data interpretation, is optional. It involves fitting mathematical models (also referred as a non-physics-based model), such as Artificial Neural Networks, Auto-Regressive Models, state space models etc., to the processed data. These models are not formulated with any consideration of the underlying physics of the constructed system, rather they aim to accurately capture and replicate the patterns associated with the data. In this manner, they are most concerned about identifying when the constructed system behavior has changed rather than identifying the underlying cause of the change. This approach has advantages of requiring minimal user interaction and being able to address large data sets, and is a powerful tool for continuous monitoring of structures.

Step 5 (par 1.4.5): *Selection* and *Calibration of Physics-Based Models*. These models are formulated to explicitly recognize the underlying physics of the constructed system. It is recommended that several different modeling strategies are employed and compared to ensure the model selected for calibration is appropriate. The model calibration process typically involves optimizing a set of model parameters to minimize the difference between initial model and experimental results. Approaches to this model calibration (also known as model updating) can be classified based on how they select the parameters to identify, the formulation of their objective functions (to minimize), the optimization approach they employ (e.g. gradient-based or non-gradient based), and whether or not they explicitly address uncertainties.

Step 6 (par 1.4.6): *Utilization* of *Models for Decision-Making*.

<u>Outline of Report (par 1.5):</u> The state-of-the-art related to each of the six steps is discussed in depth by experts in the corresponding disciplines and application areas, and future research and application needs are to be identified.
In addition, this report offers summaries of selected examples of St-Id applications to buildings and bridges. Whether the calibrated models were actually utilized for decision-

making and the scenarios and analytical approaches that were employed for this purpose are specifically discussed. The societal system (policy, planning, financing, legal, etc.) that impact the utilization of St-Id in practice are also identified in each case study along with possible strategies for overcoming current barriers.

**Chapter 2** is on *A-Priori Modeling*.

Sub-headings:

- 2.1 Introduction
- 2.2 Classification of a priori models
- 2.3 Common model types
  - 2.3.1 Phenomenological models
  - 2.3.2 Structural models
  - 2.3.3 Finite element models
- 2.4 Modeling constraints unique to St-Id
- 2.5 Construction of a priori models through 3D CAD
- 2.6 Quality control requirements
  - 2.6.1 Modeling errors
  - 2.6.2 Input errors
  - 2.6.3 Analysis errors
- 2.7 Closing remarks

Most a priori models are based on assumptions of linearity and stationarity. In general these assumptions need not be made; however, in the absence of response data from the specific constructed system, it is difficult to justify the complications associated with nonlinear constitutive relations or stochastic finite element analysis. Starting with simpler, greatly idealized phenomenological geometric models to help conceptualize a constructed system, together with the site, soil and foundations, and then gradually increasing the detail and complexity of the model as the system is better understood are recommended. The utility of the a priori physically based models lies in its ability to identify key mechanism and provide an expected range of response to allow an efficient and robust experimental program to be designed and carried out.

Phenomenological models have the lowest geometric resolution and typically consist of a few elements to describe or investigate the key response mechanisms of constructed systems. These models mostly employ simple one-dimensional (e.g. plane or space frame elements) and discrete elements (e.g. translational or rotational springs, point masses).

Structural models typically employ both one-dimensional (plane or space frame elements) and two-dimensional elements (e.g. plate or shell elements). In an effort to remain consistent with the three dimensional geometry of the structure, various link elements, constraints, and rigid offsets are included.

Finite element models have the finest geometric resolution and employ the full range of elements available including three-dimensional solids. The primary advantage of this class of models is their ability to simulate the response and effect of complex structural details, connections, stress concentration, etc.

Modeling constraints unique to St-Id are discussed in par. 2.4. There are two unique constraints that must be satisfied for cases where the a priori model will be used for updating. The first constraint is that the a priori model must be constructed using an analysis package that either incorporates or can interface with updating software if a "formal" model updating is desired. It is possible to perform a heuristic-based, manual model updating without this constraint; however, this approach can be tedious and is greatly limited. In general, two approaches have been used to satisfy this constraint. First, the use of an analysis package that directly incorporates programming capabilities or the use of third party updating software such as FEMTools or Dakota has been employed. In some cases, however, these may be of limited usefulness as such analysis packages may not contain the elements or post-processing capabilities that the analyst desires. The second approach is to export the

simulation model to general analysis software such as Matlab, which can then be used to analyze and update the model. The disadvantages of this approach are related to the time and effort required to write the code to solve the model and write an output file that can be read by post-processing software. The second unique constraint involves the requirement that the most uncertain aspects of the model need to be parameterized in such a way that they can be included in the updating process (degree of composite actions, contribution of non-structural elements, etc.).

Quality Control Requirements for finite element analysis are discussed in par. 2.6. The first category of errors is *modeling errors*, especially conceptual errors and incompleteness (par. 2.6.1), which are the most difficult to detect, especially in the case of large and complex structures. In this regard, the most common errors arise due to over-idealization of the geometry, boundary and intermediate support conditions, connection stiffnesses, member releases, interfaces between various structural sub-systems, movement systems, etc. The general rule is to carefully evaluate the relative contributions of all possible displacement and deformation mechanisms and incorporate these in the model unless it is justifiable to ignore some. A common example relates to the importance of axial, shear and torsional deformations and any effects of geometric nonlinearity on local and global response, especially when creep and temperature effects are considered. In some cases, the friction in intended movement mechanism may lead to a locking of these mechanisms.

The second category of errors is *input errors* (par. 2.6.2). This is an important issue as the size of a model gets larger. Generation of a 3D CAD model that is then scrupulously and systematically checked for geometric accuracy of all local and global details against 2D fabrication and construction drawings and up-to-date photographs, which may then be directly transferred to a computer model, is a good measure for quality control. But even in this case errors can occur. A case is reported, where a model was constructed by transferring a 3D CAD model directly as an input file to an analysis program, but various element data categories were subsequently discovered to contain errors that were not discovered by checking the output of static or eigenvalue analysis. However, these errors did lead to significant errors in some of the member forces and were discovered only after extensive checking of the input data through transferring the data files into spreadsheet format.

The third category of errors is *analysis errors* (par. 2.6.3), which would not be discovered unless analysis output is suspected. Until all of the global and local displacements, reactions, forces and stresses are verified for physical consistency and correlated against a sufficient amount of reliable experimental data including frequencies, mode shapes, displacements, rotations and strains, such errors should always be considered.

In some cases, even if there are no errors in the input data, the mathematical extrapolation formulations of the finite elements used and their geometry may lead to significant errors in the resulting displacements and stresses without affecting global equilibrium. An example is the "locking" phenomenon associated with certain finite elements resulting in a finite element model having an apparent stiffness that may be significantly higher than the structure being simulated. Another example relates to apparent nodal stresses that may greatly exceed the actual average stresses within an element due to numerical errors. Finally, even slight mechanisms of nonlinearity such as opening of previously formed cracks or slippages at connections in the actual structure may contribute to significant attenuations in peak stresses that may not be properly simulated in the analysis.

**Chapter 3** is on *Experimental Considerations*.

Sub-headings

- 3.1 Introduction
- 3.2 Classification of experiments based on input
  - 3.2.1 Static input
    - 3.2.1.1 Controllable (measurable and unmeasurable) static loads
    - 3.2.1.2 Uncontrollable (measurable and unmeasurable) static

Unfortunately, full-scale evaluation of in-service structures is typically limited to the low-level response regime of the structure because of difficulties exciting these structures in a controlled manner at higher levels necessary to validate performance at the limit states design. As in-service structural performance can be characterized in terms of both static and dynamic parameters, the experimental part of the St-Id process is likely to involve measurements of both dynamic and static signals. Note that while all signals vary in time, dynamic signals used for System Identification are defined as varying fast enough (about an equilibrium configuration that may itself vary more slowly in time) that inertia (mass) properties of the structure are engaged.

The design of an experimental program for characterizing and evaluating full-scale performance of civil infrastructure is governed by a large number of constraints including:

- Expectations from the experimental studies, requirements, constraints and specifications (whether commercial, applied research or pure research)
- Experiment duration (different strategies due to robustness, cost and manpower constraints are adopted according to the type of exercise which broadly fits in three categories)
- Structural attributes that will affect the quantity and types of instrumentation used
- Environmental and operational constraints
- Logistics and accessibility

The main components of a St-Id experiment are thus:

1. Selection of inputs, their locations and their means of measurement
2. Selection of outputs, their locations and their means of measurement
3. Gathering and transmission of signals to recorder/ logger
4. Conversion of signals to storable form, typically via analog-to-digital conversion for storage on computer disk digital streaming
5. Data storage, typically on local or remote computer, occasionally on a dedicated logger
6. Data inspection/ quality control by real-time local processing/ presentation
7. User interface and real-time alerting/ reporting (in the case of monitoring applications)

Un-measurable and uncontrollable inputs are termed "ambient" inputs, otherwise they amount to "noise" when controllable and measurable inputs are used (par. 3.2).

Either trucks or other load sources such as concrete blocks or water – containers may be used to load a bridge while critical responses are measured (par. 3.2.1.1)

European researchers in particular are now switching from *forced vibration testing (FVT)* to alternative procedures such as "free vibration" and *ambient vibration testing (AVT)*, but there remain specific circumstances when FVT is the only viable solution, such as floor vibration performance assessment requiring reliable experimental assessment of modal mass. Transfer functions or frequency response functions (FRFs) scale the input (forcing) to output (response) via either mass or stiffness so both can be identified using this type of test. Stiffness information with good signal-to-noise ratio can be recovered using modest forces due to resonant amplification, and controllable dynamic loads provide the possibility of studying nonlinearities. FVT may also be necessary when the available ambient excitation is unable to excite critical structural modes with a good enough signal-to-noise ratio for reliable measurement (par. 3.2.2.1).

*"Long-stroke" electro-dynamic shakers*, where stroke length allows the shaker force to be developed from low frequencies are now widely used in civil applications. While their peak output (450 N) is less than the multi – kN outputs of hydraulic shakers, they do not have the logistical disadvantage of requiring hydraulic power packs. While small shakers may not be capable of global structural excitation, they may be useful for local measurements, e.g. of stay cables in bridges or floor panels in a building (par. 3.2.2.1.3).

*Impulsive testing* is an attractive proposition for full-scale dynamic testing because the short duration of the excitation translates to a broad-band excitation (par. 3.2.2.1.4).

*Wind excitation* is typically the dominant loading on low frequency structures, i.e. with fundamental frequencies below 1 Hz, with diminishing effect at higher frequencies: this includes long span bridges and tall buildings. Alongwind loading is the nearest excitation source to the ideal stationary Gaussian white noise process, exhibiting excellent random character and relatively smooth spectra over a broad frequency range. There is also relatively weak correlation over the span of structure likely to be excited, hence in principle both symmetric and non-symmetric modes may be excited (par. 3.2.2.3.1).

Unlike wind, *dynamic excitation* due to highway vehicles is far from white noise, but is colored, with concentration of force around "body bounce" and "axle hop" frequency bands of approximately 2-5 Hz and 10 -15 Hz, so that it may not be possible to identify a peak in a Fourier spectrum uniquely as a bridge mode. Further, vehicle traffic is not usually a constant stream (i.e. it is non-stationary) and for small bridges, the vehicle dynamics can have a significant influence on modal properties, although cars have minimal effect (par 3.2.2.3.2).

Typically, wave motion has most of its energy content below 2 Hz (par. 3.2.2.3.3).

*Laser interferometry* can be used for measuring motion along line of sight at distances up to 500 m and down to frequencies of 0,1 Hz. Tracking lasers are available for applications to measure dynamic transverse motion at distances over 100 m. These systems require reflective marker, resolution is about 0,01% of range and frequency response is compatible with structural vibration modes (par. 3.3.2.1).

The possibility of simultaneously collecting synchronized displacement signals from distributed locations is realized in a microwave-based system that works on the same principle as radar. For instance, such a system was used to measure a 183 m chimney. Thirty six points on the structure were tracked simultaneously with a resolution approaching 0,01 mm at a sample rate of 50 Hz (higher speeds are possible), enabling a partial mode shape to be recovered. The system has also been used successfully for operational modal analysis of a large concrete bridge. Such a system is ideal where attachment of traditional sensors is impossible, although performance and ability to track parts of structure depends on the radar reflectivity of the structure and its surroundings (par. 3.3.2.6).

For the measurement of relative displacements, reference stations or fixed positions are necessary. However, quite often no convenient reference positions (for laser or total station measurements at long range) are available (e.g. in built-up urban areas), and even when available, atmospheric conditions may degrade the system performance. An alternative solution that provides absolute displacements is to use high-precision accelerometers or seismometers, since their signals are derivatives of displacement, which can in principle be recovered by the inverse operation of integration. The significant problem in this case is that numerical integration of the digitized signals results in amplification of the inevitable signal noise at low frequencies $\omega$, in proportion to $1/\omega$ for velocity and $1/\omega^2$ for (the more common) acceleration signals (par. 3.3.2.9). For Frequencies < 0,16 Hz the amplification becomes greater 1.

The *electro-mechanical impedance (EMI)* technique is relatively new entrant in the field of structural health monitoring (SHM). Thin patches of the ceramic lead zirconate titanate (PZT), surface–bonded on the host structure, play the key role as "impedance transducer" in this technique. They act as collocated actuators and sensors and employ ultrasonic vibrations (typically in 30-400 kHz range) to obtain characteristic admittance "signatures" of the structure. The sensitivity of the PZT patches is high enough to capture structural damage at the incipient stage, well before it acquires detectable macroscopic dimensions. The main limitation of the EMI technique is the localized zone of influence of the PZT patch as an impedance transducer. In addition, the impedance analyzers used with PZT patches have been cumbersome and expensive, but recently an inexpensive impedance measurement chip was developed (3.3.9).

An alternate simple form of wireless monitoring is to use *synchronized autonomous recorders*, with post processing to splice data records together. Accurate timing is provided

by GPS antennae, but delays of the order of milliseconds cannot be avoided, limiting such a system to applications on structures with natural frequencies well below 10 Hz (par. 3.4.2).

For dynamic signals, sample rates would depend on the structure size and frequency range of the loading. For global response of long span bridges (>500m) and tall buildings (>200m) 10 Hz bandwidth is more than adequate, requiring sample rates approximately 2,5 times larger to provide room for anti-alias filtering. For short span highway bridges and pedestrian structures, bandwidth up to 40 Hz will suffice. Seismometers typically default to 100 Hz bandwidth (par 3.5.1).

*Non-destructive evaluation (NDE)* applications have greatly progressed over the last decade especially for bridge type structure's inspections. The dominant practice in evaluation of bridge decks is by visual inspection and use of simple nondestructive methods like chain drag and hammer sounding. Modern nondestructive evaluation of concrete and concrete bridge decks has its origins in geophysics. A number of techniques introduced exploit various physical phenomena (acoustic or seismic, electric, electromagnetic, thermal, etc.) to detect and characterize specific deterioration processes or defects, as summarized in Table 19 (par. 3.6).

Table 19. NDE techniques and their application to bridge deck deterioration/defect detection and characterization (Aktan et al. 2011)

| | **NDT Method** | **Defect/deterioration applications** | **Other applications** |
|---|---|---|---|
| Electro-Magnetic | Ground Penetrating Radar (GPR) | Deterioration of concrete induced by corrosion, salt and acid actions, water penetration. Indirect delamination detection. | Thickness of the deck, concrete cover, rebar location. |
| | Electrical Conductivity | Damage to rebars and tendons | |
| | Spectral Induced Polarization (SIP) | Detection of voids and presence of moisture | |
| Acoustic/ Seismic | Impact Echo (IE) | Corrosion induced deck delamination detection and characterization. | Thickness of the deck. Investigation of crack |
| | Ultrasonic-echo (UPE) | Detection of voids and other anomalies. | Localization of rebars and tendons |
| | Ultrasonic Surface Waves (USW) | Measurement of degradation of mechanical properties (modulus, strength) | |
| | Ultrasonic Transmission (UPV) | Measurement of degradation of mechanical properties, detection of voids and cracks | Reinforcement and tendon ducts detection |
| Chemical/ Potential | Potential mapping | Corrosion of reinforcement | |
| | Laser Induced Breakdown Spectroscopy (LISB) | Near surface analysis of ingress of chemicals | |
| Thermal | Infrared (IR) Thermography | Detection of debonding of overlays, delamination, presence of moisture and near surface voids. | |

**Chapter 4** is on *Data Processing and Direct Data Interpretation*.

Sub-headings:



Successful data interpretation leads to the following benefits (par. 4.1):

- Increased efficiency and effectiveness of visual inspection by providing information relating to what to look and where
- Improved decision making for further instrumentation and testing
- Better estimation of structural reliability
- Better overall structural management for decisions such as replacement planning, retrofit strategies and maintenance budget expenditures
- For civil infrastructure owners and designers, improved insight into what happens to structures during service
- Development of an integrated framework for structural condition assessment
- Increased generic knowledge of in-service structural behavior that can be distilled into educational materials for students and practitioners
- Quantitative contribution to extending concepts of performance-based structural engineering

Non-parametric models are defined as non-physical numerical models that in some cases allow data condensation and reconstruction using a limited number of parameters. For this type of model, the structural identification process is generally a parametric curve-fit of the given mathematical functions to the measured data. The parameters themselves do not have normally any direct physical interpretation. The primary goal of this approach is to detect anomalies in behavior. Anomalies are detected as a difference in measurements with respect to measurements recorded during an initial period. More specifically, this approach involves examining changes over a certain period during the life of a structure. Table 20 summarizes the strengths and weaknesses of non-physics based as well as physics based models.

Table 20. Examples of strengths and weaknesses of model-free and model-based data interpretation (Aktan et al. 2011)

| **Interpretation types** | **Strengths** | **Weaknesses** |
|---|---|---|
| **Non-Physics Based (Direct Signal Analysis)**<br>Most appropriate when<br>- many structures need to be monitored<br>- there is time for training the system | • No modeling costs<br>• May not need for damage scenarios<br>• Many options for signal analysis<br>• Incremental training can track damage accumulation<br>• Good for long-term use on structures for early detection of situations requiring model-based interpretation | • Physical interpretation of the signal may be difficult<br>• Weak support for decisions on rehabilitation and repair<br>• Indirect guidance for structural management activities such as inspection and further measurement<br>• Cannot be used to justify replacement avoidance |
| **Physics-based (Structural or Modal Models)**<br>Most appropriate when<br>- design model is not accurate<br>- structure has strategic importance<br>- damage is suspected<br>- there are structural management challenges | • Interpretation is easy when links between measurements and potential causes are explicit<br>• The effects of changes in loading and use can be predicted<br>• Guidance for further inspection and measurement<br>• Consequences of future damage can be estimated<br>• Support for planning rehabilitation and repair<br>• May help justify replacement avoidance | • Modeling is expensive and time consuming<br>• Errors in models and in measurements can lead to identification of the wrong model<br>• Large numbers of candidate models are hard to manage<br>• Identification of the right model could require several interpretation - measurement cycles<br>• Complex structures with many elements have combinatorial challenges |

Examples of the data-driven models include autoregressive models (AR) and variants such as ARNA, ARX and ARMAX models and the rational polynomial model. In the case of the rational polynomial model, it is interesting to note that although the polynomial coefficients themselves have no particular physical meaning, they can easily be converted to the form of a modal model (par. 4.2).

Anomaly detection (may also be referred as outlier analysis) is an important consideration for any long term monitoring effort. While structural health monitoring has particular interest in detecting anomalies associated with a change to the constructed system (indicative of damage), anomalies in measured signals can also arise for a number of reasons tied to noise, operational and environmental variations of a structure, interference or sensor malfunction. Thus long term monitoring efforts often implement a variety of automated processing measures to remove electrical spikes, sensor drifts and other distortions, which include digital filtering and local spline fitting/interpolation, as well as observe for any changes that may indicate alteration of the constructed system. The sophistication of these methods can be enhanced through the use of statistical significance tests that identify anomalies whenever metrics exceed thresholds established during some training period on the undamaged/new structure. While the total correlation value should be constant or stationary in normal conditions, when damage occurs these values change (par. 4.2.1).

One of the most basic transforms applied in the analysis of data is the *Fourier Transform*, whose subsequent interpretation in auto or cross power spectral densities enables a basic representation of the energy associated with the various modes contributing to measured data. Unfortunately, the fact that data is often characterized by non-stationary or non-linear

features obscured by the harmonic bases of the Fourier Transform prompted a departure from this classical approach. To overcome the limitations of Fourier Transforms, a Short-Time Fourier Transform (STFT) was developed by introducing a short-duration window w(t) centered at time $\tau$ to the harmonic bases of the original transform. The spectral coefficients could then be determined over this short length of data, which is assumed to be stationary. One hallmark of Fourier and Short-Time Fourier Transforms is the fact that their frequency resolution is fixed, making them ill-suited for analysis of signals that may have both low and high frequency components. This motivated an alternative approach using basis functions with compact support in both frequency and time and then scaled via dilations to optimally adjust their resolutions based on the frequency being analyzed, yielding a multi-resolution analysis. The Wavelet Transform (WT) was engineered with this mind. The continuous Wavelet Transform (CWT) is a linear transform that generates a time-frequency energy density called a scalogram by decomposing a signal x(t) via basis functions that are simply dilations and translations of the parent wavelet g(t) through the convolution with the signal. Dilation by the scale a, inversely proportional to frequency, represents the periodic or harmonic nature of the signal. Any function satisfying basic admissibility conditions can serve as a parent wavelet. One of the most common wavelets used in Civil Engineering applications is the Morlet wavelet, which effectively has a Gaussian-windowed Fourier basis (par 4.2.2.1).

*Principal Components Analysis (PCA)* is an example of a linear data reduction tool that is capable of compressing data and reducing its dimensionality so that essential information is retained and made easier to analyze than the original data set. The main objective is to transform a number of related process variables to a smaller set of uncorrelated variables. A key step is finding those principal components that contain most of the information. PCA is based on an orthogonal decomposition of the covariance matrix of the process variables along directions that explain the maximum variation of the data, usually contained in only the first few principal components (par. 4.2.2.3).

**Chapter 5** is on *Selection, Application and Calibration of Physics-Based Models*.

Sub-headings:

- 5.1 Introduction
  - 5.1.1 Structural model
    - 5.1.1.1 Use of a single model
    - 5.1.1.2 Use of multiple models
  - 5.1.2 Modal model)
- 5.2 Model application for structural identification
  - 5.2.1 Identification using structural models
  - 5.2.2 Identification using modal models
    - 5.2.2.1 Complex exponential method
    - 5.2.2.2 Random decrement technique
    - 5.2.2.3 Power spectral approaches
    - 5.2.2.4 Applications in base-excited structures
- 5.3 Model calibration
  - 5.3.1 Error characterization
  - 5.3.2 Objective functions for error minimization
    - 5.3.3 Model calibration techniques
      - 5.3.3.1 Manual, heuristic based model calibrations
      - 5.3.3.2 Optimal matrix update methods
      - 5.3.3.3 Sensitivity–based update methods
      - 5.3.3.4 Eigenstructure assignment method
      - 5.3.3.5 Hybrid matrix update methods
      - 5.3.3.6 Evolutionary optimization
- 5.4 Closing remarks

The modal model consists of modal frequencies, modal vectors (also called mode shapes) and modal damping ratios, which are the modal parameters. The modal model is different from

the structural model in that it does not contain specific information about the structural connectivity or the geometric distribution of mass, stiffness and structural damping. But the modal parameters also describe the resonant spatial and temporal behavior of the structure. Further, the modal parameters are directly analogous to the eigensolution of the structural mass and stiffness matrix. This parallel between the experimentally derived and analytically derived components makes the modal model well suited for the process of model correlation. Structural frequency response function can be measured using standard experimental techniques. The modal model is defined in a coordinate space known as modal coordinates. These coordinates form a generalized basis for describing the vibratory motion of the structure with a relatively small number of parameters (par.5.1.2)

Most methods of parameter identification are essential regression techniques that estimate the parameters in models to simulate physical systems based on the outputs of (and in some cases input to) the systems using various optimization algorithms. Some of the classical approaches of parameter estimation include weighted least-squares estimation, best linear unbiased estimation (BLUE), maximum likelihood for deterministic parameters, mean squared, maximum a posteriori, weighted least-squares and BLUE for random parameters. More recently, the extended Kalman filter, $H\infty$ filter, sequential Monte Carlo methods and regression techniques based on support vector machines have been used in parameter identification for constructed systems (par. 5.2).

Methods for modal parameter identification can be categorized according to the order of the data that they use (low or high order) and according to the domain that the data is analyzed in (time or frequency domain), the number of inputs and outputs, and even the input excitation (ambient vs. seismic) they are acquired under (par. 5.2.2).

Differences between in-situ and predicted structural parameters and responses may arise from simplifications employed in the modeling process, e.g. in the representations of the boundary and support conditions, connectivity between various structural elements, unknown material properties and constitutive relationships (particularly those associated with soil and concrete), and energy dissipation (damping) mechanisms as well as measurement errors. However, differences may also arise due to temperature influences during measurements, changes in the condition of the structure due to factors such as deterioration, support settlement, loss of pre-stressing forces and changes in end conditions, and so far the effects of temperature, modeling errors and structural degradation cannot be easily distinguished in any case.

The calibration process involves selecting a small number of model parameters that have uncertainty so their values cannot be known a priori. Once these parameters are selected, various procedures are used to find their values for which the measurements best match the model predictions. This process can be particularly challenging due to the degree of freedom mismatch, as the number of response measurement locations is significantly less than the number of degrees of freedom in the finite element model (par. 5.3).

Modeling error ($e_{mod}$), defined as the difference between predicted response of a given model and that of an ideal model, has been studied by many researchers. Often modeling error have been classified by three components ($e_1$, $e_2$, $e_3$), respectively representing the error due to discrepancy between the behavior of the mathematical model and that of the real structure, the error introduced in the numerical computation of the solution of partial differential equations, and the error arising from inaccurate assumptions made during simulation. The component $e_3$ may be further separated into two parts ($e_{3a}$, $e_{3b}$) which respectively arise from assumptions made when using the model (typically assumptions related to boundary conditions such as support characteristics and connection stiffness) and from errors in values of model parameters such as moment of inertia and Young's modulus. Additionally, measurement errors ($e_{meas}$) - the difference between real and measured quantities - must be considered. Measurement errors result from equipment (precision, stability, robustness) as well as on-site installation faults (par. 5.3.1).

There are several different physics based equations that are used as either objective functions or constraints for the model updating, depending upon the parameters that are to be identified and the identification algorithm being used. No matter how they are cast,

objective functions seek to quantify the discrepancy between the model predictions and in-situ measurements or properties. A variety of measured responses and related parameters can be used for this purpose, including static displacements, tilts and strains, boundary condition elements, modal parameters (natural frequencies and mode shapes), modal strain energy, higher order derivatives, and flexibility-based error functions.

The objective function can also be derived directly from the equations of motion to yield the "modal force error equation". This approach requires that mass and stiffness matrices from a linear elastic FEM to be known prior to the identification process. Substituting the eigenvalues (modal frequencies) and eigenvectors (mode shapes) measured from the structure into this equation along with the mass and stiffness matrices from the original model yields a vector that is defined as the "modal force error", or "residual force". This vector represents the harmonic force vector that would have to be applied to the structure at this modal frequency in order to satisfy force equilibrium. Optimization schemes for structural identification seek to minimize the objective function by variation of the parameters in the model. However, optimization schemes are driven by purely mathematical consideration. This fact can lead to the optimization scheme preferring to vary certain model parameters into physically inconsistent states, e.g. perturbing a particular material density or a structural damping parameter to be negative. To avoid such situations, it is important to optimize the objective functions subject to constraints on the parameter values.

Constraints are typically enforced in one of two ways. The first way is "implicitly", in which the constraint is incorporated into the model form, such as the relationship between an element’s elastic modulus and its elemental stiffness matrix. The second way a constraint can be enforced is "explicitly", in which the optimization scheme contains the possible perturbations to the model parameters to be within the limits imposed by the constraints. In this manner the possible perturbations to the parameters are reduced to a set that will still satisfy the constraints.

In modal parameter identification, constraints such as modal frequency and modal damping remaining positive are typically imposed explicitly after the optimization has been completed.

In structural parameter identification, there are several constraints that are used either implicitly or explicitly, depending upon the optimization algorithm that is used. Preservation of the mass, damping, and stiffness matrix (referred to collectively as "property matrices") symmetry is used as a constraint. Preservation of the property matrix sparsity (the zero/nonzero pattern of the matrix) is also used as a constraint. The preservation of sparsity is one way to preserve the allowable load paths of the structure in the updated model. Preservation of the property matrix positivity is also used as a constraint. This constraint ensures that situations such as negative inertia or negative damping do not occur. (par 5.3.2).

One of the most basic approaches to model calibration is the parameter sensitivity study, to examine the impact of varying selected parameters on the simulated dynamic properties of a structure, and thus, to determine the most critical parameters for the global model calibration. Parameter sensitivity based model updating can be conducted using an automated procedure or manually by analyzing the sensitivity of the parameters and understanding their impact on the structural response not only mathematically, but also in physical terms. In such an approach it is seen that the most sensitive parameters can be material properties, boundary, and continuity conditions. In bridges, critical properties include the elastic modulus of concrete, properties of rigid links used to connect shell and beam finite elements in decks, boundary conditions, and force releases and kinematics of the movement systems.

For the initial global calibration, the discrepancy between the global dynamic properties that are measured and the model responses are minimized by changing the identified sensitive parameters. Models calibrated in this way are referred to as the "only globally calibrated" model of the structure. To permit local calibration of the numerical models, it is possible to conduct controlled load tests on the structure with known loads. Local calibration can then be performed by comparing and correlating these experimental results with the corresponding FEM analytical responses.

Once globally calibrated, the model's ability to simulate the modal properties can be investigated using an eigenvalue analysis of the model under different values of the most critical parameters of the structure (par 5.3.3.1).

One class of model updating is based on the modification of structural model matrices such as mass, stiffness and damping to reproduce as closely as possible the measured static or dynamic response from the data. These methods solve for the updated matrices (or perturbations to the nominal model that produce the updated matrices) by forming a constrained optimization problem based on the structural equations of motion, the nominal model, and the measured data. Comparisons of the updated matrices to the original correlated matrices provide an indication of correlation of the model and the experimental data. The problem is generally formulated as a Lagrange multiplier or penalty-based optimization (par 5.3.3.2).

Another class of matrix update methods, sensitivity-based update method, is based on the solution of a first-order Taylor series that minimizes an error function of the matrix perturbations. This involved the determination of a modified model parameter vector (consisting of material and/or geometric parameters), where the parameter perturbation vector is determined using the Newton-Raphson method. The main difference between the various sensitivity-based update schemes is the method used to estimate the sensitivity matrix, which can be either experimental or analytical quantities. For experimental sensitivity, the orthogonality relations can be used to compute the modal parameter derivatives. Analytical sensitivity methods usually require the evaluation of the stiffness and mass matrix derivatives, which are less sensitive than experimental sensitivity matrices to noise in the data and to large perturbations of the parameters.

A methodology for sensitivity-based matrix update was developed, which takes into account variations in system mass and stiffness, center of mass locations, changes in natural frequency and mode shapes, and statistical confidence factors for the structural parameters and experimental instrumentation. The method uses a hybrid analytical/ experimental sensitivity matrix, where the modal parameter sensitivities are computed from the experimental data, and the matrix sensitivities are computed from the analytical model (par. 5.3.3.3).

A two-step model calibration technique was developed for damage-detection in large structures with limited instrumentation. The first step uses optimal matrix update to identify the region of the structure where damage has occurred. The second step is a sensitivity-based method, which locates the specific structural element where damage has occurred. (par. 5.3.3.5).

**Chapter 6** is on *Utilization of St-Id for Assessment and Decision Making*.

Sub-headings:

6.1 Introduction
6.2 Performance-based engineering
6.3 Risk-based decision-making
6.4 Quantitative vs. qualitative risk assessment
6.5 Closing remarks

Decision-making, especially related to structural maintenance, preservation or replacement occurs at a complicated intersection between technical considerations and social, political, environmental, and economic considerations.

For example, many private building owners are concerned about how interventions or investigations into structural performance are viewed by the general public, as impressions of safety or serviceability concerns may negatively influence a building's market or rental values (par. 6.1).

Over the last decade there has been a significant amount of work in the area of performance based engineering for constructed systems. While a lot of this attention has been focused on

design, this work also provides the proper framework and context for decision-making related to existing constructed systems. Key limit states and desired performances of constructed systems are defined from a holistic standpoint. Table 21 is taken from a report of the ASCE Performance-Based Engineering Committee and illustrates the key goals for constructed systems for various limit states and various perspectives (domains: Engineered, Human-societal, Natural). Table 21 is representative of common concerns of owners that have been influenced either directly or indirectly by applications of St-Id. Examples of direct influences include structural safety, stability of failure, hazards risk management, etc., where applications of St-Id have the ability to provide information directly related to the relevant goals. In the case of indirect influences, St-Id applications have the ability to contribute to meeting goals, but in an indirect way. Examples of this type of influence include organizational effectiveness, transparency and maintainability, among others (par 6.2).

Table 21. Multi-dimensional performance-matrix for constructed systems (Aktan et al. 2011)

| **Domain / Limit state** | **Engineered** | **Human-societal** | **Natural** |
|---|---|---|---|
| **Operational and utility** | Safety<br>Security<br>Efficiency | Transparency<br>Organizational effectiveness<br>Fiscal prudence | Sustainability<br>Minimal impact<br>Hazards risks<br>Management |
| **Engineering** | Serviceability<br>Durability<br>Safety<br>Stability of failure | Inspectable<br>Maintainable<br>Adaptable<br>Renewable | Recyclable<br>Carbon footprint<br>Unobtrusive |
| **Societal goals** | Long-term economic sustainability<br>Preserve culture | Healthy and just society<br>Promote good governance | Respecting the environment<br>Rely on sound science |

A versatile metric to measure the degree of performance (or lack of performance) related to the goals shown in Table 21 is the term "risk". The typical definition of risk is the probability of an event occurring (such as a structural failure) times the consequences associated with the event. In general terms, owners are concerned with the following three questions:

1. What is the probability that the structure will fail to perform, p(f), related to operations/ maintenance, serviceability/ durability, or safety?; ⇒ hazard, vulnerability
2. What are the consequences associated with the structure failing to perform?; ⇒ exposure
3. How certain are the estimates to question (1) and (2)? ⇒ uncertainty premium.

The product of the probability of failure (depending on hazard and vulnerability) and the consequences of failure (exposure) are referred as the actual risk. In practice, there is an additional component termed uncertainty premium, which is related to question 3 above and represents the uncertainty associated with the estimates of the other components of risk. Hence, "perceived risk" is the sum of actual risk and uncertainty premium, which is visually illustrated in Fig. 84. This figure shows three axes which correspond to the three components of risk typically used in civil/structural engineering: hazard, vulnerability and exposure. The volume of the box defined by these components represents the risk posed by a structure for the various hazards considered. The perceived risk is generally much larger than the actual risk, owing to the conservative nature of infrastructure decision-makers (notable exceptions are some cases of bridge failures).

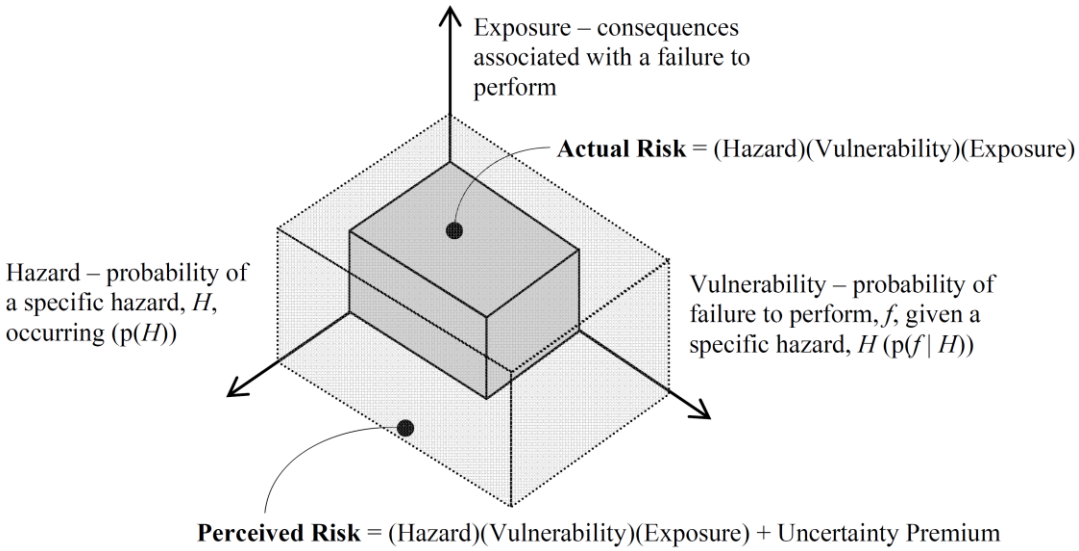



Fig. 84. Schematic representation of actual and preceived risk (Aktan et al. 2011)

The role of St-Id in decision-making can be seen as reduction of the uncertainty premium.

With the focus of St-Id being the better understanding of the performance of constructed systems, this reduction in the uncertainty premium afforded by St-Id is nearly always related to better estimates of vulnerability. But the possible reduction will be limited if the principal uncertainty is related to the level of hazards, such as wind pressure on high-rise buildings or truck weights, speed and frequency for bridges.

Table 22 provides some examples of hazards, vulnerabilities and exposures for various limit states of constructed systems (par. 6.3).

Although the formal definitions of risk and vulnerability are useful from a conceptual standpoint, it is almost impossible to estimate them in absolute, quantitative terms. A model which is not only representative of the structure as observed during testing, but also at the limit state of interest, is required. Such a case may involve a serviceability concern and a St-Id that monitored the structure during normal service conditions. However, when for ultimate limit states a St-Id using ambient vibration monitoring is used to assess seismic vulnerability, additional considerations/investigations are necessary. One has to consider, that the experimental data acquired would be representative of the low-amplitude, linear response of the structure. This data will carry very little information related to higher- amplitude, nonlinear response, which will be mobilized during the considered earthquake (par 6.4).

Table 22. Some relevant performance limit states, hazards, vulnerabilities and exposures with regard to bridges (Aktan et al. 2011)

| Performance Limit States | Hazards | Vulnerabilities | Exposures |
|---|---|---|---|
| **Safety: Geotechnical/ Hydraulic** | • Flowing water<br>• Debris and ice<br>• Seismic<br>• Vessel Collision<br>• Flood | • Scour/Undermining<br>• Loss of support<br>• Soil liquefaction<br>• Unseating of superstructure<br>• Settlement<br>• Overtopping | • Loss of human life<br>• Replacement and repair costs<br>• Impact of removal from service related to:<br>  • Safety – life line,<br>  • Economic<br>  • Social – mobility<br>  • Defense |
| **Safety: Structural** | • Seismic<br>• Repeated loads<br>• Trucks and overloads<br>• Vehicle collision<br>• Fire | • Lack of ductility and redundancy<br>• Fatigue and fracture<br>• Overloads<br>• Details and bearings | |
| **Serviceability, Durability and Maintenance** | • Winter maintenance practices<br>• Climate<br>• Intrinsic Loads<br>• Impact (Vertical)<br>• Environment | • Corrosion<br>• Cracking/spalling<br>• Excessive deflections/ vibrations<br>• Chemical attacks/reactions<br>• Difficulty of maintenance | • User costs<br>• Maintenance costs<br>  • Direct<br>  • Indirect – delays, congestion, etc. |
| **Functionality and Cost** | • Traffic<br>• Special traffic and freight demands | • Network redundancy and adequacy<br>• Geometry and roadway alignment | • Loss of human life and property (accidents)<br>• Economic and social impacts of congestion |

**<u>Chapter 7</u>** is on *<u>Case Studies – Structural Identification of Buildings</u>*.

Sub-headings:

7.1 Chicago Full-Scale Monitoring Program
- 7.1.1 Program description
- 7.1.2 Objectives of St-Id application
- 7.1.3 Model development
- 7.1.4 Experimental studies
- 7.1.5 Data analysis and model calibration
- 7.1.6 Interpretation

7.2 Four Seasons Building
- 7.2.1 Building description
- 7.2.2 Objective of St-Id application
- 7.2.3 Model development
- 7.2.4 Experimental studies
- 7.2.5 Data analysis, model calibration
- 7.2.6 Interpretation and decision making)

7.3 Three-Story Concrete building in CSMIP
- 7.3.1 Program description

Chapter 7 offers an inventory of 38 instrumented buildings. Since many applications are a result of California Strong Motion Instrumentation Program (CSMIP), this program will receive additional attention in Section 7.3. In fact, strong motion programs have generated more than 150 instrumented buildings in the United States, 100 sites in Japan and 40 sites in Taiwan. This is in stark contrast to the situation outside of seismic zones, especially in the United States, where owners fear that public disclosure of monitoring efforts may generate public misconceptions regarding the building's condition and even liability issues.

The *Chicago Full-Scale Monitoring Program* (section 7.1) has the overarching goal of providing the first systematic full-scale validation of tall building design practice in the country. Based on data collected in this study, it has been reaffirmed that concrete in general dissipates more energy than steel, but more importantly that structural systems with increasingly more cantilever action manifest less viscous damping and show less amplitude dependence.

The case study *Four Seasons Building* (section 7.2) provided the opportunity to deploy sensors with high density and impart controlled excitations to provide valuable insight into the performance of this common type of structural system and reveal the reasons for its poor performance during the Northridge earthquake. Several of the updated stiffness factors were significantly reduced from their initial values and corresponded well to locations of observed damage in the building.

From case study *Three-Story Concrete Building in CSMIP* (section 7.3) it may be concluded that the amplitude of vibrations (shaking intensity) appears to influence the structure, and the SSI effects in opposite directions; and thus, the combined effect, which shows up in the flexible-base properties has no discernable overall trend on the observed natural frequencies. The fixed base frequencies are affected by the level of vibrations more significantly than the flexible base frequencies. In both directions, stronger ground motions lead to an overall decrease in the fixed-base frequencies, which are representative of the flexibility of the structure alone. The increase in the vibration amplitudes leads to an overall increase in the transverse, and an overall decrease in the flexible-base frequencies, which are representative of the flexibility of the structure and the surrounding soil media together.

The case study *Guangzhou New TV Tower* is given in section 7.4. To ensure safety during construction and operational performance during typhoons and earthquakes, a sophisticated long-term structural health monitoring system consisting of about 800 sensors has been implemented for on-line monitoring. In the meanwhile, a hybrid mass damper control system

is installed on the main tower while two tuned mass dampers are suspended on the antenna mast for suppressing wind-induced vibration.

The case study *Seven-Story RC Building Slice* is given in section 7.5. The objective of this test program was to verify the seismic performance of a mid-rise RC wall building designed for lateral forces obtained from displacement-based design methodology, which are significantly smaller than those dictated by current force-based seismic design provisions in the United States. As a payload project, system and damage identification studies were performed on the test structure at different damage states to evaluate the performance of the applied methods.

**Chapter 8** is on *Case Studies – Structural Identification of Bridges.*

Sub-headings:

*8.1 Henry Hudson Bridge*
- 8.1.1 Bridge description
- 8.1.2 Objective of St-Id application (Step 1)
- 8.1.3 System identification method development (Step 2)
- 8.1.4 Experimental studies (Step 3)
- 8.1.5 Data analysis (Step 4) and model calibration (Step 5)

*8.2 Throgs Neck Bridge*
- 8.2.1 Bridge description
- 8.2.2 Objective of St-Id application (Step 1)
- 8.2.3 System identification method development (Step 2)
- 8.2.4 Experimental studies (Step 3)
- 8.2.5 Data analysis (Step 4) and model calibration (Step 5)

*8.3 Golden Gate Bridge*
- 8.3.1 Bridge description
- 8.3.2 Objective of St-Id application
- 8.3.3 Experimental program
- 8.3.4 System identification and data analysis
- 8.3.5 Comparison and interpretation

*8.4 Vincent Thomas Bridge*
- 8.4.1 Bridge description
- 8.4.2 Objective of St-Id application
- 8.4.3 Bridge identification using seismic vibration data
- 8.4.4 Forensic study of ship-bridge collision accident using St-Id techniques)

*8.5 Hakucho Suspension Bridge*
- 8.5.1 Bridge description
- 8.5.2 Objective of St-Id application
- 8.5.3 Instrumentation and ambient vibration monitoring
- 8.5.4 Data analysis and system identification
- 8.5.5 Structural Identification and interpretation

*8.6 Yokohama Bay Bridge*
- 8.6.1 Bridge description
- 8.6.2 Objective of St-Id application
- 8.6.3 System identification method development
- 8.6.4 Experimental studies
- 8.6.5 Data analysis and interpretation

*8.7 Alfred Zampa Memorial Bridge*
- 8.7.1 Bridge description and experimental studies
- 8.7.2 System identification methods used
- 8.7.3 Experimental studies
- 8.7.4 Data analysis and interpretation

The case study *Henry Hudson Bridge* is given in section 8.1. The primary objective for conducting the St-Id of the bridge was to support a seismic vulnerability assessment study and, if required, a subsequent retrofit design. This study illustrated the significant modeling (epistemic) uncertainties that can challenge the reliability of FE models for large constructed systems. Without performing a St-Id of the test bridge, the seismic evaluation and possible retrofit designs would have been underpinned by an FE model that was poorly correlated with the actual response of the structure.

The case study *Throgs Neck Bridge* is given in section 8.2. Structural safety inspection and rehabilitation have been performed several times on the bridge since 1980, which included replacing the roadway decks, repairing the structural steel, modifying expansion joints, replacing existing rocker bearings, improving the drainage system, re-wrapping the main cables, and rehabilitating the electrical systems. As part of a seismic vulnerability assessment an ambient vibration monitoring study of the suspended spans and towers of the bridge was carried out. The seismic design criteria required that the first three vertical modes and the first three horizontal modes shall be compared to those obtained from ambient vibration measurements of the bridge for verification of the structural model. The constructed FE model apparently satisfies the requirements of the Seismic Design Criteria and is ready to be used as a tool for bridge maintenance related decision making.

The case study *Golden Gate Bridge* is given in section 8.3. The objective of this study is to present a statistical analysis of the vibration modes of the Golden Gate Bridge using ambient acceleration data obtained from large-scale deployment of a wireless sensor network (WSN). The contribution is to demonstrate that the spatial and temporal sensing possible with WSNs provides high resolution and confidence in the identified vibration modes. As examples of using the confidence intervals, three sets of earlier identified vibration modes of the Golden Gate Bridge are compared with the statistical results obtained from WSN data. These data sets were obtained using different identification procedures with data from earlier deployments of accelerometers, finite element models and the peak picking methods. The identified vibration modes are generally similar to the statistical analysis of the data from WSN. Some of the mode shape ordinates, however, are outside the confidence intervals: In this comparison, it must be recognized that the bridge has been retrofitted since the earlier data was collected including a complete replacement of the roadway deck. This comparison indicates that the earlier estimates of mode shapes are not very accurate because of the change in the main-span, but also because of the quality of the data and the error in the system identification method for the earlier estimates. The 1985 data best matches the confidence intervals for the higher frequency vertical modes, indicating that the change in

dynamic properties of the bridge most affected the lower frequency modes. In the case of the developed FE model all the modes except for the second transverse mode fall within the confidence intervals.

The case study *Vincent Thomas Bridge* is given in section 8.4. The bridge has been monitored with twenty-six accelerometers. These sensors were installed within the framework of the California Strong Motion Instrumentation Program (CSMIP), and have been in operation since 1972. The recording of the sensor data, however, is trigger-based and data are saved on permanent data storage devices only if the raw data are characterized as seismic signals. Therefore, valuable data from ambient and abnormal loading conditions other than earthquakes, such as ship-bridge collision, are wasted. Hence, the main objective of the St-Id application for the bridge is to quantify the structural performance of the bridge when subjected to various loading conditions and significant environmental effects, which is infeasible with traditional visual bridge inspection approaches.

The case study *Hakucho Suspension Bridge* is given in section 8.5. Portable as well as embedded sensor networks are utilized to measure wind condition and dynamic response of the bridge. The initial objective was to verify the results of wind tunnel test, especially concerning the aerodynamic forces. This was the first initiative in Japan to monitor a long-span bridge with a very dense array of sensors (i.e. forty measurement points on one half-span of the bridge) for over two weeks. Despite variations of natural frequencies and damping ratios, there seem to be clear trends between frequencies, damping ratios and acceleration amplitudes (and consequently, the wind speed). Results show that in general the natural frequencies decrease as the wind velocities increase and damping ratios increase as the wind velocities increase. The variations of natural frequencies and damping ratios are more apparent in the low-order modes as evident by the slopes of the linear trend. The mode shape components reveal two different trends. The real parts of mode shape vectors do not exhibit a distinct trend, indicating no obvious changes. The modal phase angle computed from the imaginary part of the mode shape vectors, however, revealed a clear trend. It was observed that the phase difference is large when the root-mean-square (rms) of acceleration is very small and decreases when acceleration rms becomes large. These phase differences indicate that the system is non-proportionally damped. The locality effect of phase difference that was concentrated mainly at the edge of girder suggests the contribution of additional damping and stiffness caused by friction force at the bearings. In addition, the decrease and increase of natural frequencies and damping ratios indicated the effect of aerodynamic force along the girder. To study the extent of these effects, they were modeled as additional stiffness and damping: (1) located at the edge of the girder to represent the friction force at the bearings and expansion devices and (2) distributed alongside the girder to illustrate the aerodynamic forces. The results suggest the contribution of aerodynamic forces was much smaller than the effect of the friction force at the bearing. The aerodynamic force contribution is on the order of one-percent when compared to the contribution of the friction force, and its behavior is in agreement with the aerodynamic force obtained from wind tunnel results. Furthermore, the additional damping and stiffness due to the friction force display clear trends: small damping and large stiffness during low-amplitude vibrations. When the wind speed increases the damping also increases, which is when the bearings are unstuck, whereas the stiffness is decreasing as the result of increasing flexibility of the structure.

The case study *Yokohama Bay Bridge* is given in section 8.6. Because of the high intensity of seismic activities in Japan, monitoring for seismic response has been widely employed for decades especially for bridges with special features such as curved decks and bridges with new technologies such as base-isolation. The bridge was constructed on a soft soil that necessitated a new special foundation system. It is also located near an active fault and close to the epicenter of the 1923 Great Kanto Earthquake. Therefore to confirm the seismic design and to monitor the bridge performance during earthquakes, a comprehensive and dense array monitoring system was installed. The objectives of the monitoring system are the evaluation of seismic performance, verification of and comparison with seismic design, and observation of possible damage. Particular attention is given to the seismic isolation device in the form of Link Bearing Connection (LBC). In general, natural frequencies identified from earthquake records are in good agreement with those from the ambient and forced vibration

tests. The frequencies are almost constant with respect to earthquake amplitude. Modal damping ratios of several lower modes, however, indicate magnitude dependence. The performance of LBC was evaluated using modal characteristics identified from strong motion records. LBC is a type of connection designed to minimize the inertial force of superstructure from being directly transferred to substructures. In the design, the LBC is expected to function as a hinged connection especially during large vibrations in longitudinal direction. This implies that the girder and pier-cap work as separated units, and therefore the force from the superstructure will not be transmitted into the end-piers. Three typical first longitudinal modes were found from system identification with the main focus on the relative modal displacement between end-piers and girder. During small earthquakes, the LBC has yet to function as a fixed connection. Therefore higher natural frequencies due to the stiffer connection were observed. A mixed hinged-fixed mode was observed during moderate earthquakes. The fully hinged connections at both of the end-piers were observed mostly during large earthquakes.

The case study *Alfred Zampa Memorial Bridge* is given in section 8.7. The bridge is the first major suspension bridge built in the United States since the 1960s. The design and construction of the bridge incorporates several innovative features that have not been used previously for a suspension bridge in the USA, namely (1) orthotropic (aerodynamic) steel deck; (2) reinforced concrete towers; and (3) large-diameter drilled shaft foundations. The bridge is also the first suspension bridge worldwide with concrete towers in a high seismic zone. In this study, three output-only system identification methods, namely the multiple-reference natural excitation technique combined with the eigensystem realization algorithm (MNExT-ERA), the data-driven stochastic subspace identification (SSI-DATA) method, and the enhanced frequency domain decomposition (EFDD), were applied to identify the modal parameters of the bridge based on bridge vibration data collected from two types of tests: ambient vibration test and forced vibration tests based on controlled traffic loads. From the modal identification results obtained, it was observed that the natural frequencies identified using the three different methods are in excellent agreement, while the relative difference in the damping ratios identified using different methods is significantly larger.

The case study *Langensand Bridge* is given in section 8.8. The bridge is a good example of innovative structural engineering using composite design. During the design stage, engineers made justifiable conservative assumptions regarding aspects such as the composite behavior and support conditions. Behavioral models using these assumptions often underestimate the load-bearing capacity of the structure. An accurate estimate of the reserve capacity of the bridge was sought by the owner (City of Lucerne) who manages many structures in the city. Such an estimate is useful to the owner for tasks such as routing heavy vehicles across the city and for later decisions in cases of modifications and deterioration. The bridge model has several parameters that are assigned values by stochastic sampling during candidate model generation process. The multi-model candidate selection approach described in Section 5.1.1.2 is used for structural system identification. A set of 1000 models is generated by sampling several modeling assumptions. Models are then filtered using collected measurements to obtain the set of candidate models. The candidate models were able to predict the service behaviour of the structure within 7% of measured values. Estimates using candidate models showed that the structure has 30% reserve capacity compared with the design model for the vertical displacement criteria. The study also found that minimizing the discrepancies between predictions and measurements would have led to wrong models that overestimate the reserve capacity.

The case study *Sunrise Movable Bridge* is given in section 8.9. Movable bridges experience major deterioration as compared to regular fixed bridges due to their complex structural, mechanical and electrical system and even a minor malfunction of any component can cause an unexpected failure of bridge operation. This necessitated a comprehensive SHM plan to monitor the most common issues associated with movable bridges. This includes observation of traffic-induced strains and accelerations and their comparison with analytical results from a Finite Element Model. Strain data from opening and closing operations are also evaluated and compared with FE model results.

The case study *New Svinesund Bridge* is given in section 8.10. Due to the uniqueness of the design, a monitoring system was installed on the bridge to check that the bridge is built as designed, to get a better understanding of the bridge's behavior, and to produce an initial database of the undamaged bridge. The monitoring program started during the construction phase and was kept running during the first years of service. Later the decision was taken to use the available measurement data for St-Id. The aim was to obtain a more accurate FE model, which may later, if kept up-to-date, be used for the structural assessment and maintenance. Furthermore, the possibility to use the measurement program to estimate uncertain structural parameters regarding boundary conditions and interactions between parts of the structure was studied. It was shown that the bridge behaves as designed. The high discrepancy in the numerical and experimental eigenfrequencies could be explained by a restrained bridge deck movement under ambient vibration. Furthermore, the load tests revealed a stiffer behavior of the bridge. This could partially be explained by the increased arch stiffness due to further hardening of the concrete and the contribution of the reinforcement to the arch stiffness. Besides that, an increase of the bridge deck girder stiffness by about 15% led to best agreement between the experimental and numerical responses. The reason for this is still unclear but might be due to the contribution of the railing system. Furthermore, a high influence of temperature and other environmental conditions on the monitored properties could be observed. Hence, an improved understanding of structural changes due to temperature and environmental conditions is needed to discern structural changes due to temperature and environmental conditions from structural changes due to damage.

### *6.1 Summary of Literature*

F. Duco, J. Faye, S. Caperaa, E. Reubrez. 2011. ***Seismic vulnerability assessment using the instrumentation of an existing building***. Key Engineering Materials, Volume 482.

France is a country composed of moderate seismic hazard regions but however vulnerable to earthquakes. Indeed, only a few parts of existing buildings have been built using paraseismic regulation. Several current large-scale seismic vulnerability assessment methods are used, as Hazus or Risk-UE, but they are inappropriate to the analysis of a specific building. In our case, we use an experimental approach to study the elastic behaviour of existing buildings: ambient vibration analyses seem to be an interesting way to determine the vulnerability. Ambient noise testing with Output-Only Modal Identification is a low-cost non-destructive method to provide vibration data from civil engineering structures like buildings. The interest of this method is to obtain dynamic parameters with only natural excitations: wind, traffic, human activity... In the frame of the "Plan séisme des Hautes-Pyrénées", the building considered is the relatively regular 18-storey Ophite Tower located in Lourdes, France. The vibration measurements are conducted using a 24-channel system connected to an acquisition station. The modal parameters of this building (natural frequencies, modal shapes and damping) are calculated using the stochastic subspace identification method. These parameters, extracted from in situ data, are then used to calibrate a model. Having defined damage level criterion, the response motion, produced by seismic events, will lead to the determination of the vulnerability curves of Ophite Tower.

C. Michel, P. Guéguen, P.Y. Bard. 2008. ***Dynamic parameters of structures extracted from ambient vibration measurements: An aid for the seismic vulnerability assessment of existing buildings in moderate seismic hazard regions***. Soil Dynamics and Earthquake Engineering, 28(8):593-604.

During the past two decades, the use of ambient vibrations for modal analysis of structures has increased as compared to the traditional techniques (forced vibrations). The frequency domain decomposition (FDD) method is nowadays widely used in modal analysis because of its accuracy and simplicity. In this paper, we first present the physical meaning of the FDD method to estimate the modal parameters. We discuss then the process used for the evaluation of the building stiffness deduced from the modal shapes. The models considered here are 1D lumped-mass beams and especially the shear beam. The analytical solution of the equations of motion makes it possible to simulate the motion due to a weak to moderate earthquake and then the inter-storey drift knowing only the modal parameters (modal model). This process is finally applied to a nine-storey reinforced concrete (RC) dwelling in Grenoble (France). We successfully compared the building motion for an artificial ground motion deduced from the model estimated using ambient vibrations and recorded in the building. The stiffness of each storey and the inter-storey drift were also calculated.

S. Hans, C. Boutin, C. Chesnais. 2008. ***How far in-situ measurements may help to assess building vulnerability?*** Proceedings of 14$^{th}$ World Conference on Earthquake Engineering, Beijing, China.

The use of noise data in seismic diagnosis of buildings is analyzed. From the responses to ambient noise, harmonic excitation and shocks, the dynamic behaviour of usual buildings is identified in the range of $10-5-10-2g$. Taking advantage of the demolition, the influence of the light work elements, full precast façade panels, bearing masonry walls and the presence of neighbouring joined buildings is determined. These experiments show that noise measurements efficiently provide reliable data of real interest for understanding the actual building behaviour. Then, the integration of these data in a vulnerability diagnosis is presented. It is shown that regular concrete structures are described by suited beam modeling. Thus for a given structure, taking into account the noise data, the adequate beam model and maximum tensile and compression strains of concrete and steel as damage

criteria, two levels of ground acceleration can be determined, namely the Seismic Thresholds of Elasticity (STE) and of Yielding (STY). Quantify the levels that onset the structural damages and the plastic hinge may be a useful tool for vulnerability diagnosis. This concept presents the practical advantages to be based on real data, to minimize the introduction of uncertain assumptions and to provide an acceleration level easily comparable with the reference acceleration of the codes.

C. Michel, E. Lattion, M. Oropeza, P. Lestuzzi. 2009. ***Vulnerability assessment of existing masonry buildings in moderate seismicity areas using experimental techniques***. 2009 Asian-Pacific Network of Centers for Earthquake Engineering Research (ANCER) Workshop, Urbana, Illinois, United States.

Vulnerability assessment in moderate seismicity areas needs particularly accurate estimations of low damage grades, since they cause a great part of economic losses. For moderate level of shaking, the most important parameter is the fundamental resonance frequency as it varies with amplitude with consequences on seismic demand. In order to accurately but simply model the response of existing buildings to moderate earthquakes, in situ and laboratory tests together with a frequency drop approach were used. The dynamic behaviour of existing buildings, for which it is difficult to obtain plans and material physical properties, is first assessed by recording ambient vibrations simultaneously at several points. Resonance frequencies, damping ratios and modal shapes are derived using the Frequency Domain Decomposition method, a simple but efficient modal analysis method that allows decomposing modes, even close. These modal parameters, used in a linear multiple degree-of-freedom model, give the building response to weak ground motions. Additionally, an amplitude-frequency relationship has been derived from pseudo- dynamic tests of full-scale unreinforced masonry (URM) structures with reinforced concrete (RC) slabs made by ELSA laboratory in Ispra (Italy). This relationship is used in the MDOF model that can then be turned into a non-linear elastic model that should be able to well represent maximal response until moderate damage. Then, a 164 ground motions database selected in the European database is used to compute fragility curves, assuming inter-story (I-S) drift limits extracted from the literature and laboratory tests. In this study, two URM buildings with RC slabs, typical of Swiss construction between 1940 and 1960 located in Visp (Valais) have been tested using full-scale ambient vibrations. A simple non-linear elastic model based on these tests and on laboratory pseudo-dynamic tests is built and used to compute fragility curves from slight to severe damage. The results are compared with displacement-based computations and numerical models.

P. Ricci, G. Verderame, G. Manfredi, M. Pollino, F. Borfecchia. 2011. ***Multilevel approach to large scale seismic vulnerability assessment of reinforced concrete buildings***. XIV Convegno di Ingegneria Sismica - ANIDIS, Bari, Italy.

In this paper, a seismic vulnerability assessment at large scale is described, within the SIMURAI project. A field survey was carried out in order to gather detailed information about geometric characteristics, structural typology and age of construction of each single building. An airborne Remote Sensing (RS) mission was also carried out over the municipality of Avellino, providing a detailed estimate of 3D geometric parameters of buildings through a quite fast and easy to apply methodology integrating active LIDAR technology, aerophotogrammetry and GIS techniques. An analytical seismic vulnerability assessment procedure for Reinforced Concrete buildings is illustrated and applied to the building stock considering (i) field survey data (assumed as a reference) and (ii) LIDAR data combined with census data as alternative sources of information, according to a multilevel approach. A comparison between the obtained results highlights an acceptable scatter when data provided by RS techniques are used.

Flesch R. 2007. ***In-situ assessment, monitoring and typification of buildings and infrastructures***. Lessloss (Risk Mitigation for Earthquakes and Landslides), Sub-Project 5, Deliverable D19, D19A, D20, D20a, European manual for in-situ assessment of important existing structures, IUSS Press, Pavia.

LESSLOSS SP5 is on In-situ Assessment, Monitoring and Typification of Buildings and Infrastructure. It focuses on innovative and state of the art methods for the assessment of the following important existing structures:

- Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, telecommunication facilities, etc. (importance class IV according to EN1998-1:2005)
- Bridges of critical importance for maintaining communications, especially in the immediate post-earthquake period, bridges where failure is associated with a large number of probable fatalities and major bridges, where a design life greater than normal is required (importance class III according to EN 1998-2:2005)
- Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions, etc. (importance class III according to EN1998-1:2005)
- Industrial facilities, where secondary risks exist, e.g. the risk of release of toxic and/ or explosive materials
- Cultural heritage

It is of high importance, that these structures remain greatly undamaged and serviceable. If necessary, their earthquake resistance has to be increased based on the results of assessment. Normally the highest degree of knowledge is necessary to afford safety assessments for such structures. It is attained by setting a detailed, often 3D, structural model, based on careful surveys updated by using measured dynamic properties. It is one of the basic ideas of SP5 to integrate experimental techniques into the assessment procedure, which is an advantage one has in the case of existing buildings.

If such investigations are carried out in the pre-earthquake phase, measures for seismic upgrading can be undertaken in due time. In the post-earthquake phase these investigations enable the determination of the remaining safety and the serviceability, especially if mathematical models representing the previous status of the structure were elaborated before, which may in general already be characterized by damage). Hence, the task damage detection is crucial within LESSLOSS/ SP5.

The most important product of SP5 is the European Manual for in-situ Assessment of the Earthquake Resistance of Important Existing Structures. The manual provides also an overlook over literature useful for practical application. Many criteria and tools are already existing, but too less evident and therefore not accessible to the designers/ practitioners.

### *6.2 Literature Overview*

Aktan A.E., T. Kijewski-Correa, F. N. Catbas. 2011. *Structural identification of constructed facilities; approaches, methods and technologies for effective practice of st-id*. State of the Art report by ASCE & SEI (Committee on Structural Identification of Constructed Systems).

Antonioni G., G. Spadoni, V. Cozzani. 2007. *A methodology for the quantitative risk assessment of major accidents triggered by seismic events*. Journal of Hazardous Materials, 147(1-2): 48-59.

Ashley C. 1978. *Proposed international standards concerning vibrations in buildings*. Proceedings of symposium on instrumentation for ground vibration and earthquakes. Institution of Civil Engineers, London, 153-162.

Bachmann H., W. Ammann. 1987. *Vibrations in structures induced by man and machines*. IABSE Structural Engineering Documents, Zürich.

Bermúdez C., A. Barbat, L. Pujades, J. González-Drigo. 2008. *Seismic vulnerability and fragility of steel buildings*. Proceedings of 14th World Conference on Earthquake Engineering, Beijing, China.

Bernardini G., G. De Pasquale, A. Bicci, M. Marra, F. Coppi, P. Ricci. 2007. *Microwave interferometer for ambient vibration measurement on civil engineering structures: 1. Principles of the radar technique and laboratory tests.* Proceedings of Experimental Vibration Analysis for Civil Engineering Structures, Porto, Portugal.

Bernardini G., G. De Pasquale, N. Gallino, C. Gentile. 2007. *Microwave interferometer for ambient vibration measurement on civil engineering structures: 2. Application to full-scale bridges*. Proceedings of Experimental Vibration Analysis for Civil Engineering Structures, Porto, Portugal.

Biondi S., E. Candigliota. 2008. *In situ tests for seismic assessment of R.C*. structures. Proceedings of 14th World Conference on Earthquake Engineering, Beijing, China.

Biondi S., E. Candigliota. 2008. Non-destructive tests for existing R.C. structures assessment. Proceedings of the International FIB Symposium: Tailor Made Concrete Structures: New Solutions for Our Society, Amsterdam, Netherlands.

Boller C., F.K. Chang, Y. Fujino. 2009. *Encyclopedia of structural health monitoring*. John Wiley & Sons, New York, NY.

Borfecchia F., M. Pollino, L. De Cecco, A. Lugari, S. Martini, L. La Porta, E. Ristoratore, C. Pascale. 2010. *Active and passive remote sensing for supporting the evaluation of the urban seismic vulnerability*. Italian Journal of Remote Sensing, 42(3): 129-141.

Borzi B., R. Pinho, H. Crowley. 2008. *Simplified pushover-based vulnerability analysis for large-scale assessment of RC buildings*. Engineering Structures, 30: 804-820.

Brownjohn J.M.W. 2007. *Structural health monitoring of civil infrastructure*. Philosophical Transactions of the Royal Society A, 365(1851): 589-622.

Brownjohn J.M.W., A. Pavic, P. Reynolds. 2008. *Advances in dynamic testing methods for in-situ civil engineering structures*. Proceedings of 7th European Conference on Structural Dynamics, Southampton, United Kingdom.

Caprili S., L. Nardini, W. Salvatore, G.F. Bortone, N. Signorini. 2010. *Evaluation of seismic vulnerability of a complex RC existing building*. Proceedings of 14th European Conference on Earthquake Engineering, Ohrid, Macedonia.

Caverson R.G., P. Waldron, M.S. Williams. 1994. *Review of vibration guidelines for suspended concrete slabs*. Canadian Journal of Civil Engineering, 21(6): 931-938.

Deraemaeker A., K. Worden. 2010. *New trends in vibration based structural health monitoring*. CISM Lecture Notes Vol. 520, Springer, Vienna, Austria.

Diaz-López O., L. Esteva, J. Garcia-Pérez. 2010. *Practical tools for the estimation of the seismic vulnerability of buildings*. Proceedings of 14th European Conference on Earthquake Engineering, Ohrid, Macedonia.

Duco F., J. Faye, S. Caperaa, E. Reubrez. 2011. *Seismic vulnerability assessment using the instrumentation of an existing building*. Key Engineering Materials, Volume 482: 79-87.

Flesch R. 2007. *In-situ assessment, monitoring and typification of buildings and infrastructures*. Lessloss (Risk Mitigation for Earthquakes and Landslides), Sub-Project 5, Deliverable D19, D19A, D20, D20a, European manual for in-situ assessment of important existing structures, IUSS Press, Pavia.

Gehl P., S. Sy, D. Seyedi. 2011. *Developing fragility surfaces for more accurate seismic vulnerability assessment of masonry buildings*. Proceedings of Computational Methods in Structural Dynamics and Earthquake Engineering, Corfu, Greece.

Gkimousis I., V. Koumousis. 2011. Fragility assessment of RC frames collapse capacity. Proceedings of Computational Methods in Structural Dynamics and Earthquake Engineering, Corfu, Greece.

Guéguen P., V. Jolivet, C. Michel, A.-S. Schveitzer. 2010. *Comparison of velocimeter and coherent lidar measurements for building frequency assessment*. Bulletin of Earthquake Engineering, 8(2):327-338.

Hans S., C. Boutin, C. Chesnais. 2008. *How far in-situ measurements may help to assess building vulnerability?* Proceedings of 14th World Conference on Earthquake Engineering, Beijing, China.

International Organization for Standardization. 2010. *ISO 4866:2010 Mechanical vibration and shock - Vibration of buildings - Guidelines for the measurement of vibrations and evaluation of their effects on building*.

Kappos A., G. Panagopoulos, C. Panagiotopoulos, G. Penelis. 2006. *A hybrid method for the vulnerability assessment of R/C and URM buildings*. Bulletin of Earthquake Engineering, 4:391-413.

Kircil M., Z. Polat. 2006. *Fragility analysis of mid-rise R/C frame buildings*. Engineering Structures, 28(9): 1335-1345.

Koduru S.D., T. Haukaas. 2010. *Probabilistic Seismic Loss Assessment of a Vancouver High-Rise Building*. Journal of Structural Engineering, 136(3): 235-245.

Koutsourelakis P.S. 2010. *Assessing structural vulnerability against earthquakes using mult-dimensional fragility surfaces: A Bayesian framework*. Probabilistic Engineering Mechanics, 25(1): 49-60.

Kühn B., M. Lukić, A. Nussbaumer, H.P. Günther, R. Helmerich, S. Herion, M.H. Kolstein, S. Walbridge, B. Androic, O. Dijkstra, Ö. Bucak. 2008. *Assessment of existing steel structures: recommendations for estimation of remaining fatigue life*. Stahlbau, 77: 595-607.

Magliulo G., G. Fabbrocino, G. Manfredi. 2008. *Seismic assessment of existing precast industrial buildings using static and dynamic nonlinear analyses*. Engineering Structures, 30: 2580-2588.

Michel C., P. Guéguen, P.Y. Bard. 2008. *Dynamic parameters of structures extracted from ambient vibration measurements: An aid for the seismic vulnerability assessment of existing buildings in moderate seismic hazard regions*. Soil Dynamics and Earthquake Engineering, 28(8):593-604.

Michel C., E. Lattion, M. Oropeza, P. Lestuzzi. 2009. *Vulnerability assessment of existing masonry buildings in moderate seismicity areas using experimental techniques*. 2009 Asian-Pacific Network of Centers for Earthquake Engineering Research (ANCER) Workshop, Urbana, Illinois, United States.

Michel C., P. Guéguen, P. Lestuzzi, P.-Y. Bard. 2010. *Comparison between seismic vulnerability models and experimental dynamic properties of existing buildings in France*. Bulletin of Earthquake Engineering, 8(6): 1295-1307.

Michel C., P. Guéguen, M. Causse. 2011. *Seismic vulnerability assessment to slight damage based on experimental modal parameters*. Earthquake Engineering and Structural Dynamics, online first, doi: 10.1002/eqe.1119.

Mistler M., D. Heiland. 2009. *Artificial excitation methods for dynamic tests in buildings*. Proceedings of Experimental Vibration Analysis for Civil Engineering Structures, Wroclaw, Poland.

Miyashita T., H. Ishii, K. Kubota, Y. Fujino. 2007. *Advanced vibration measurement system using laser Doppler vibrometers for structural monitoring*. Proceedings of Experimental Vibration Analysis for Civil Engineering Structures, Porto, Portugal.

Moschonas I., A. Kappos. 2011. *Generalized fragility curves for bearing-supported skew bridges, for arbitrary angle of incidence of the seismic action*. Proceedings of Computational Methods in Structural Dynamics and Earthquake Engineering, Corfu, Greece.

Oliveira C. S., A. Roca, X. Goula. 2006. *Assessing and managing earthquake risk: geo-scientific and engineering knowledge for earthquake risk mitigation: developments, tools, techniques*. Vol. 2, Springer.

Oropeza M., C. Michel, P. Lestuzzi. 2010. *A simplified analytical methodology for fragility curves estimation in existing buildings*. Proceedings of 14th European Conference on Earthquake Engineering, Ohrid, Macedonia.

Pagnini L., R. Vicente, S. Lagomarsino, H. Varum. 2008. *A mechanical method for the vulnerability assessment of masonry buildings*. Proceedings of 14th World Conference on Earthquake Engineering, Beijing, China.

Park J., P. Towashiraporn, J.I. Craig, B.J. Goodno. 2008. *Analytical fragility assessment of a low-rise URM building*. Proceedings of 14th World Conference on Earthquake Engineering, Beijing, China.

Paultre P., J. Proulx. 1997. *Dynamic testing of large-scale structures*. Structural Engineering International, 7(1): 29-34.

Pinto P.E., P. Franchin. 2010. *Issues in the Upgrade of Italian Highway Structures*. Journal of Earthquake Engineering, 14(8): 1221-1252.

Pitilakis et al. 2011. *Fragility functions for common RC building types in Europe*. SYNER-G, Work Package 3, Deliverable D3.1.

Pitilakis et al. 2011. *Fragility functions for common masonry building types in Europe*. SYNER-G, Work Package 3, Deliverable D3.2.

Pottatheere P., P. Renault. 2008. *Seismic vulnerability assessment of skew bridges*. Proceedings of 14th World Conference on Earthquake Engineering, Beijing, China.

Reynders E., M. Schevenels, G. De Roeck. 2011. *MACEC: A Matlab toolbox for experimental and operational analysis*. Report BWM-2011-01.

Ricci P., G. Verderame, G. Manfredi, M. Pollino, F. Borfecchia. 2011. *Multilevel approach to large scale seismic vulnerability assessment of reinforced concrete buildings*. XIV Convegno di Ingegneria Sismica - ANIDIS, Bari, Italy.

Rota M., A. Penna, G. Magenes. 2010. *A methodology for deriving analytical fragility curves for masonry buildings based on stochastic nonlinear analyses*. Engineering Structures, 32: 1312-1323.

Sohn H., C.R. Farra, F.M. Hemez, D.D. Shunk, D.W. Stinemates, B.R. Nadler, J.J. Czarnecki. 2004. *A review of structural health monitoring literature: 1996-2001*. Los Alamos National Laboratory Report, LA-13976-MS.

Smith A.L., S.J. Hicks, P.J. Devine. 2007. *Design of floors for vibration: A new approach*. SCI Publication Vol. 354, The Steel Construction Institute, Ascot, Berkshire.

The American Association of State Highway and Transportation Officials. 1998. *AASHTO LRFD bridge design specifications (SI Units)*. Second Edition.

Uhl T., A. Hanc, K. Mendrok, P. Kurowski, B. Peeters, E. Moya, H. Van der Auweraer. 2007. *Wireless sensor network for bridge vibration monitoring – design and results*. Proceedings of Experimental Vibration Analysis for Civil Engineering Structures, Porto, Portugal.

Umemura H., H. Takizawa. 1982. *Dynamic response of reinforced concrete buildings*. IABSE-AIPC-IVBH, Zürich.

Vargas D.R. 2008. *Seismic vulnerability evaluation of RC urban bridges*. 14th World Conference on Earthquake Engineering, Beijing, China.

Viscardi M., G. Isernia, F. Amoroso, L. Lecce. 2007. *A long term monitoring system for vibration control of civil structures*. Proceedings of Experimental Vibration Analysis for Civil Engineering Structures, Porto, Portugal.

Wenzel H., D. Pichler. 2005. *Ambient vibration monitoring*. John Wiley & Sons, The Atrium Southern Gate, Chichester, West Sussex.

Willford M.R., P. Young. 2007. *A design guide for footfall induced vibration of structures*. The Concrete Centre, Camberley.

Yi J.-H., D. Kim, M. Q. Feng. 2009. *Periodic seismic performance evaluation of highway bridges using structural health monitoring system*. Structural Engineering and Mechanics, 31(5): 527-544.

# 7 References


Apaydin N. 2002. *Seismic Analysis of Fatih Sultan Mehmet Suspension Bridge*. PhD Dissertation Department of Earthquake Engineering, Boğaziçi University, Istanbul, Turkey.

Brincker R., P. Andersen, N.-J. Jacobsen. 2007. *Automated frequency domain decomposition for operational modal analysis*. In Proceedings of the 25th International Modal Analysis Conference, Orlando, United States.

Caetano E., F. Magalhães, Á. Cunha, O. Flamand, G. Grillaud. 2007. *Comparison of stochastic identification methods applied to the natural response of Millau Viaduct*. In Proceedings of EVACES2007, Experimental Vibrations of Civil Engineering Structures, Porto, Portugal.

Çatbaş F.N., T. Kijewski-Correa, A.E. Aktan. 2011. *Structural Identification (St-Id) of Constructed Facilities*. ASCE SEI Committee on Structural Identification of Constructed Systems, American Society of Civil Engineers (ASCE), Structural Engineering Institute (SEI).

Ceriotti M., L. Mottola, G.P. Picco, A.L. Murphy, S. Gună, M. Corrà, M. Pozzi, D. Zonta, P. Zanon. 2009. *Monitoring heritage buildings with wireless sensor networks: The Torre Aquila deployment*. In Proceedings of the 2009 International Conference on Information Processing in Sensor Networks, San Francisco, United States.

Cunha Á, F. Magalhães, E. Caetano. 2006. *Output-only modal identification of Luiz I bridge before and after rehabilitation*. Proceedings of ISMA2006 International Conference on Noise and Vibration Engineering, Leuven, Belgium, pp. 1165–1178.

Doebling S.W., C.R. Farrar, M.B. Prime, D.W. Shevitz. 1996. *Damage Identification and Health Monitoring of Structural and Mechanical Systems from Changes in Their Vibration Characteristics: A Literature Review*. Los Alamos National Laboratory, LA-13070-MS.

De Roeck G., G. Degrande, G. Lombaert, G. Müller. 2011. *Proceedings of the 8th International Conference on Structural Dynamics, EURODYN2011*, Leuven, Belgium.

De Roeck G., E. Reynders, D. Degrauwe. 2008. *Variance of extracted modal parameters and subsequent damage identification*. In Proceedings of the 7th European Conference on Structural Dynamics: Eurodyn 2008, Southampton, UK.

Deraemaeker A., E. Reynders, G. De Roeck, J. Kullaa. 2008. *Vibration based Structural Health Monitoring using output-only measurements under changing environment*. Mechanical Systems and Signal Processing, 22(1): 34–56.

Dumanoglu A.A., Brownjohn J.M.W., Severn R.T. 1992. *Seismic analysis of the Fatih Sultan Mehmet (Second Bosporus) Suspension Bridge*. Earthquake Engineering and Structural Dynamics, 21:881–906.

Eucentre. 2011. *Prove d'identificazione dinamica del tunnel di collegamento con struttura metallica dell'Aerostazione di Malpensa (VA)*. Protocollo 2011.LAB.019 EUC 081/2011U.

Eucentre. 2008. *Prove d'identificazione dinamica del santuario di Vicoforte di Mondovì (CN)*. Protocollo 2008.005.LAB EUC 35/2008U.

Friedl H., M. Ralbovsky, R. Flesch. 2009. *Simulationsverbesserung durch gezielte dynamische In-situ Versuche für Boden und Bauwerk*. 3. VDI-Tagung Baudynamik; ISBN: 978-3-18-092063-4 / ISSN: 0083-5560; VDI Wissensforum GmbH, 2063: 133-147.

Friedl H., M. Ralbovsky, W. Köllner, A. Lechner. 2007. *Seismic analysis of the reactor building Seibersdorf*. Technical Report.

Gazetas G. 1983. *Analysis of machine foundation vibrations: state of the art*. Soil Dynamics and Earthquake Engineering, 2(1): 2-42.

GeoSIG. 2011. *GMS-xx and GSR-IAx User Manual*. V8.

Goethals I., B. De Moor. 2002. *Model reduction and energy analysis as a tool to detect spurious modes*. Proceedings of ISMA2002 International Conference on Noise and Vibration Engineering, Leuven, Belgium, pp. 1307–1314.

He L., E. Reynders, T.-Y. Hsu, G. De Roeck. 2011. *Analysis of dynamic coupling between spans of two multi-span bridges using ambient vibration measurements*. The 8th International Conference on Structural Dynamics, Leuven, Belgium.

Katsikaridis T. 2011. *Study on Identification of Dynamic Characteristics of Structures By Ambient Vibration Testing: Case Studies on the Bosporus Suspension Bridge and Two-Typical Building Structures*. MSc Thesis, Bogazici University, KOERI, Istanbul, Turkey.

Kausel E., J.M. Roesset, J.T. Christian. 1976. *Nonlinear behavior in soil–structure interaction*. Journal of Geotechnical Engineering Division – ASCE, 102(12): 1159–1178.

Kaya, Y. 2009. *Tools and techniques for real time modal identification*. PhD Dissertation, Earthquake Engineering Department, Boğaziçi University, Istanbul, Turkey.

Lei Y., A.S. Kiremidjian, K.K. Nair, J.P. Lynch, K.H. Law. 2005. *Algorithms for time synchronization of wireless structural monitoring sensors*. Earthquake Engineering & Structural Dynamics, 34(6): 555–573.

Ljung L. 1999. *System identification: theory for the user*. Prentice Hall, Upper Saddle River, NJ, Second Edition.

Lynch J.P., K.J. Loh. 2006. *A summary review of wireless sensors and sensor networks for structural health monitoring*. Shock and Vibration Digest, 38(2): 91–128.

Lynch J.P. 2002. *Decentralization of wireless monitoring and control technologies for smart civil structures*. Technical Report Report No. 140, Stanford University, John A. Blume Engineering Center.

Mylonakis G., S. Nikolaou, G. Gazetas. 2006. *Footings under seismic loading: Analysis and design issues with emphasis on bridge foundations*. Soil Dynamics and Earthquake Engineering, 26(9): 824-853.

Pakzad S.N., G.L. Fenves, S. Kim, D. Culler. 2008. *Design and implementation of scalable wireless sensor network for structural monitoring*. Journal of Infrastructure Systems, 14(1): 89–101.

Pavic A., J.M.W. Brownjohn, P. Reynolds. 2008a. *Typical specifications for long-term vibration monitoring of tall buildings*. Technical report, Department of Civil & Structural Engineering, University of Sheffield.

Pavic A., J.M.W. Brownjohn, P. Reynolds. 2008b. *Typical specifications for short-term ambient vibration survey (AVS) of tall buildings*. Technical report, Department of Civil & Structural Engineering, University of Sheffield.

Pecker A. 1984. *Dynamique des Sols*. Presses Ponts et Chaussees, Paris, France.

Peeters B., G. De Roeck. 2001a. *One-year monitoring of the Z24-bridge: environmental effects versus damage events*. Earthquake Engineering and Structural Dynamics, 30: 149–171.

Peeters B., G. De Roeck. 2001b. *Stochastic system identification for operational modal analysis: A review*. ASME Journal of Dynamic Systems, Measurement, and Control, 123(4): 659–667.

Peeters B., H. Van der Auweraer, P. Guillaume, J. Leuridan. 2004. *The PolyMAX frequency-domain method: a new standard for modal parameter estimation?* Shock and Vibration, 11(3–4): 395–409.

Peeters B., G. De Roeck. 1999. *Reference-based stochastic subspace identification for output-only modal analysis*. Mechanical Systems and Signal Processing, 13(6):855–878.

Picozzi M., C. Milkereit, C. Zulfikar. 2009. *Wireless technologies for the monitoring of strategic civil infrastructures: an ambient vibration test on the Fatih Sultan Mehmet Suspension Bridge in Istanbul, Turkey*. Springer Science & Business Media B.V. 2009.

Ralbovsky M., H. Friedl, S. Wittmann. 2011. Dynamic Behavior of UHPFRC Arch Bridge Wild. Proceedings of Experimental Vibrational Analysis of Civil Engineering Structures Conference, pp.693-700, Varenna, Italy.

Reynders E., M. Schevenels, G. De Roeck. 2011a. *MACEC 3.2: a MATLAB toolbox for experimental and operational modal analysis*. Report BWM-2011-01, Department of Civil Engineering, K.U.Leuven.

Reynders E., J. Houbrechts, G. De Roeck. 2011b. *Automated interpretation of stabilization diagrams*. Modal analysis topics, Proceedings of the 29th International Modal Analysis Conference Series, Jacksonville, United States, 3: 189-201.

Reynders E., G. De Roeck. 2010. *Subspace identification for operational modal analysis*. New trends in vibration based structural health monitoring, Springer, Vienna, Austria, pp. 55–106.

Reynders E. 2009. *System identification and modal analysis in structural mechanics*. PhD thesis, Department of Civil Engineering, K.U.Leuven.

Reynders E., R. Pintelon, G. De Roeck. 2008a. *Uncertainty bounds on modal parameters obtained from Stochastic Subspace Identification*. Mechanical Systems and Signal Processing, 22(4): 948-969.

Reynders E., G. De Roeck. 2008b. *Reference-based combined deterministic-stochastic subspace identification for experimental and operational modal analysis*. Mechanical Systems and Signal Processing, 22(3):617–637.

Shih C.Y., Y.G. Tsuei, R.J. Allemang, D. Brown. 1988. *Complex mode indicator function and its applications to spatial domain parameter estimation*. Mechanical Systems and Signal Processing, 2(4):367–377.

Smith A.L., S.J. Hicks, P.J. Devine. 2009. *Design of floors for vibration: A new approach (SCI Publication P354)*. The Steel Construction Institute, Ascot, Berkshire.

Sohn H., C.R. Farrar, F.M. Hemez, D.D. Shunk, D.W. Stinemates, B.R. Nadler, J.J. Czarnecki. 2004. *A review of structural health monitoring literature: 1996-2001*. Los Alamos National Laboratory, LA-13976-MS.

Spencer B.F., M.E. Ruiz-Sandoval, N. Kurata. 2004. *Smart sensing technology: Opportunities and challenges*. Journal of Structural Control and Health Monitoring, 11(4):349–368.

Stewart J.P., G.L. Fenves, R.B. Seed. 1999a. *Seismic soil–structure interaction in buildings: Analytical methods*. Journal of Geotechnical and Geoenvironmental Engineering, 125(1): 26-37.

# Appendix A



There are 65 questions in this survey

## Generall

**1 [Header generall 1.1 ]**

**Questionnaire on Application and Performance of current Approaches in Field Testing for Earthquake Engineering**

**2 [Introduction]**

**Please fill in this questionnaire at your earliest convenience.**

***All information given in this questionnaire will be confidential. No information will be passed onto any third parties without explicit permission from you.***

**Questions witch are marked with an * are mandatory questions.**

**Compatibel webbrowser: Tested for webbrowser IE 6.0 or higher and Mozilla Firefox.**

---

**INTRODUCTION**

***For the purposes of this research, field testing is defined as the complete range of activities in this topic, like building or bridge structural response/performance studies, Soil-foundation-structure interaction (SFSI) studies or response/performance studies for geo-structures or soil deposits.***

**The aim of this questionnaire is to collect existing knowledge about field testing equipment and techniques for geotechnical and structural earthquake engineering experimentation.**

**THE ROLE OF FIELD TESTING METHODS FOR STRUCTURES**

**In situ investigations are in general necessary not only for earthquake assessment, but also for health monitoring and evaluation of the existing condition of the structure.**

**If you want more Information Please click yes.**

Please choose **all** that apply:

☐ Yes

**3 [Introduction EX]**

**Field testing:**

---

**Understanding structural response to earthquake loading is essential for the design of new structures and the assessment and seismic upgrading of existing structures. Further the prediction of soil liquefaction and subsidence, lateral spreading, and site amplification are based on small-strain laboratory and field testing, which does not duplicate the nonlinear strain levels that occur during major earthquakes. For analyzing the response of near-surface geological layers to earthquake loading mobile, large-scale field equipment can be used to produce dynamic motion in the ground comparable to that produced during earthquakes, so the loading conditions that structures are designed for.**

**Field experiments will significantly enhance the fundamental knowledge of earthquake effects associated with the behavior of structures and geological layers, thereby reducing loss-of-life and economic losses from future earthquakes.**

**In principle it is possible to measure the following physical parameters:**

- **Strains**
- **Displacements, velocities, accelerations, (transverse and rotational components, even if measurement of rotational components is less easy)**

**From measured time histories the dynamic properties (Modal Parameters) can be obtained.**

**Dynamic field tests or in-situ testing methods are necessary for the evaluation of the dynamic behaviour (modal parameters) and the actual condition of existing structures. In-situ measurements are recommended in order to verify the numerical models and to increase the reliability of the numerical approaches. A first FE-model of the tested structure, which is elaborated on the basis of the design documents can be fitted to measured results by an optimisation approach. This procedure is called "model updating". Dynamic in-situ measurements are also recommended for the verification of the effects of structural changes and strengthening applied to buildings.**

**By means of experimental modal analysis the identification of the modal parameters (natural frequencies, mode shapes, and damping ratios) of a structure from input and/or output measurements is possible. For this purpose, the mode shapes of a structure must be excited measurably. In the case of input/output measurements also the frequency dependent impedance (force vibration velocity ratio) is obtained. The modal parameters are used to perform ongoing earthquake analysis or assessment. Since the updated model is based on measured results, it represents in a realistic manner the behaviour of the structure during the starting phase of a seismic event. Using this model for an earthquake analysis it is possible to forecast and therefore to point out the weak spots in structural members, if any.**

**To measure the vibration response of a structure, a sufficient excitation should be available. This excitation is basically possible in two different ways, on the one hand by forced excitation and on the other by ambient excitation.**

**http://www.nera-eu.org/**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '2 [Introduction]' ( Please fill in this questionnaire at your earliest convenience. All information given in this questionnaire will be confidential. No information will be passed onto any third parties without explicit permission from you. Questions witch are marked with an * are mandatory questions. Compatibel webbrowser: Tested for webbrowser IE 6.0 or higher and Mozilla Firefox. INTRODUCTION For the purposes of this research, field testing is defined as the complete range of activities in this topic, like building or bridge structural response/performance studies, Soil-foundation-structure interaction (SFSI) studies or response/performance studies for geo-structures or soil deposits. The aim of this questionnaire is to collect existing knowledge about field testing equipment and techniques for geotechnical and structural earthquake engineering experimentation. THE ROLE OF FIELD TESTING METHODS FOR STRUCTURES In situ investigations are in general necessary not only for earthquake assessment, but also for health monitoring and evaluation of the existing condition of the structure. If you want more Information Please click yes. )

**4 [1]**

**Question 1**

**Have you used any measurement technologies with respect to field testing?**

**If YES, please continue.**

**If NO, please forward the email about this survey to others who may be knowledgeable about this subject.**

*

Please choose **only one** of the following:

- ○ Yes
- ○ No

**5 [personal]**

**Personal Details**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. )

Please write your answer(s) here:

| | |
|---|---|
| First Name: | |
| Last Name: | |
| Company: | |
| Position: | |

**6 [Country]Country: ***

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. )

Please choose **only one** of the following:

- ○ Afghanistan
- ○ Albania
- ○ Algeria
- ○ Andorra
- ○ Angola
- ○ Argentine
- ○ Armenia
- ○ Australia
- ○ Austria
- ○ Azerbaijan
- ○ Bahamas
- ○ Bahrain
- ○ Bangladesh
- ○ Barbados
- ○ Belarus
- ○ Belgium
- ○ Belize
- ○ Benin
- ○ Bhutan
- ○ Bolivia
- ○ Bosnia and Herzegovina
- ○ Botswana
- ○ Brazil
- ○ Bulgaria
- ○ Burkina Faso
- ○ Burma
- ○ Burundi
- ○ Cambodia
- ○ Cameroon
- ○ Canada
- ○ Cape Verde
- ○ Central African Republic

- Chad
- Chile
- China
- Colombia
- Comoros
- Congo
- Costa Rica
- Côte d'Ivoire
- Croatia
- Cuba
- Cyprus
- Czech Republic
- Denmark
- Djibouti
- Dominica
- Dominican Republic
- East Timor
- Ecuador
- Egypt
- El Salvador
- Equatorial Guinea
- Eritrea
- Estonia
- Ethiopia
- Fiji
- Finland
- France
- Gabon
- Gambia
- Georgia
- Germany
- Ghana
- Greece
- Grenada
- Guatemala
- Guinea
- Guinea-Bissau
- Guyana
- Haiti
- Honduras
- Hungary
- Iceland
- India
- Indonesia
- Iran
- Iraq
- Ireland
- Israel
- Italian
- Jamaica
- Japan
- Jordan
- Kazakhstan
- Kenya
- Kiribati
- North Korea
- South Korea
- Kuwait
- Kyrgyz
- Laos
- Latvia
- Lebanon
- Lesotho
- Liberia
- Libya
- Liechtenstein
- Lithuania
- Luxembourg

- Madagascar
- Malawi
- Malaysia
- Maldives
- Mali
- Malta
- Marshall Islands
- Mauritania
- Mauritius
- Mexico
- Micronesia
- Moldova
- Monaco
- Mongolia
- Montenegro
- Morocco
- Mozambique
- Namibia
- Nauru
- Nepal
- Netherlands
- New Zealand
- Nicaragua
- Niger
- Nigeria
- Norway
- Oman
- Pakistan
- Palau
- Palestine
- Panama
- Papua New Guinea
- Paraguay
- Peru
- Philippines
- Poland
- Portugal
- Qatar
- Romania
- Russia
- Rwanda
- Saint Kitts and Nevis
- Saint Lucia
- Saint Vincent and the Grenadines
- Samoa
- San Marino
- São Tomé and Principe
- Saudi Arabia
- Senegal
- Serbia
- Seychelles
- Sierra Leone
- Singapore
- Slovakia
- Slovenia
- Solomon Islands
- Somalia
- South Africa
- Spain
- Sri Lanka
- Sudan
- Suriname
- Swaziland
- Sweden
- Switzerland
- Syria
- Tajikistan
- Tanzania

- ○ The former Yugoslav Republic of Macedonia
- ○ Thailand
- ○ Togo
- ○ Tonga
- ○ Trinidad and Tobago
- ○ Tunisia
- ○ Turkey
- ○ Turkmenistan
- ○ Tuvalu
- ○ Uganda
- ○ Ukraine
- ○ United Arab Emirates
- ○ United Kingdom
- ○ United States
- ○ Uruguay
- ○ Uzbekistan
- ○ Vanuatu
- ○ Venezuela
- ○ Vietnam
- ○ Yemen
- ○ Zambia
- ○ Zimbabwe
- ○ Vatican City
- ○ Kosovo
- ○ Taiwan

**7 [2]**

**Question 2**

**Is your organisation private, public or both?**

**If both or other, please explain your answer below.**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. )

Please choose **all** that apply:

- ☐ Private
- ☐ Public body
- ☐ Research organisation (nonprofit)
- ☐ University
- ☐ Industry including small and medium enterprises
- ☐ Other: _____

**8 [3]**

**Question 3**

**What is the special area of work (within earthquake engineering) of your organisation mainly specialised in?**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. )

Please choose **all** that apply:

- ☐ Design of new structures
- ☐ Maintenance and repair
- ☐ Regulatory/standard
- ☐ Assessment and retrofitting of existing structures
- ☐ Construction/contracting
- ☐ Research & Development
- ☐ Other: _____

**9 [3A]**

**Question 3A**

**Please provide a short summary of the main services your organization offers in relation to EQ Engineering.**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. )

Please write your answer here:

**10 [4]**

**Question 4**

**What is the focus of your company, with respect to field testing and measurement technologies?**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. )

Please choose **at most** 1 answers:

- ☐ Infrastructure authority, looking to use measurement technologies
- ☐ Provider of measurement technologies
- ☐ Consultant, using measurement data as possible assessment of structures
- ☐ Consultant, addressing possible use of measurement technologies
- ☐ Other: _____

**11 [5]**

**Question 5**

**Select the type of application for which you haved used field testing technologies?**

*

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. )

Please choose **all** that apply:

- ☐ Condition monitoring for Life Cycle Assessment (e.g. vibration levels, etc)
- ☐ Obtaining modal parameters for updating approaches (Model reliability improvement)
- ☐ Structural Health Monitoring (Damage detection)
- ☐ Assessment/verification after retrofitting/strengthening
- ☐ Design verification (e.g. monitoring pre-stress forces, etc.)
- ☐ Pre and Post earthquake assessment
- ☐ Seismic early warning systems
- ☐ Alert systems for structural collapse

**12 [6]**

**Question 6**

**Which kind of structures do you investigate with in-situ field testing?**

*

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. )

Please choose **all** that apply:

- ☐ Bridges
- ☐ Buildings

**Section Bridges**

**13 [Header]**

**[SECTION BRIDGES]**

**[SECTION BRIDGES]**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Bridges' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? )

**14 [7]**

**Question 7**

**What is the monitoring rate?**

*

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Bridges' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? )

Please choose **all** that apply:

- ☐ Permanent monitoring
- ☐ Once-only monitoring
- ☐ Periodical monitoring

**15 [7A]**

**Question 7A**

**If Periodical monitoring :**

**Interval**

*

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Bridges' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? ) *and* Answer was 'Periodical monitoring' at question '14 [7]' ( Question 7 What is the monitoring rate? )

Please choose **only one** of the following:

- ○ annual
- ○ biennial
- ○ triennial
- ○ four-yearly
- ○ five-yearly
- ○ > 5 years interval

**16 [8]**

**Question 8**

**How do you design the measurement grid and the instrumentation before the actual field test?**

*

**Only answer this question if the following conditions are met:**

° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Bridges' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? )

Please choose **only one** of the following:

- ○ Based on previous experience with similar structures
- ○ Based on available structural analysis (finite element) results of the designed structure
- ○ No specific strategy, just distributing the sensors over the structure

**17 [9]**

**Question 9**

**How many bridges of the following types have you assessed with permanent field testing?**

**Please fill in the number of structures in the table, e.g. 3 Girder bridges, 2 arch bridges, etc.**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Bridges' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? ) *and* Answer was 'Permanent monitoring' at question '14 [7]' ( Question 7 What is the monitoring rate? )

| | Bridges with <5 sensors | Bridges with <10 sensors | Bridges with <20 sensors | Bridges with <30 sensors | Bridges with <40 sensors | Bridges with >40 sensors |
|---|---|---|---|---|---|---|
| Arch bridge | | | | | | |
| Cable-net bridge | | | | | | |
| Cable-stayed bridge | | | | | | |
| Covered bridge | | | | | | |
| Girder bridge | | | | | | |
| Hyperbolic paraboloid bridge | | | | | | |
| Movable bridge | | | | | | |
| Pontoon bridge | | | | | | |
| Rigid frame bridge | | | | | | |
| Stressed ribbon bridge | | | | | | |
| Suspension bridge | | | | | | |
| Trestle bridge | | | | | | |
| Truss bridge | | | | | | |

**18 [10]**

**Question 10**

**How many bridges of the following types have you assessed with only-once or periodical field testing?**

**Please fill in the number of structures in the table, e.g. 3 Girder bridges, 2 arch bridges, etc.**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Bridges' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? ) *and* Answer was 'Periodical monitoring' or 'Once-only monitoring' at question '14 [7]' ( Question 7 What is the monitoring rate? ) *and* Answer was 'Periodical monitoring' or 'Once-only monitoring' at question '14 [7]' ( Question 7 What is the monitoring rate? )

| | Bridges with <5 sensors | Bridges with <10 sensors | Bridges with <20 sensors | Bridges with <30 sensors | Bridges with <40 sensors | Bridges with >40 sensors |
|---|---|---|---|---|---|---|
| Arch bridge | | | | | | |
| Cable-net bridge | | | | | | |
| Cable-stayed bridge | | | | | | |
| Covered bridge | | | | | | |
| Girder bridge | | | | | | |
| Hyperbolic paraboloid bridge | | | | | | |
| Movable bridge | | | | | | |
| Pontoon bridge | | | | | | |
| Rigid frame bridge | | | | | | |
| Stressed ribbon bridge | | | | | | |
| Suspension bridge | | | | | | |
| Trestle bridge | | | | | | |
| Truss bridge | | | | | | |

**19 [11]**

**Question 11**

**Which parameters are monitored?**

*

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Bridges' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? )

Please choose **all** that apply:

- ☐ Acceleration
- ☐ Absolute displacements
- ☐ Relative displacements
- ☐ Crack size
- ☐ Velocity
- ☐ Strain
- ☐ Tilt
- ☐ Force
- ☐ Pressure
- ☐ Temperature
- ☐ Wind speed
- ☐ Humidity
- ☐ Other: _____

**20 [12]**

**Question 12**

**Which sensor technologies are in use?**

*

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Bridges' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? )

Please choose **all** that apply:

- ☐ Traditional sensing technologies (electric signals)
- ☐ Optical fiber technology
- ☐ Non-contact measurements (Microwave interferometry, Laser, etc.)
- ☐ Wireless sensors
- ☐ GPS
- ☐ other: _____

**21 [13]**

**Question 13**

**Which excitation do you use for field testing measurements?**

*

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Bridges' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? )

Please choose **all** that apply:

☐

Ambient vibration analysis

***(Ambient vibration analysis means the measurement of the structural responses trigged by natural (ambient) sources. In contrast to forced vibration no artificial excitation of the structure takes place. Therefore those structural vibrations are measured, which are excited by natural, the so-called "ambient" sources like wind, traffic of all kinds, microseismicity, etc)***

☐

Forced vibration analysis

***(For example oscillators or other dynamic excitation mechanisms are used as exciters by applying them in one or more defined excitation points simultaneously or one after the other. The reactions to this known excitation (input) are recorded by sensors (output).***

**22 [13A]**

**Question 13A**

**If you use forced vibration analysis, please specify:**

*

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Bridges' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? ) *and* Answer was '

Forced vibration analysis

***(For example oscillators or other dynamic excitation mechanisms are used as exciters by applying them in one or more defined excitation points simultaneously or one after the other. The reactions to this known excitation (input) are recorded by sensors (output).***

' at question '21 [13]' ( Question 13 Which excitation do you use for field testing measurements? )

Please choose **all** that apply:

- [ ] Impact hammer
- [ ] Drop-weight
- [ ] Snap-back, step relaxation or free vibration
- [ ] Rotating eccentric mass exciters
- [ ] Linear or reciprocating mass exciters
- [ ] Mobile vibration generators
- [ ] other: _____

**23 [14]**

**Question 14**

**Which structural parameters do you extract from measurement data?**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Bridges' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? )

Please choose **all** that apply:

- [ ] Modal parameters
- [ ] Static parameters
- [ ] Others: _____

**24 [14A]**

**Question 14A**

**If you use Modal parameters:**

**Only answer this question if the following conditions are met:**
° Answer was 'Modal parameters ' at question '23 [14]' ( Question 14 Which structural parameters do you extract from measurement data? )

Please choose **all** that apply:

- [ ] Natural frequency
- [ ] Mode shapes
- [ ] Damping

**25 [14B]**

**Question 14B**

**If you use Static parameters:**

**Only answer this question if the following conditions are met:**
° Answer was 'Static parameters' at question '23 [14]' ( Question 14 Which structural parameters do you extract from measurement data? )

Please choose **all** that apply:

- [ ] Influence Line
- [ ] Others: _____

**26 [15]**

**Question 15**

**Which analysis method for data processing do you mainly use?**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Bridges' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? )

Please choose **all** that apply:

- [ ] Frequency domain
- [ ] Time Domain

**27 [15A]**

**Question 15A**

**If you use Frequency domain:**

**Only answer this question if the following conditions are met:**
° Answer was 'Frequency domain' at question '26 [15]' ( Question 15 Which analysis method for data processing do you mainly use? )

Please choose **all** that apply:

- [ ] Peak Picking
- [ ] Frequency Domain Decomposition (FDD)
- [ ] Other: _____

**28 [15B]**

**Question 15B**

**If you use Time Domain:**

**Only answer this question if the following conditions are met:**
° Answer was 'Time Domain' at question '26 [15]' ( Question 15 Which analysis method for data processing do you mainly use? )

Please choose **all** that apply:

- ☐ Random Decrement
- ☐ Recursive Techniques (ARMA)
- ☐ Maximum Likelihood Methods
- ☐ Stochastic Subspace Identification Methods (SSI)
- ☐ Other: _____

**29 [TEXT B]**

**Please click on the "Next button" for starting the next section.**

**Only answer this question if the following conditions are met:**
° Answer was 'No' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. )

**30 [TEXT B 1.1]**

**Please click on the "Next button" for starting the next section.**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was NOT 'Buildings' or 'Bridges' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? ) *and* Answer was 'Buildings' or 'Bridges' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? )

**Section Buildings**

**31 [HEADER BUILDINGS 1.1]**

**[SECTION BUILDINGS]**

**[SECTION BUILDINGS]**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Buildings' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? )

**32 [16]**

**Question 16**

**What is the monitoring rate?**

*

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Buildings' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? )

Please choose **all** that apply:

- ☐ Permanent monitoring
- ☐ Once-only monitoring
- ☐ Periodical monitoring

**33 [16A]**

**Question 16A**

**If Periodical monitoring :**

**Interval**

*

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Buildings' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? ) *and* Answer was 'Periodical monitoring' at question '32 [16]' ( Question 16 What is the monitoring rate? )

Please choose **only one** of the following:

- ○ annual
- ○ biennial
- ○ triennial
- ○ four-yearly
- ○ five-yearly
- ○ > 5 years interval

**34 [17]**

**Question 17**

**How do you design the measurement grid and the instrumentation before the actual field test?**

*

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Buildings' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? )

Please choose **only one** of the following:

- ○ Based on previous experience with similar structures
- ○ Based on available structural analysis (finite element) results of the designed structure
- ○ No specific strategy, just distributing the sensors over the structure

**35 [18]**

**Question 18**

## Residential buildings:

**How many Residential buildings have you assed with permanent field testing?**

**Please fill in the number of structures, e.g.3 residential buildings, etc.**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Buildings' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? ) *and* Answer was 'Permanent monitoring' at question '32 [16]' ( Question 16 What is the monitoring rate? )

| | Buildings with <5 sensors | Buildings with <10 sensors | Buildings with <20 sensors | Buildings with <30 sensors | Buildings with <40 sensors | Buildings with >40 sensors |
|---|---|---|---|---|---|---|
| Steel, reinforced concrete; numer of stories ≤ 5 | | | | | | |
| Steel, reinforced concrete; numer of stories >5 | | | | | | |
| Masonry; numer of stories ≤ 5 | | | | | | |
| Masonry storey; numer of stories > 5 | | | | | | |
| Other | | | | | | |

**36 [19A]**

**Question 19A**

## Industrial buildings:

**How many Industrial buildings have you assessed with permanent field testing?**

**Please fill in the number of structures in the tabel, e.g. 5 industrial buildings, etc.**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Buildings' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? ) *and* Answer was 'Permanent monitoring' at question '32 [16]' ( Question 16 What is the monitoring rate? )

| | Buildings with <5 sensors | Buildings with <10 sensors | Buildings with <20 sensors | Buildings with <30 sensors | Buildings with <40 sensors | Buildings with >40 sensors |
|---|---|---|---|---|---|---|
| Steel, reinforced concrete; numer of stories ≤ 5 | | | | | | |
| Steel, reinforced concrete; numer of stories >5 | | | | | | |
| Masonry; numer of stories ≤ 5 | | | | | | |
| Masonry; numer of stories >5 | | | | | | |
| Other | | | | | | |

**37 [19B]**

**Question 19B**

## Life Line Structures:

**How many Life Line Structureshave you assessed with permanent field testing?**

**Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, etc. and buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Buildings' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? ) *and* Answer was 'Permanent monitoring' at question '32 [16]' ( Question 16 What is the monitoring rate? )

| | Buildings with <5 sensors | Buildings with <10 sensors | Buildings with <20 sensors | Buildings with <30 sensors | Buildings with <40 sensors | Buildings with >40 sensors |
|---|---|---|---|---|---|---|
| Steel, reinforced concrete; numer of stories ≤ 5 | | | | | | |
| Steel, reinforced concrete; numer of stories >5 | | | | | | |
| Masonry; numer of stories ≤ 5 | | | | | | |

| | Buildings with <5 sensors | Buildings with <10 sensors | Buildings with <20 sensors | Buildings with <30 sensors | Buildings with <40 sensors | Buildings with >40 sensors |
|---|---|---|---|---|---|---|
| Masonry; numer of stories >5 | | | | | | |
| Other | | | | | | |

**38 [19C]**

**Question 19C**

## Historical buildings:

**How many Historical buildings and monumentshave you assessed with permanent field testing?**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Buildings' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? ) *and* Answer was 'Permanent monitoring' at question '32 [16]' ( Question 16 What is the monitoring rate? )

| | Material | Height | Buildings with <10 sensors | Buildings with <20 sensors | Buildings with <30 sensors | Buildings with <40 sensors | Buildings with >40 sensors |
|---|---|---|---|---|---|---|---|
| #1 | | | | | | | |
| #2 | | | | | | | |
| #3 | | | | | | | |

**39 [20]**

**Question 20**

## Residential buildings:

**How many Residential buildings have you assessed with only-once and periodical field testing?**

**Please fill in the number of structures in the tabel, e.g. 5 residential buildings, etc.**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Buildings' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? ) *and* Answer was 'Once-only monitoring' or 'Periodical monitoring' at question '32 [16]' ( Question 16 What is the monitoring rate? ) *and* Answer was 'Once-only monitoring' or 'Periodical monitoring' at question '32 [16]' ( Question 16 What is the monitoring rate? )

| | Buildings with <5 measurement points | Buildings with <10 measurement points | Buildings with <20 measurement points | Buildings with <30 measurement points | Buildings with <40 measurement points | Buildings with >40 measurement points |
|---|---|---|---|---|---|---|
| Steel, reinforced concrete; numer of stories ≤ 5 | | | | | | |
| Steel, reinforced concrete; numer of stories > 5 | | | | | | |
| Masonry; numer of stories ≤ 5 | | | | | | |
| Masonry; numer of stories >5 | | | | | | |
| Other | | | | | | |

**40 [20B]**

**Question 20B**

## Industrial buidlings:

**How many Industrial buidlings have you assessed with only-once and periodical field testing?**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Buildings' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? ) *and* Answer was 'Once-only monitoring' or 'Periodical monitoring' at question '32 [16]' ( Question 16 What is the monitoring rate? ) *and* Answer was 'Once-only monitoring' or 'Periodical monitoring' at question '32 [16]' ( Question 16 What is the monitoring rate? )

| | Buildings with <5 measurement points | Buildings with <10 measurement points | Buildings with <20 measurement points | Buildings with <30 measurement points | Buildings with <40 measurement points | Buildings with >40 measurement points |
|---|---|---|---|---|---|---|
| Steel, reinforced concrete; numer of stories ≤ 5 | | | | | | |
| Steel, reinforced concrete; numer of stories > 5 | | | | | | |
| Masonry; numer of stories ≤ 5 | | | | | | |
| Masonry; numer of stories >5 | | | | | | |
| Other | | | | | | |

**41 [20C]**

**Question 20C**

## Life Line Structures:

**How many Life Line Structures have you assessed with only once and periodical field testing?**

**Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, etc.**

**and buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Buildings' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? ) *and* Answer was 'Once-only monitoring' or 'Periodical monitoring' at question '32 [16]' ( Question 16 What is the monitoring rate? ) *and* Answer was 'Once-only monitoring' or 'Periodical monitoring' at question '32 [16]' ( Question 16 What is the monitoring rate? )

| | Buildings with <5 measurement points | Buildings with <10 measurement points | Buildings with <20 measurement points | Buildings with <30 measurement points | Buildings with <40 measurement points | Buildings with >40 measurement points |
|---|---|---|---|---|---|---|
| Steel, reinforced concrete; numer of stories ≤ 5 | | | | | | |
| Steel, reinforced concrete; numer of stories >5 | | | | | | |
| Masonry; numer of stories ≤ 5 | | | | | | |
| Masonry; numer of stories >5 | | | | | | |
| Other | | | | | | |

**42 [20D]**

**Question 20D**

## Historical buildings:

**How many Historical buildings and monuments have you assessed with only once and periodical field testing?**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Buildings' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? ) *and* Answer was 'Once-only monitoring' or 'Periodical monitoring' at question '32 [16]' ( Question 16 What is the monitoring rate? ) *and* Answer was 'Once-only monitoring' or 'Periodical monitoring' at question '32 [16]' ( Question 16 What is the monitoring rate? )

| | Material | Height | Buildings with <5 measurement points | Buildings with <10 measurement points | Buildings with <20 measurement points | Buildings with <30 measurement points | Buildings with <40 measurement points |
|---|---|---|---|---|---|---|---|
| #1 | | | | | | | |
| #2 | | | | | | | |
| #3 | | | | | | | |

**43 [21]**

**Question 21**

**Which parameters are monitored?**

*

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Buildings' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? )

Please choose **all** that apply:

- ☐ Acceleration
- ☐ Absolute displacements
- ☐ Relative displacements
- ☐ Crack size
- ☐ Velocity
- ☐ Strain
- ☐ Tilt
- ☐ Force
- ☐ Pressure
- ☐ Temperature
- ☐ Wind speed
- ☐ Humidity
- ☐ other: _____

**44 [22]**

**Question 22**

**Which sensor technologies are in use?**

*

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Buildings' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? )

Please choose **all** that apply:

- ☐ Traditional sensing technologies (electric signals)
- ☐ Optical fiber technology
- ☐ Non-contact measurements (Microwave interferometry, Laser, etc.)
- ☐ Wireless sensors
- ☐ GPS

☐ other: _____

**45 [23]**

**Question 23**

**Which excitation do you use for field testing measurements?**

*

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Buildings' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? )

Please choose **all** that apply:

☐

Ambient vibration analysis

***(Ambient vibration analysis means the measurement of the structural responses trigged by natural (ambient) sources. In contrast to forced vibration no artificial excitation of the structure takes place. Therefore those structural vibrations are measured, which are permanently excited more or less intensively by natural, the so-called "ambient" sources like for example wind, traffic of all kinds, microseismicity, etc)***

☐

Forced vibration analysis

***(For example oscillators or other dynamic excitation mechanisms are used as exciters by applying them in one or more defined excitation points simultaneously or one after the other. The reactions to this known excitation (input) are recorded by sensors (output).***

**46 [23A]**

**Question 23A**

**If you use forced vibration analysis, please specify:**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Buildings' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? ) *and* Answer was '

Forced vibration analysis

***(For example oscillators or other dynamic excitation mechanisms are used as exciters by applying them in one or more defined excitation points simultaneously or one after the other. The reactions to this known excitation (input) are recorded by sensors (output).***

' at question '45 [23]' ( Question 23 Which excitation do you use for field testing measurements? )

Please choose **all** that apply:

☐ Impact hammer
☐ Drop-weight
☐ Snap-back, step relaxation or free vibration
☐ Rotating eccentric mass exciters
☐ Linear or reciprocating mass exciters
☐ Mobile vibration generators
☐ other: _____

**47 [24]**

**Question 24**

**Which structural parameters do you extract from measurement data?**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Buildings' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? )

Please choose **all** that apply:

☐ Modal parameters
☐ Static parameters
☐ Others: _____

**48 [24A]**

**Question 24A**

**If you use Modal parameters:**

**Only answer this question if the following conditions are met:**
° Answer was 'Modal parameters ' at question '47 [24]' ( Question 24 Which structural parameters do you extract from measurement data? )

Please choose **all** that apply:

☐ Natural frequencies
☐ Mode shapes
☐ Damping

**49 [24B]**

**Question 24B**

**If you use Static parameters:**

**Only answer this question if the following conditions are met:**
° Answer was 'Static parameters' at question '47 [24]' ( Question 24 Which structural parameters do you extract from measurement data? )

Please choose **all** that apply:

- [ ] Influence Line
- [ ] Others: _____

**50 [25]**

**Question 25**

**Which analysis method for data processing you mainly use?**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Buildings' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? )

Please choose **all** that apply:

- [ ] Frequency domain
- [ ] Time Domain

**51 [25A]**

**Question 25A**

**If you use Frequency domain:**

**Only answer this question if the following conditions are met:**
° Answer was 'Frequency domain' at question '50 [25]' ( Question 25 Which analysis method for data processing you mainly use? )

Please choose **all** that apply:

- [ ] Peak Picking
- [ ] Frequency Domain Decomposition (FDD)
- [ ] Other: _____

**52 [25B]**

**Question 25B**

**If you use Time Domain:**

**Only answer this question if the following conditions are met:**
° Answer was 'Time Domain' at question '50 [25]' ( Question 25 Which analysis method for data processing you mainly use? )

Please choose **all** that apply:

- [ ] Random Decrement
- [ ] Recursive Techniques (ARMA)
- [ ] Maximum Likelihood Methods
- [ ] Stochastic Subspace Identification Methods (SSI)
- [ ] Other: _____

**53 [TEXT C]**

**Please click on the "Next button" for starting the next section.**

**Only answer this question if the following conditions are met:**
° Answer was 'No' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. )

**54 [TEXT C1]**

**Please click on the "Next button" for starting the next section.**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Bridges' at question '12 [6]' ( Question 6 Which kind of structures do you investigate with in-situ field testing? )

## Finally

**55 [26]**

**Question 26**

**For field testing, what types of sensors and how many in total are used in your company?**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. )

Please choose the appropriate response for each item:

| | 0 | <5 | <10 | <20 | <30 | <40 | >40 |
|---|---|---|---|---|---|---|---|
| Vibration sensors | ○ | ○ | ○ | ○ | ○ | ○ | ○ |
| Geophones | ○ | ○ | ○ | ○ | ○ | ○ | ○ |
| Displacment transducers | ○ | ○ | ○ | ○ | ○ | ○ | ○ |
| Temperature sensors | ○ | ○ | ○ | ○ | ○ | ○ | ○ |
| Strain gauges | ○ | ○ | ○ | ○ | ○ | ○ | ○ |
| Load cells | ○ | ○ | ○ | ○ | ○ | ○ | ○ |
| Inclinometer | ○ | ○ | ○ | ○ | ○ | ○ | ○ |
| Acoustic sensors | ○ | ○ | ○ | ○ | ○ | ○ | ○ |
| Ultrasonic sensors | ○ | ○ | ○ | ○ | ○ | ○ | ○ |
| Wind speed sensors | ○ | ○ | ○ | ○ | ○ | ○ | ○ |
| Humidity sensors | ○ | ○ | ○ | ○ | ○ | ○ | ○ |
| others | ○ | ○ | ○ | ○ | ○ | ○ | ○ |

**56 [27]**

**Question 27**

**Are there national standards or guidelines in your country, that impose field testing as a tool for Earthquake Engineering?**

*

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. )

Please choose **only one** of the following:

○ Yes
○ Under development
○ No

**57 [27A]**

**Question 27A**

**If yes, please give the reference:**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Yes' at question '56 [27]' ( Question 27 Are there national standards or guidelines in your country, that impose field testing as a tool for Earthquake Engineering? )

Please choose **all** that apply:

☐ National standards regarding field testing for bridges
☐ National standards regarding field testing for buildings
☐ National standards regarding field testing for other structures
☐ Other: _____

**58 [27B]**

**Question 27B**

**National standards regarding field testing for bridges:**

*

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'National standards regarding field testing for bridges' at question '57 [27A]' ( Question 27A If yes, please give the reference: )

Please write your answer(s) here:

Issued by: _____
Enforcement Year: _____

**59 [27C]**

**Question 27C**

**National standards regarding field testing for buildings:**

*

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'National standards regarding field testing for buildings ' at question '57 [27A]' ( Question 27A If yes, please give the reference: )

Please write your answer(s) here:

Issued by:

Enforcement Year:

**60 [27D]**

**Question 27D**

**National standards regarding field testing for other structures:**

*

**Only answer this question if the following conditions are met:**
° Answer was 'National standards regarding field testing for other structures' at question '57 [27A]' ( Question 27A If yes, please give the reference: )

Please write your answer(s) here:

Other structure:

Issued by:

Enforcement year:

**61 [27E]**

**Question 27E**

**If the development of guidelines is still under progress, please briefly describe the content and goal of the efforts.**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'Under development' at question '56 [27]' ( Question 27 Are there national standards or guidelines in your country, that impose field testing as a tool for Earthquake Engineering? )

Please write your answer here:

**62 [27F]**

**Question 27F**

**If not, are there voluntary standards or regulations issued by national authorities or associations that supply these approaches? (Please give the reference)**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. ) *and* Answer was 'No' at question '56 [27]' ( Question 27 Are there national standards or guidelines in your country, that impose field testing as a tool for Earthquake Engineering? )

Please write your answer here:

**63 [28]**

**Last Question (Question 28)**

**Are there any needs for Information and Training for field testing?**

*

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. )

Please choose **only one** of the following:

- ○ Yes
- ○ No

Make a comment on your choice here:

If YES, Briefly describe in note form what kind of information will be needed:

**64 [Email]Please enter your email address here: ***

**Only answer this question if the following conditions are met:**
° Answer was 'Yes ' at question '63 [28]' ( Last Question (Question 28) Are there any needs for Information and Training for field testing? )

Please write your answer here:

**65 [further comments]If you have any further comments you would like to make on the subject of this questionnaire, please do so within the space provided below:**

**Only answer this question if the following conditions are met:**
° Answer was 'Yes' at question '4 [1]' ( Question 1 Have you used any measurement technologies with respect to field testing? If YES, please continue. If NO, please forward the email about this survey to others who may be knowledgeable about this subject. )

Please write your answer here:

**Thank you very much for taking the time to complete this questionnaire,**
**your help is very important to the success of this major EU project and highly appreciated.**

http://www.nera-eu.org

01.01.1970 – 01:00

Submit your survey.
Thank you for completing this survey.