SIBYL

(Se**I**smic monitoring and vulnera**B**ilit**Y** framework for civi**L** protection)

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Deliverable DC1: **Guidelines for the building assessment procedure and short-term monitoring**

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1. Introduction

Seismic vulnerability analysis of existing buildings is primarily based on their structural evaluation. A comprehensive seismic analysis should properly take into consideration all factors which may influence performance of the structures under seismic excitation, including the local ground conditions, type of foundations, overall configuration and possible geometrical or physical irregularities of the system, cross-sectional and mechanical properties of all structural (and non-structural) members which are intended to provide the integrity and seismic resistance of the overall system and its constituents.

In accordance with Eurocode 8, Part 3 (EN 1998-3: 2005) for assessing the earthquake resistance of existing structures, the input data can be collected from a variety of sources, including available construction documentation (design drawings and specifications), contemporary building codes and standards, field investigations and in-situ or laboratory measurements and tests. Depending on the sources of information and the methods of data collection, the amount and quality of the required input data as well as the level of related uncertainty may considerably differ. In this regard, for identifying appropriate methods of analysis and corresponding confidence factors, Eurocode 8 defines three knowledge levels: KL1: Limited knowledge, KL2: Normal knowledge, KL3: Full knowledge. The appropriate knowledge level (KL) is based on the amount and quality of the collected and available information. The factors determining the appropriate knowledge level are:

i) geometry: the geometrical properties of the structural system and of such nonstructural elements (e.g. masonry infill walls) as may affect structural response.
ii) details: these include the amount and detailing of reinforcement in reinforced concrete, connections between steel members, the connection of floor diaphragms to lateral resisting structure, the bond and mortar jointing of masonry and the nature of any reinforcing elements in masonry,
iii) materials: the mechanical properties of the constituent materials.

These three knowledge levels and corresponding data sources as well as methods of analysis and the values of the confidence factors recommended by Eurocode 8, Part 3 are presented in Table 1.1.

As SIBYL aims at developing of a fast and simple approach for assessment of seismic vulnerability of existing structures (either singles or a group of buildings), we consider (in addition to the three mentioned knowledge levels) also a lower accuracy/higher uncertainty level, which can be designated as “very limited knowledge level” or KL0. This knowledge level is not considered in the current normative regulations; however, we proceed from the fact that there exists a great variety of practical situations, when engineers and specialists have to deal with very limited amount of information available for seismic evaluation of existing structures. Accordingly, considering the level KL0, we assume that no construction documentation is available and the structural evaluation (as well as the vulnerability assessment) is based solely on a quick, superficial survey of the structure; that is the aggregate knowledge level KL0 is significantly less than as described for knowledge level KL1 in Eurocode 8.

Table 1.1: Knowledge levels and corresponding methods of analysis (EN 1998-3: 2005)

	Limited knowledge (KL1)	Normal knowledge (KL2)	Full knowledge (KL3)
Geometry	From original outline construction drawings with sample visual survey or from full survey	From original outline construction drawings with sample visual survey or from full survey	From original outline construction drawings with sample visual survey or from full survey
Details	Simulated design in accordance with relevant practice and from limited in-situ inspection	From incomplete original detailed construction drawings with limited in-situ inspection or from extended in-situ inspection	From original detailed construction drawings with limited in-situ inspection or from comprehensive in-situ inspection
Material	Default values in accordance with standards of the time of construction and from limited in-situ testing	From original design specifications with limited in-situ testing or from extended in-situ testing	From original test reports with limited in-situ testing or from comprehensive in-situ testing
Analysis	Linear analysis methods, either static or dynamic	Linear or nonlinear analysis methods, either static or dynamic	Linear or nonlinear analysis methods, either static or dynamic
Confidence Factors	CF_{KL1}=1.35	CF_{KL2}=1.20	CF_{KL3}=1.00

In the framework of the SIBYL study we will mainly keep to the level KL1 (limited knowledge), as this level reflects, in our opinion, the most plausible situations in the routine practice of Civil Protection. At the same time, we will also consider briefly the levels KL2 and KL0. The level KL2 (normal knowledge) corresponds to the situations, when comparatively higher accuracy is pursued. The level KL0 (very limited knowledge) corresponds to reverse (but also plausible) situations, when a group of buildings or even built-up areas have to be investigated within tight deadlines, provided that a rough vulnerability estimation is acceptable.

In the current practice of earthquake engineering and risk assessment for urban areas there exist a variety of approaches for seismic evaluation and vulnerability analysis of existing buildings; see, for example, an overview in Guegen (2013), Pitilakis et al. (2014), Yepes-Estrada et al (2016). Considering both the different methods of data collection and the methods of modelling and analysis, the range of existing approaches (as well as the level of involved uncertainties) is very broad: from rough vulnerability estimation on the basis of fuzzy information obtained with the use of satellite images (e.g., Geiß et al., 2014) or ground-based remote-sensing techniques (e.g., Pittore and Wieland, 2013) to engineering scoring based on the data obtained from sidewalk surveys (e.g., FEMA P-154, 2015) or in-situ inspections of different detail (EN 1998-3: 2005), from calculations based on empirical models (e.g., Lagomarsino and Cattari, 2013) to more sophisticated non-linear static (e.g., Fajfar, 2000, FEMA-440, 2005,) or dynamic (e.g., Vamvatsikos and Cornell, 2002) computational analyses using structure-specific or time-dependent vulnerability models (e.g., Pitilakis et al., 2014, Karapetrou et al, 2016).

While selecting an appropriate approach for practical use, one should understand that, depending on the used input data and the selected methods of analysis, not only the level of involved uncertainty, but also the amount of required resources may vary considerably. For example, considering the computational approaches, which may allow achieving higher accuracy one should keep in mind that, apart from the requirements of more detailed input data and extended time, special engineering software would also be needed as well as special qualification of the user. On the other hand, considering simplified empirical approaches in addition to the inherently higher uncertainty, it is worth noting, that very often those methods are developed for region-specific building typologies and, therefore, cannot be used universally.

The goal of this part of the project (Action C) is to develop and offer to the end-users (Civil Protection authorities) a simple, low-cost, rapid and, along with that, scientifically robust approach for vulnerability estimation. At the same time, we have to take into consideration the fact that the targeted end-users can meet various situations in practice (corresponding to different knowledge levels, ranging supposedly from KL0 to KL2); therefore the operational framework should be flexible and afford a set of different ways (techniques) to solve the problems of vulnerability analysis in different possible conditions balancing required accuracy within available time frames.

The prepared Guidelines for seismic assessment of buildings (Section 2) are accompanied with Explanatory Notes (Section 3), containing description of the conceptual and scientific background of the approach, and an example of practical application of the approach (Section 4).

2. Guidelines for seismic building assessment

Generally, the main steps of the building evaluation procedure should include: data collection, structural modelling and seismic evaluation.

2.1. Data collection

The key input data required for constructing an adequate structural model and seismic vulnerability assessment of buildings are summarized in Table 2.1.

Table 2.1: Main input data required for structural modelling and vulnerability assessment of buildings

NN	Data required for structural modelling and vulnerability assessment
1	Lateral load-resisting system and material of bearing structures
2	Overall dimensions and shape of the building (including the presence and location of separation joints)
3	Presence of irregularities (physical or geometrical) in plan or in elevation
4	Dimensions and location of structural components (columns, walls, braces, shafts, slabs)
5	Cross-sectional (shape, reinforcement ratio) and the material properties (concrete and steel strength values, elastic moduli, specific density) of the structural members
6	Presence of non-structural elements and other building components, which can contribute to the stiffness and/or mass distribution and their characteristics
7	Year of construction (and modification) of the building and its previous and current occupancy (as well as importance class)
8	Current state of the preservation and physical condition of structural elements

The following main data sources should be used for collecting the required data:

1. Construction documentation (drawings and specifications).
2. In-situ collected data (structural survey and measurements).
3. Simulated design (following to the contemporary codes and standards).

The thoroughness and detail of the data collection procedure as well as the method of analysis should be selected depending on the pursued knowledge level (KL2 – normal knowledge, KL1 – limited knowledge, KL0 – very limited knowledge), considering and balancing, on the one hand, the acceptable accuracy and, on the other hand, the available time frames and resources.

2.1.1. Construction documentation

The requested construction documentation should include both drawings (plans, cross-sections, structural details) and specifications. The graphical and textual parts are complementary and should provide the information (see Table 2.1) necessary for the structural analysis and seismic evaluation.

Using the available documentation, the researcher should be able to identify the structural system to resist both vertical and lateral loads and material of bearing structures, overall geometry of the building and its structural components and connections between them. The structural details and mechanical properties of construction materials should be obtained from the design specifications or from original test reports.

Information about the year of construction (or subsequent modifications) of the building as well as about its occupancy (both current and previous) should be obtained from the building owner or from the municipal authorities. If there was any modification of the whole building or its structural parts, or the building was seriously damaged and repaired/recovered, the corresponding documentation should be also requested and analyzed.

If full and up-to-date construction documentation is available, in this case only short in-situ survey would be necessary for checking the correspondence between the available documentation and the actual state of the building. In case of discrepancies, the actual in-situ findings should have priority for the vulnerability assessment. If significant discrepancies are found out during the survey, then a fuller survey and in-situ inspection should be conducted.

2.1.2. In-situ measurements

For the knowledge level KL0 the necessary information, as a rule, can be obtained from a short visual survey, identifying both the structural system and the material of bearing structures as well as observable strengths and weaknesses of the building and the corresponding vulnerability class can be assigned immediately on site (EMS-98). For higher knowledge levels (KL1 or KL2) the researcher should collect information with the purpose of constructing an adequate and sound structural model of the building suitable for accurate computational analyses.

For achieving the level KL1, if the construction documentation is not complete or not available, the lacking information about geometry and details can be obtained from visual survey and limited in-situ inspection, while the material properties can be taken as default values following to the standards of the time of construction or from limited in-situ testing.

For achieving the level KL2, incomplete construction documentation can be updated from limited in-situ inspection; otherwise, if the documentation is not available, an extended in-situ inspection should be conducted. Correspondingly, the material properties can be obtained from original design specifications with limited in-situ testing or from extended in-situ testing.

One should take into consideration the requirements of Eurocode 8 for different levels of inspection and testing (EN 1998-3: 2005, Table 3.2). In particular, percentage of elements to be checked for details should be not less than 20% (for limited inspection) and not less than 50% (for extended inspection). At the same time, the number of samples should not be less than 1 per floor (for limited testing) and not less than 2 per floor for extended testing.

During the structural survey, if the construction drawings are not available, a schematic plan should be sketched on site, showing the overall dimensions and shape of the entire building, location of separation joints and also indicating the location of structural elements and their dimensions. Sketches or/and photos of the overall view as well as of different parts of the building should also be taken and documented, as they will be helpful for the further analysis of collected information and generating an adequate structural model.

Particular attention should be paid to examination of the current physical condition of structural elements, including possible presence of any damage or degradation due to concrete carbonation, steel corrosion, etc.

In the course of in situ-measurements special tools and techniques for non-destructive testing (NDT) are necessary for obtaining the actual characteristics of the investigated structure (as described in Table 2.1). A list of recommended tools and methods for in-situ data collection, in particular, about geometry, structural details and mechanical properties of materials is presented in Table 2.2.

Table 2.2: Recommended tools and methods for in-situ data collection

Data to be measured	Recommended tools and methods
Geometry (dimensions of the building and its components)	Laser distance meter, measure tapes
Structural details (presence and location of reinforcement bars, depth of concrete cover)	Metal detectors (e.g., Hilti PS 50 Multidetector, https://www.hilti.de/), Malhotra and Carino (2004)
Concrete quality and strength	Rebound hammer test; ultrasonic pulse velocity measurements; pull-out tests, (e.g., http://www.proceq.com/), EN 13791 (2007), Bungey et al (2006)
Thickness of structures (when the back-side is not accessible)	Impact-echo techniques (e.g., http://www.impact-echo.com/), Sansalone and Street (1997)

Considering existing variety of NDT techniques and tools and depending on the pursued knowledge level as well as available time and resources, the user can decide which of the recommended methods and tools can be used for practical applications.

2.1.3. Simulated design

For the knowledge level KL1, if the input parameters necessary for structural modelling and analyses are not available either from the construction documentation or from in-situ measurements (e.g., reinforcement ratio of structural elements or some properties of construction materials), the default values may be assumed in accordance with standards and usual practice at the time of construction of the building. The Eurocode 8 suggests, nevertheless, that limited inspections and in-situ testing for the most critical structural members should be performed to check that the assumptions correspond to the actual situation.

2.1.4. Sort-term vibration measurements and operational modal analysis

The goal of the vibration measurements is to analyze the dynamic characteristics of the building using the ambient vibrations recorded in operational conditions. For this purpose a number of sensors should be installed in the building for a certain period of time. The measurements can be conducted in the day time or (to reduce the level of noise) in the night time.

The spatial arrangement of sensors should be aimed at providing the possibility of analysis of the dynamic behavior of both the whole building and its units (especially, if the structure has a complex configuration). Depending on the number of available sensors, they should be installed on every floor (or every second floor) of the building close to the corners and to the separation joints. The most vulnerable (from the engineering point of view) parts of the building (e.g., locations of irregularities, previously damaged, repaired or modified structural components) should also be monitored and analyzed.

The minimum number of sensors necessary for identifying the fundamental vibrational modes is 3; one of them should be installed at the level of the ground floor, and two other sensors should be installed on the top of the building. The two upper sensors should be installed in the opposite ends of the building diagonally to make it possible catching the fundamental frequencies in both directions.

The recommended duration of the vibration measurements is 30 minutes, though for the modal analysis of normal buildings a few (2 to 3) minutes of digital records of vibrations should be sufficient.

The processing of the recorded data would require special software (like, e.g. MACEC, Reynders et al., 2014). If such software is not available, the fundamental frequency of buildings can be estimated using empirical formulae (e.g., EN 1998-1, 2004, Goel and Chopra, 1997, Crowley and Pinho, 2006).

2.2. Structural analysis and seismic evaluation

The structural model for the analyses corresponding to the knowledge levels KL1 and KL2 is set up based on the collected information as described in Section 2.1. The values of input parameters obtained from in-situ tests (or from additional sources of information) should be corrected using the confidence factors (CF) for appropriate knowledge level, as prescribed by Eurocode 8, Part 3 (EN 1998-3: 2005). The confidence factors, which account for the corresponding uncertainty level, are CF_{KL1} =1.35 for KL1, CF_{KL2} =1.20 for KL2.

For the structural analysis corresponding to the knowledge level KL1 a special computational procedure based on simplified integral structural model (SISM) of the building was developed within the framework of SIBYL and implemented using MS Excel software (detailed description of the developed procedure is given in Deliverable DC3).

The procedure starts with generation of simplified integral structural model of the building. First, the topological model reflecting the principal characteristics of the real building (overall dimensions, number of floors and their heights, grid of columns, etc.) is constructed. Then the model is updated with the collected information about structural details (including cross-sectional data) as well as mechanical properties of the structural components. The input data should be prepared in the form of an Excel sheet (following the operating instructions presented in Deliverable DC3).

When the user complete the data input, the calculations start automatically. As a first iteration, for the model based on the collected input data, the vibrational modes and frequencies are numerically computed and compared with the measured data to check the quality of the model. In case of notable discrepancies, the parameters of the generated model should be corrected (tuned) correspondingly.

If the calculated differences between the measured and numerical values lie within 20%, the model may be considered acceptable for the further analysis. The results of comparison in the Excel tool are presented in color using conventional traffic light indicator, correspondingly the results shown in green color are acceptable, while in case of red color parameters of the model should be corrected to achieve better fit. The improvement (refinement) of the model should be done using those building characteristics, which are associated with largest uncertainties in the available input data (e.g., they may be related to unknown cross-section sizes or mechanical properties of the building components).

As the next step of the procedure, the relationships between load and deformation for the three different limit states of reinforced concrete structures are calculated, namely: (1) first cracking in the concrete; (2) first yielding in the reinforcement; (3) failure of the structure. It is assumed that the point (2) approximately corresponds to the Limit State of Damage Limitation (DL, permanent drifts are negligible), while the point (3) approximately corresponds to the Limit State of Near Collapse (NC, large permanent drifts) in accordance with the definitions of EC8, Part 3. The corresponding moment-curvature diagrams are generated graphically for all structural elements in an additional spreadsheet of the Excel tool and can be checked immediately by the user.

Further, at the next step of the procedure, using the modeled moment-curvature relationships, the forces corresponding to occurrence of the limit states are calculated for every floor of the building. Comparing them with the actual seismic forces for the site allows concluding about the expected damage level for the investigated building under the seismic load at the location site. The computed results are also presented in terms of traffic light indicator (green or red), showing floor-by-floor whether and which one of the considered limit states is expected to occur in the building.

For the computational analysis (KL1 and KL2) of probable damage the seismic input is described in terms of spectral acceleration, taking into consideration the measured (or estimated) fundamental frequencies of the building. In case of KL0, for the rough EMS-based damage evaluation, the seismic hazard is considered in terms of macroseismic intensity. The level of seismic hazard should be taken from the seismic microzonation map (if available for the site) or from the national building code. Commonly, the hazard level corresponding to a non-exceedance probability of 90% in 50 years (mean return period of 475 years) should be considered for seismic evaluation of ordinary buildings.

For assigning the level of ground acceleration the importance class of the building (depending on the consequences of collapse for human life, on their importance for public safety and civil protection in the immediate post-earthquake period, and on the social and economic consequences of collapse) should be taken into consideration (Eurocode 8, Part 1, Table 4.3).

The developed SISM-based method and, correspondingly, the described Excel-based procedure can be used for the knowledge level KL1. Some more details of the developed approach are given in Explanatory Notes (Section 3.3 of this Deliverable). Detailed operating instructions for users of the tool can be found in the project deliverable DC3: Documentation for the developed software tools.

For the lower knowledge level KL0 the seismic vulnerability can be roughly evaluated immediately on site using the vulnerability classification table of EMS-98 (the details of the EMS-based approach are given in Section 3.4 of this Deliverable).

If higher knowledge levels (KL2 or even KL3) are pursued, provided that the amount and quality of collected input data meet corresponding requirements, then more-sophisticated methods of analysis are recommended (e.g., Fajfar, 2000, Karapetrou, 2016). Methodological framework for deriving building-specific fragility functions of RC buildings is outlined in Section 3.6; more detailed consideration of those approaches, however, is beyond the scope of this study.

3. Explanatory notes

Conceptually, the approach to be developed in the study should be building-specific, implying that every building needs an individual consideration; at the same time, as mentioned above, it should be simplified, cost-efficient and fast enough to be applicable for relatively large built-up areas in a comparatively short time, combining, therefore, a rational assessment rate with an acceptable accuracy of estimation.

The operational framework is schematically shown in Table 3.1, considering different possible approaches for different knowledge levels, depending on the available input data and selected methods of structural evaluation. We take as a premise that the choice of the appropriate method for seismic evaluation of a building (or a group of buildings) should be done by Civil Protection practitioners depending on their needs, taking into account pursued/achieved knowledge level and available time frames.

Table 3.1: Selection of the method of seismic evaluation

<table>
<tr><td colspan="3">Data collection and pursued/achieved knowledge level</td></tr>
<tr><td>KL2</td><td>KL1</td><td>KL0</td></tr>
<tr><td colspan="3">Structural analysis and seismic evaluation</td></tr>
<tr><td>Non-linear static or dynamic analysis</td><td>SISM-based analysis</td><td>Vulnerability classification</td></tr>
<tr><td colspan="3">Probable damage assessment</td></tr>
<tr><td colspan="2">Probability of different limit-states (in terms of inter-story drift, ISD) vs spectral acceleration</td><td>Probability of different damage grades (EMS) vs macroseismic intensity</td></tr>
</table>

One should bear in mind that depending on the knowledge level and selected method of analysis, the accuracy of vulnerability estimates (as well as the level of related uncertainty) may considerably differ. The level of epistemic uncertainty increases with decreasing the knowledge level, i.e. from the left side to the right side in the table; in the same direction decreases the accuracy of the methods used for the building assessment as well as the time and resources required for the building evaluation. Obviously, reducing the level of uncertainty can be achieved through additional costs, such as collecting more detailed information about the buildings and using more sophisticated models and methods of structural analysis.

In this context, while selecting the method, it is worthwhile to bear in mind that uncertainty related to the input data and uncertainty associated with the selected method of analysis should be comparable. On the one hand, it would not be rational to use more complex methods of analysis with insufficient input data, because in such case one cannot gain in accuracy. On the other hand, deliberately using simpler (and rougher) techniques, if a fuller dataset is available, would lead to losing accuracy of results. This should be taken into consideration by the end-users, while balancing between pursued accuracy and available time frames for selecting the appropriate approach for the vulnerability assessment.

3.1. Data required for seismic evaluation

The soundness of the structural modelling and subsequently correctness and accuracy of the vulnerability estimation will depend on the quality and completeness of the input information. Therefore, a special attention should be paid to the proper collection and interpretation of the data.

In the first line, for analyzing structural performance of the building under seismic loads and assessment of its vulnerability, one needs information about the lateral-load-resisting system (which should be identified in both directions) and the material of bearing structures. This information can be considered as minimum information, which allows one to make a first rough judgement about the seismic vulnerability of the building. Obviously, this information is of primary importance for the purposes of structural modelling.

Furthermore, for the needs of structural modelling the overall dimensions of the building should be known, including information about the shape of the building as well as about the presence and location of separation joints, which, generally, are designed to divide complex structures into simple structural units, which are dynamically decoupled. Also there should be identified possible irregularities of the building (both in plan and in elevation, e.g., incoming corners, discontinuous shear walls, soft story, etc.), which can considerably influence seismic performance and increase the vulnerability of the structure.

Additionally, for developing the structural model one certainly needs information about the dimensions and location of structural elements (columns, walls, slabs), including the information about cross-sections of the structural members and their material properties (strength, elastic moduli, specific density). If the data are not available from the construction documentation, they can be measured on site (using non-destructive testing for a few selected elements and then extrapolating the measured data for other elements of the building). Otherwise, these parameters can be estimated using simulated design. For this purpose the information about the occupancy of the building under study and the year of construction (modification) can be helpful and, therefore, should also be obtained.

For proper modeling one should pay attention to the current state of the preservation (maintenance of the building, presence of deficiencies or possible damage from past earthquakes). This refers to the state of the structural system, while superficially the building may appear to be in good condition, but the presence of structural deficiencies may considerably increase its vulnerability.

The above-listed information can be considered as a baseline of data needed for developing a simplified structural model for the analyses. At the same time, there can be other factors which may influence seismic performance of structures and, therefore, their consideration can be helpful for refining parameters of the structural model and achieving better accuracy in the vulnerability assessment. In particular, information about non-structural elements, which are not intended to bear the loads, but can contribute to the stiffness and to the mass (defining the inertia forces), should be collected or updated using simulated design. The same goes with regard to other building_components, which can contribute to the stiffness and/or mass distribution (e.g., heavy equipment, libraries, archives, etc.).

To refine the performance of the model one should also take into consideration possible effects of soil-structure interaction; for this purpose the information about the underground part of the building (depth and type of foundation) as well as characteristics of local soil conditions would be required. It worth mentioning, that the soil parameters can be derived from the results of field measurements (similar to those conducted by the GFZ team). However, in the current approach the seismic site effects are not considered, because, as a rule, such information is neither available nor directly accessible on site. Therefore, consideration of these aspects would require additional time and resources.

Furthermore, the position of the building with respect to the neighboring buildings should be taken into account, considering possible effects of pounding during seismic vibrations.

3.2. Data sources and data collection

The main data sources for structural modelling and building evaluation are: (1) construction documentation; (2) in-situ measurements and (3) simulated design.

The construction documentation should, ideally, include construction drawings, representing the graphical part of the documentation and specifications in the written form, for both the original construction and for any subsequent modifications. If available, this documentation should contain information about all structural and non-structural members, their geometry and details and would allow for identifying the structural system of the building.

As a matter of fact, a kind of structural model can be constructed solely on the basis of the construction documentation. However, Eurocode 8 providently recommends that, even if the complete set of documentation is available, at least a limited visual survey on site should be conducted for checking correspondence between the drawings and the actual state of the building.

The practical experience shows that for many existing buildings, especially for older parts of the building stock both in urban and rural areas, the construction documentation is only partly available or not available at all. Therefore, as a rule, additional sources of information – in-situ measurements and simulated design – are to be used for collecting the lacking information.

Generally, in-situ measurements may include visual survey (aimed at identifying structural members and describing their geometry), in-situ inspection (aimed at collecting information about actual details of the structure), in-situ testing (aimed at collecting information about mechanical properties of the construction materials). Depending on the completeness of the available construction documentation and/or pursued knowledge level, the level of survey, inspection and testing can be limited, extended or comprehensive. The requirements for the different levels of inspection and testing are regulated by Eurocode 8 (EN 1998-3: 2005).

At the present time many various non-destructive techniques and devices are being developed and used for evaluating the condition of existing structures, determining their quality and properties (see, e.g., Malhotra and Carino, 2004, Bungey et al., 2006, Breysse, 2012, Helal et al., 2015, Schiebold, 2014, 2015, for an overview). Some of them are rather complicated and expensive, though can be very helpful if the higher knowledge levels are pursued (for KL2 and especially for KL3. Keeping in mind, however, the objectives of the project, and remaining in the frames of the lower knowledge levels, we consider, in the first line, easily-accessible methods and tools. Some of the recommended tools and methods for in-situ measurements and non-destructive testing are described in Section 2, Table 2.2. The choice of particular tools and techniques, however, will depend on the user needs, as well as available resources and time frames.

If some details necessary for structural modelling and seismic evaluation of a building are still not available either from the construction documentation or from in-situ measurements, the lacking input parameters (e.g., amount and layout of reinforcement in structural members, mechanical properties of construction materials) can be estimated using simulated design. The simulated design should be based on the contemporary national/international codes and standards; as well one should take into consideration the regional peculiarities of design and workmanship in the period of construction of the building.

In parallel with the mentioned in-situ activities (visual survey, detail inspections, material testing), the developed approach includes also short-term monitoring of the building, implying vibration measurements in natural operational conditions (see Section 3.5). The ambient vibration measurements represent an important component of the entire data collection procedure necessary for the building assessment. The processed results of the measurements (operational modal analysis) will serve for determining natural vibration modes and eigenfrequencies of the existing structural system. Therefore, the parameters of the structural model, constructed on the basis of the data from the structural survey, can be refined using the measured dynamic characteristics of the building. Moreover, the results of operational modal analysis can be compared with the results of previous measurements (if available) or with the results of a numerical modal analysis obtained for a structural model based on the original design documentation. Doing so, one can judge whether there are any changes in the dynamical characteristics of the structural system under consideration and even localize the possible changes, if any. This may help identifying vulnerable points of the structure.

3.3. Structural modelling and seismic evaluation for the level KL1

In the framework of the study we develop a special procedure for simplified structural analysis and vulnerability assessment of buildings, which presumably corresponds to the knowledge level KL1. The current version of the approach is limited to reinforced-concrete frame buildings with infill walls and more or less regular and symmetrical configuration in ground plan. Further, the approach is currently incapable to assess the purely shear wall buildings, the torsional vibrations as well as the coupled bending-bending and bending-torsional response modes.

The developed approach includes simplified structural modelling based on limited information on the structure collected directly on site within a short time (a few hours). Such data usually contains buildings dimensions, structural and material type, dimensions of the main structural members and their position as well as some other additional data obtained with the help of non-destructive tests and vibration measurements on site, as described in the previous paragraphs; the lacking information is obtained using simulated design.

The developed computational procedure employs the displacement method of structural mechanics with specific group variables as explained below. The simplified integral structural model (SISM) of buildings consists of one integral beam element and one mass per floor (Figure 3.1).

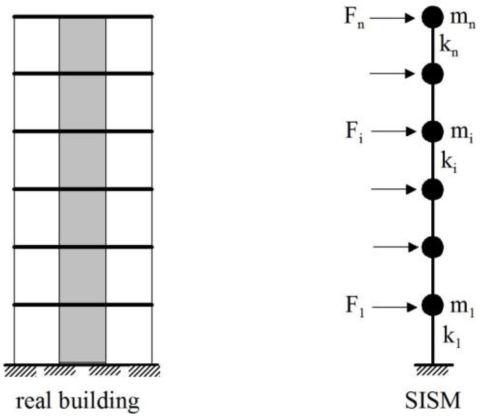


Figure 3.1: Real building and its simplified integral structural model (SISM)

Typical buildings with about 2 to 10 stories cannot be exactly modeled as a beam, irrespective of the fact whether the Bernoulli or the Timoshenko theory is applied. The dimensions of real buildings do not match typical relations suitable for the beam theory. Usually, such buildings exhibit both bending and shear deformations, which are combined in a ratio depending on the individual stiffness properties.

Simplified structural beam models for such buildings can be developed by use of the characteristic grouped degrees-of-freedom (DOF). The procedure to calculate the strength and stiffness of the equivalent beam described below shall be applied for each story individually.

We consider a single storey reinforced concrete frame with in-fill walls as depicted in Fig. 3.2, a. The single integral DOF which is necessary for estimating structural fragility is the horizontal displacement ***u*** which corresponds to a horizontal force ***F***, for example, the inertia force. This equivalent beam stiffness

$$k = \frac{F}{u}$$

shall take into account both the shear and the bending deformations of the whole frame as shown in Fig. 3.2,b.

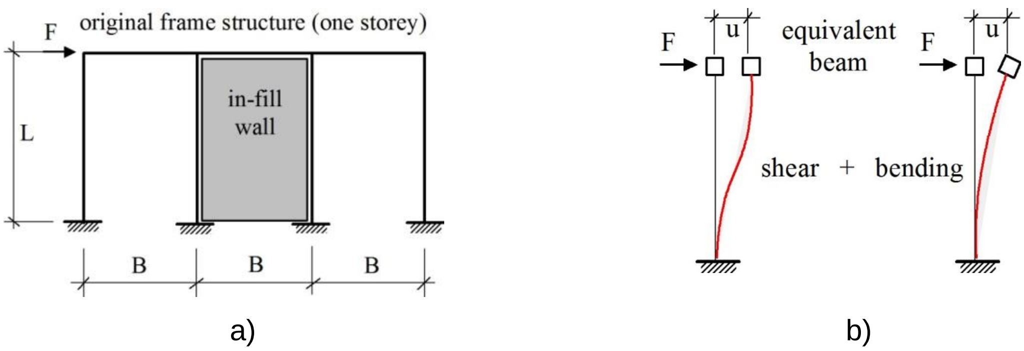


Figure 3.2: Frame structure and its equivalent beam model with a single DOF

From the structural mechanics viewpoint, we define three grouped DOF which influence this characteristic frame stiffness with respect to horizontal forces (Figure 3.3):

DOF 1: u: same horizontal displacement for all girder nodes; the corresponding energy-conjugate variable is force F;

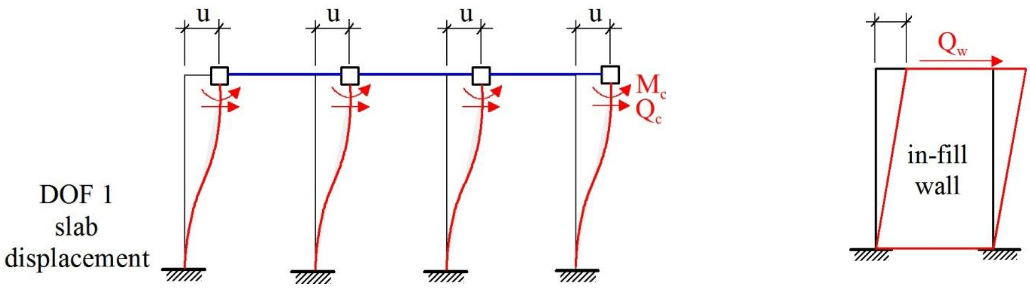
DOF 2: φ: equal rotation of all girder nodes; the corresponding energy-conjugate variable is bending moment M;

DOF 3: ψ: slab rotation; the corresponging energy-conjugate variable is the moment, consisting of a series of normal forces in each column multiplied by its distance to the point of rotation.

The integral stiffness model takes into account the individual stiffnesses of all main structural members like columns, beams, walls and a slab or girder. At that, the individual column and girder stiffnesses are taken from the classical stiffness method and include the contributions with respect to normal and shear forces as well as bending moments (Figure 3.3). They are summarized over the whole storey, leading to the following stiffness relation for the frame with 3 DOF:

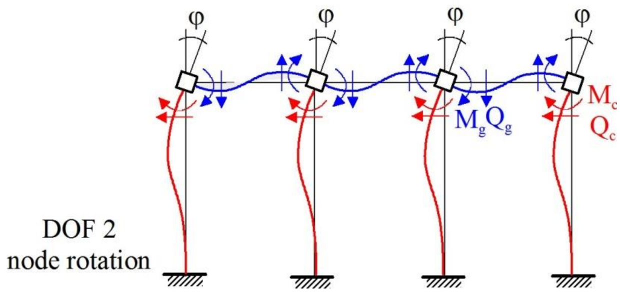
$$\mathrm{K} \cdot \mathrm{V} = \mathrm{P} \quad \rightarrow \begin{bmatrix} K_{uu} & K_{u\varphi} & 0 \\ K_{\varphi u} & K_{\varphi\varphi} & K_{\varphi\psi} \\ 0 & K_{\Delta\psi} & K_{\psi\psi} \end{bmatrix} \begin{bmatrix} u \\ \varphi \\ \psi \end{bmatrix} = \begin{bmatrix} 1 \\ 0 \\ 0 \end{bmatrix} \tag{3.3.1}$$

$$\mathrm{V} = \begin{bmatrix} u \\ \varphi \\ \psi \end{bmatrix} = \mathrm{K}^{-1} \cdot \mathrm{P} \tag{3.3.2}$$

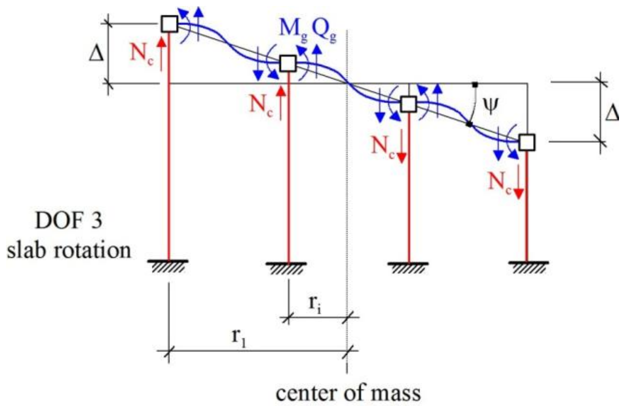


a) shear in a frame

b) shear in a wall



c) pure bending in a frame



d) bending with longitudinal forces in a frame

Figure 3.3: Three characteristic DOF with corresponding forces and displacements

The required equivalent beam stiffness K_u^* with respect to the horizontal displacement $\boldsymbol{u}$ of the slab can be calculated as a relation of the unit force to the slab displacement calculated above in (X):

$$\mathrm{K}_\mathrm{u}^* = \frac{1}{\mathrm{u}} \qquad \text{with} \qquad \mathrm{K}_\mathrm{u}^* \neq \mathrm{K}_\mathrm{uu} \tag{3.3.3}$$

This stiffness is generally smaller than K_uu which corresponds to the shear deformation only. Such an approach delivers an equivalent storey stiffness which is quite realistic and does not require any highly-sophisticated numerical analysis like the finite element simulation. The stiffness of individual members as well as the equivalent story stiffness are considered and calculated by use of the Excel sheets.

The individual story stiffnesses $\mathrm{k_i}; \mathrm{i} = 1, .., \mathrm{n}$ build a system stiffness matrix for the SISM (Figure 3.1) according to the classical displacement method. For example, we get for a three-story building:

$$\mathrm{K} = \begin{bmatrix} \mathrm{k}_1 + \mathrm{k}_2 & -\mathrm{k}_2 & 0 \\ -\mathrm{k}_2 & \mathrm{k}_2 + \mathrm{k}_3 & -\mathrm{k}_3 \\ 0 & -\mathrm{k}_3 & \mathrm{k}_3 \end{bmatrix}. \tag{3.3.4}$$

All masses of structural members shall be summarized to a story mass and taken into account in the lumped mass matrix, exemplarily for 3 stories

$$\mathrm{M} = \begin{bmatrix} \mathrm{m}_1 & 0 & 0 \\ 0 & \mathrm{m}_2 & 0 \\ 0 & 0 & \mathrm{m}_3 \end{bmatrix} \tag{3.3.5}$$

The mass of the non-structural members are generally not exactly known. Thus, they can be taken into account by increasing the mass of the structural members by a factor:

$$\widehat{\mathrm{m}}_\mathrm{i} = \mathrm{m}_\mathrm{i} \cdot \alpha \, .$$

With the stiffness and mass matrices at hand, we can calculate the mode shapes ϕ and natural circular frequencies (squared) $\omega_i^2 = diag\{\Omega\}$ of the SISM from a generalized eigenvalue problem:

$$(\mathrm{K} - \Omega \mathrm{M})\phi = 0 \, . \tag{3.3.6}$$

The physical eigenfrequencies of the SISM are given by:

$$f_i = \frac{\omega_i}{2\pi}\ [Hz]. \tag{3.3.7}$$

The model validation and improvements (if necessary) are performed by comparing the calculated and measured natural frequencies and corresponding mode shapes of the building. The main uncertainties are usually material properties like stiffness and strength, structural properties like cross-sections and joints, main shear or infill walls that are relevant for structural behavior and total masses of each story. These parameters can be varied in the model during its “tuning” phase with respect to eigenfrequencies.

In the second step, SISM is extended to catch the nonlinear behavior and limit states of the structural members in the framework of the damage and plasticity theory for reinforced concrete. The conventional limit states considered here are:

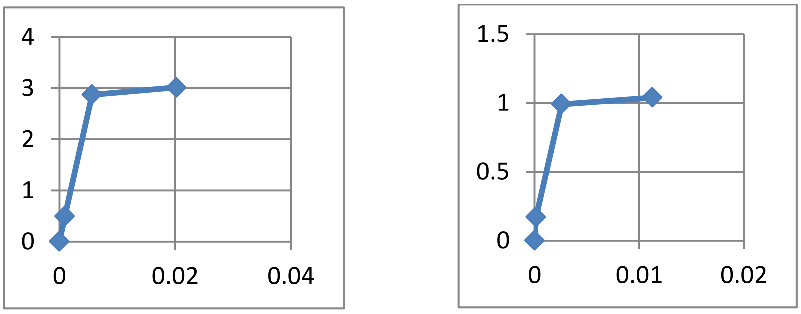
Limit state LS1: crack appearance in concrete structural members

Limit state LS2: yielding of the reinforcement bars

Limit state LS3: failure of reinforced concrete members.

As mentioned above, these three limit states are considered as performance points; at the same time, following the definitions of EC8, Part 3, we assume that LS2 approximately corresponds to the Limit State of Damage Limitation (DL), while LS3 approximately corresponds to the Limit State of Near Collapse (NC).

The above mentioned limit states are calculated for each column and shaft, as reinforced concrete beams, by means of the moment-curvature-relationship M-κ accounting for an individual axial force N in each story. A typical N-M-κ relationship contains 3 characteristic points marking the crack moment, the yield one and the failure moment, as shown in Fig. 3.4,a:



a) M [MNm] -κ [–]   b) F [MN] - u [□]

Figure 3.4: Moment-curvature-relationship and load-displacement-relationship with three characteristic states for a typical column

At that, the strength of concrete and rebars as well as the reinforcement ratio are assumed to be known, either from the design drawings or the measurements on-site or from simulated design valid for the moment of construction.

Each moment-curvature-relationship is then recalculated into the corresponding force-displacement-relationship (Fig. 3.4,b). The characteristic shear forces are determined from the clamped-clamped beam model subjected to node lateral force according to:

$$F_i = \frac{2M_i}{L}; i = 1,2,3,$$

where F_i, M_i are the force and the moment in the limit state i; L indicates the beam length.

The characteristic integral shear force of the whole story in every limit state is then calculated by summing up the corresponding individual forces of each column and shaft and taking into account the relevant wall shear forces:

$$F_{story} = \Sigma F_{colums} + \Sigma F_{shafts} + \Sigma F_{walls}.$$

The characteristic shear force for each wall is calculated from the stiffness relation with empirically reduced stiffness and top displacements equal to that of the neighboring columns in order to ensure the displacement compatibility:

$$F_{wall} = K_{wall,i} \cdot u_{column,i}\,; i = 1,2,3$$

$$\begin{gathered} K_{wall,1} = K_{wall}^{linear} \\ K_{wall,2} = 0.7 \cdot K_{wall,1}. \\ K_{wall,3} = 0.5 \cdot K_{wall,1} \end{gathered}$$

The analysis procedure described above is implemented in corresponding scripts in the Excel-sheet. The user manual is a part of deliverable DC3. The same nonlinear analysis is provided for the both horizontal directions x and y of the building.

The earthquake loading on the building is determined according to the EC 8 in form of a total horizontal shear force:

$$F_{tot} = \lambda\, m\, S_a,$$

with an importance factor λ for the building, total mass of the building m and a spectral acceleration on site S_a. The latter can be taken from the corresponding national design codes or according to direct measurements on site, like those provided by project partners GFZ and AMRA.

The total shear force F_{tot} is then distributed into individual story forces (in x and y direction separately) according to the corresponding 1st principal vibration mode Φ_1, as depicted in Fig. 3.5.

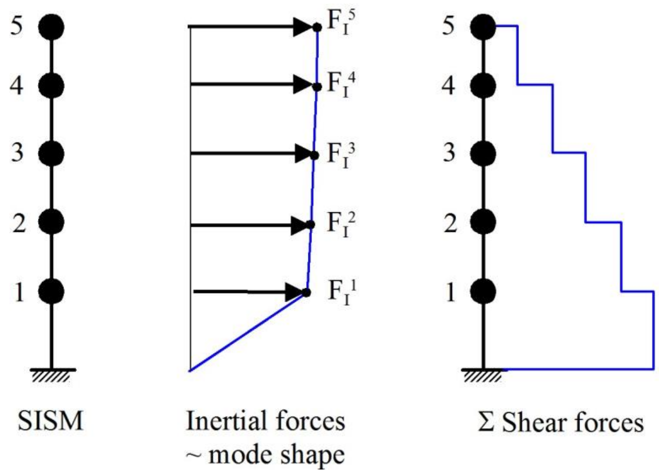


Figure 3.5: Shear forces according to the earthquake loading

The sum of all story shear forces from top to bottom, shown in the right hand part of Fig. 3.5, is equal in the ground floor to the total shear force F_{tot}.

Finally, the expected damage for the building for a given earthquake loading is determined from the comparison of the acting shear forces in each story (demand) due to Fig. 3.5 with those shear forces that are necessary to cause the corresponding limit state (capacity). The latter are calculated according to the procedure described above. The result of such a comparison is a prediction whether one limit state will be reached or not in each story individually.

It is important to note that the earthquake loading is re-calculated for each limit state anew due to the change of eigenfrequencies and mode shapes of the building as a result of damage.

More details about the computational procedure as well as operating instructions for the developed MS Excel-based application are provided in the deliverable DC3.

A considerable advantage for applying the offered approach might be the fact that it does not require using sophisticated commercial finite element software; instead the computational procedure is presented as Microsoft Excel-based tool. There are obvious benefits of this, in particular, because this software is widely used and commonly available, therefore most engineers and specialists are already familiar with this application. This implicates also good opportunities for training of civil protection practitioners and other potential users of this tool.

3.4. Vulnerability assessment for the knowledge level KL0

In situations, when a rather large group of buildings needs to be investigated within very short time frames, while very limited information about the buildings is available (or not available and should be obtained immediately on site), a rough evaluation of their seismic vulnerability can be performed using the vulnerability classification of the European Macroseismic Scale (EMS-98, Grünthal, 1998). Although such approach is mostly used for assessing the seismic vulnerability (and risk) for built-up areas, it can also be helpful to provide relevant information for rough evaluation of individual buildings.

The procedure of EMS-based vulnerability analysis starts with assigning an initial (most likely) vulnerability class (shown by a circle in Table 3.2) in dependence on the material of bearing structures and the structural type of the building under consideration. After that, using the available/collected information about the building, one should analyze the presence of factors (strengths or weaknesses) that may affect its seismic vulnerability, among which are, in particular, the quality of material, design and workmanship; regularity/irregularity of the structural system (both in physical and geometrical sense); state of preservation; possible interferences (re-planning or retrofitting) in the course of exploitation of the building, etc. Taking into account all those factors the initial estimation can be refined (within the uncertainty range shown by solid and dotted lines in Table 3.2) and the appropriate vulnerability class should be assigned for the building.

Table 3.2: Vulnerability classification of the European Macroseismic Scale (EMS-98)

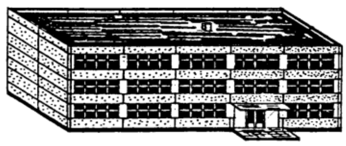
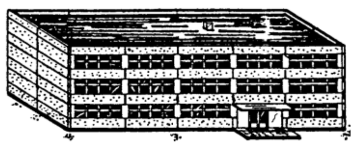
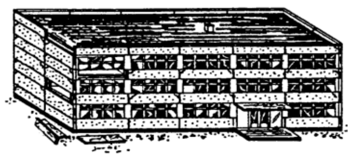
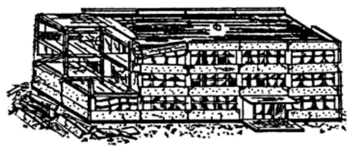
	Type of Structure	A	B	C	D	E	F
MASONRY	rubble stone, fieldstone	○					
	adobe (earth brick)	○	—				
	simple stone	···	○				
	massive stone		—	○	···		
	unreinforced, with manufactured stone units	···	○	···			
	unreinforced, with RC floors		—	○	···		
	reinforced or confined			···	○	—	
REINFORCED CONCRETE (RC)	frame without earthquake-resistant design (ERD)	···	—	○	···		
	frame with moderate level of ERD		···	—	○	—	
	frame with high level of ERD			···	—	○	—
	walls without ERD		···	○	—		
	walls with moderate level of ERD			···	○	—	
	walls with high level of ERD				···	○	—
STEEL	steel structures			···	—	○	—
WOOD	timber structures		···	—	○	—	

○ most likely vulnerability class; — probable range;
····· range of less probable, exceptional cases

Having identified the appropriate vulnerability class, one can roughly estimate probable damage grades for different levels of seismic intensities, using the definitions of intensity degrees given in EMS-98 (Grünthal, 1998). At the same time, for this purpose one can use the vulnerability relationships (in the form of damage probability matrices, fragility curves, and vulnerability functions), which are available for different vulnerability classes of EMS-98 (e.g. Giovinazzi and Lagomarsino, 2004, Tyagunov et al., 2006) and can be used for obtaining rough damage assessments.

The corresponding damage grades for reinforced concrete buildings are described in Table 3.3. Considering the description of these damage grades jointly with the definitions of Limit States given in EC8, Part 3, one may assume that the damage grade 2 approximately corresponds to the Limit State of Damage Limitation (DL), the damage grade 3 – to the Limit State of Significant Damage (SD), and the damage grade 4 – to the Limit State of Near Collapse (NC).

Table 3.3: Classification of damage to reinforced concrete buildings (EMS-98)

	**Grade 1: Negligible to slight damage (no structural damage, slight non-structural damage)** Fine cracks in plaster over frame members or in walls at the base. Fine cracks in partitions and infills.
	**Grade 2: Moderate damage (slight structural damage, moderate non-structural damage)** Cracks in columns and beams of frames and in structural walls. Cracks in partition and infill walls; fall of brittle cladding and plaster. Falling mortar from the joints of wall panels.
	**Grade 3: Substantial to heavy damage (moderate structural damage, heavy non-structural damage)** Cracks in columns and beam column joints of frames at the base and at joints of coupled walls. Spalling of concrete cover, buckling of reinforced rods. Large cracks in partition and infill walls, failure of individual infill panels.
	**Grade 4: Very heavy damage (heavy structural damage, very heavy non-structural damage)** Large cracks in structural elements with compression failure of concrete and fracture of rebars; bond failure of beam reinforced bars; tilting of columns. Collapse of a few columns or of a single upper floor.
	**Grade 5: Destruction (very heavy structural damage)** Collapse of ground floor or parts (e. g. wings) of buildings.

Another simple and fast, but obviously rough way of seismic vulnerability and damage estimation can be using generic fragility and vulnerability relationships, which have been developed in the frame of research projects dealing with earthquake risk assessment (e.g., HAZUS, RISK-UE, SINER-G). Such fragility curves for different structural types of buildings are available in literature (e.g., Pitilakis et al., 2014) and they are applicable for built-up areas with typical buildings, when building-specific information is not available or not critical for the study. At the same time, while using the generic curves, one should be aware of possible national/regional peculiarities of structural types, which might be significantly different, in spite of seemingly similar building taxonomies.

Needless to say, those simplified and rough vulnerability estimation methods would neglect many building-specific details of seismic performance of the structures and the level of uncertainty related to those approaches is rather high. In this case, the approach corresponding to the knowledge level KL0 can be used only for rough preliminary evaluation of seismic vulnerability of buildings.

3.5. Operational modal analysis (OMA) using field monitoring data

For the short term structural monitoring usually ambient vibration measurements are conducted on the building. Ambient vibrations have relatively low amplitudes compared to earthquake induced excitations. As a result of low amplitude excitation only a limited number of modes of the structure are expected to be excited. It is usually enough however to identify the first few modes of the structure in order to understand its general dynamic behaviour. The number of measurement locations in a structure depends on the expected response and also the size of the structure. For instance, if only horizontal vibrations are expected in the structure three sensors measuring the horizontal vibrations for each floor are enough. On the other hand, if there is rocking motion expected in the structure additional three or four sensors measuring the vertical vibrations are required in the foundation level. Also a set of sensors should be located where the typical floor mass and rigidity changes throughout the height of the building. Figure 3.6 shows typical building instrumentation schemes (Celebi, 2001).

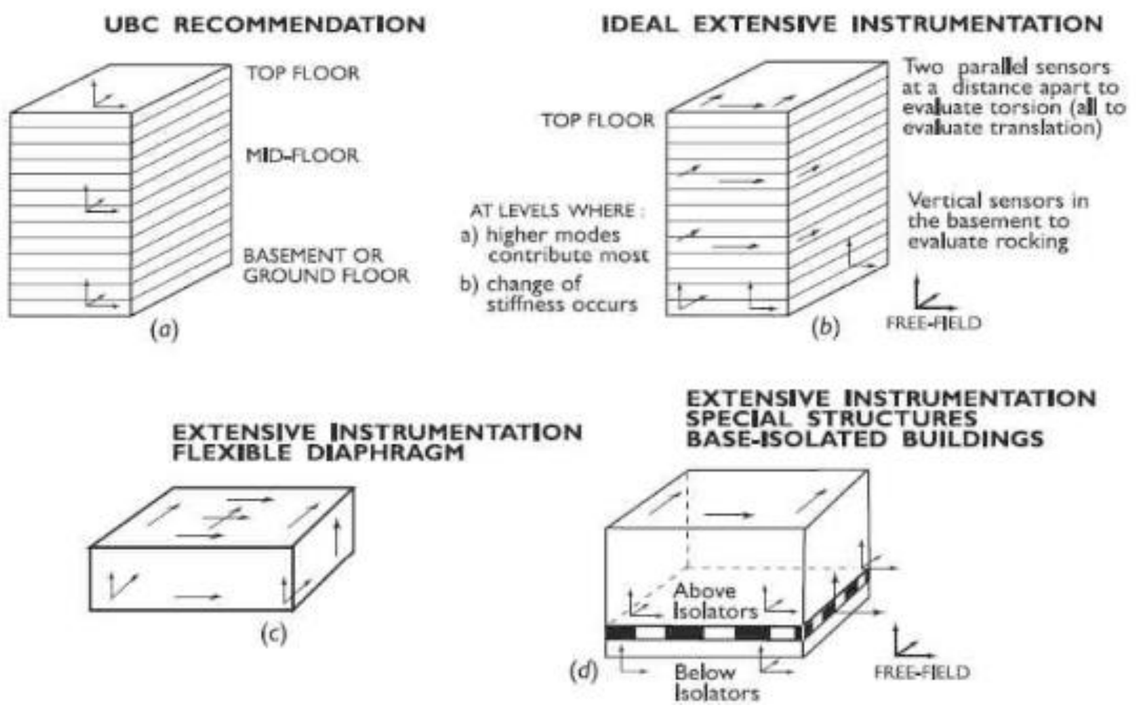


Figure 3.6: Instrumentation schemes (Celebi, 2001)

System identification is the process of building a mathematical model of a physical system based on experimental data (Ljung, 1999). From an engineering point of view the goal of system identification is to predict the physical quantities of a system based on its mathematical model (Kim, 2011). Research on linear system identification evolved in the late 1960s on the basis of control engineering (Gevers, 2006).

The typical scheme of the identification process for a linear time-invariant vibrating structure is presented in Figure 3.7. Based on the knowledge of the system's experimental response (output data) to an excitation source (input data) a parametric modal model can be derived that is defined by a set of modal parameters (eigenfrequencies, mode shapes, damping ratios). There are several deterministic or stochastic techniques developed over the past years that can be used to build the mathematical model of the dynamic structural systems in frequency or time domain based on measured data. A modal model of an artificially excited structure can be obtained based on Experimental Modal Analysis (EMA); however in case of real scale civil structures applying an artificial excitation might be difficult from technical and economical point of view. Therefore, Operational Modal Analysis (OMA) is generally preferred to forced vibration measurements since the same modal parameters can be obtained from vibration data in operational rather than laboratory conditions by modeling the interaction between the structure and its environment (e.g. wind, traffic, etc). Ambient vibration measurements are usually used to perform OMA and identify the modal parameters of a structure. In contrast to Experimental Modal Analysis, the properties of ambient excitation in Operational or Output-Only Modal Analysis are difficult or impossible to be measured. Therefore stochastic identification techniques have been developed by the assumption that the response is a realization of a stochastic process with unknown white Gaussian noise as input characterized by a flat spectrum in the frequency range of interest. Based on this assumption the excitation input is considered to have the same energy level at all frequencies implying that all modes are excited equally (Van Overschee and De Moor, 1996; Peeters, 2000).

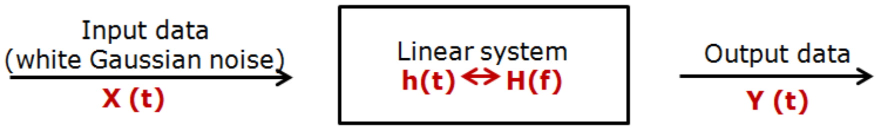


Figure 3.7: Stochastic Output-Only Identification

There are different stochastic identification techniques to extract the modal parameters of a structural system, namely the parametric and non-parametric methods. In non-parametric methods the modal parameters are estimated directly by post-processing the measured data whereas in the parametric methods the dynamic characteristics are extracted based on a parametric model that is updated to fit the recorded data. The two methods are described in more detail in the following paragraphs.

Non-parametric algorithms are traditionally associated with the Discrete Fourier Transform and the computation of auto and cross power spectra. They are widely used due to their simplicity and intuitiveness. Two of the most commonly used non-parametric methods for the modal parameters estimation of the identified system are (a) the Basic Frequency Domain or the Peak Picking (PP) method (Bendat and Piersol, 1993) and (b) the Frequency Domain Decomposition (FDD) (Brincker et al., 2000). Both methods are based on correlations of the power spectra between the outputs and the reference outputs and on decomposition of the stochastic power spectral density (PSD). In the first method the frequency peaks from the average spectra are selected that are derived based on the output recordings of the sensors. The FDD method is considered to be an improved version of the PP method and consists of decomposing the system's power spectral density into its singular values. It is shown that taking the Singular Value Decomposition SVD of the spectral matrix, the latter is decomposed into a set of auto spectral density functions each corresponding to a single degree of freedom (SDOF) system (Brincker et al., 2000).

The output PSD, $G_{yy}(j\omega)$, at discrete frequencies is decomposed by taking the Singular Value Decomposition SVD of the matrix:

$$G_{yy}(j\omega) = U_i S_i U_i^H \tag{3.5.1}$$

where $U_i=[u_{i1}, u_{i2}, \ldots, u_{im}]$ is a unitary matrix including the singular vectors u_{ij} and S_i is a diagonal matrix including the scalar singular values s_{ij}. If only one mode is dominating close to the peak then the first singular vector is an estimate of the mode shape. If two modes are dominating at this frequency peak then the two first singular vectors are estimates of the corresponding mode shapes (Figure 3.8).

These results are exact under the following conditions:

- the loading is white noise
- the structure is lightly damped and
- the mode shapes of close modes are geometrically orthogonal.

Disadvantages of the nonparametric methods are the subjective selection of the eigenfrequencies, the lack of accurate damping estimates and the determination of operational deflections shapes instead of mode shapes since no modal model is fitted to the data (Peeters and De Roeck, 1999).

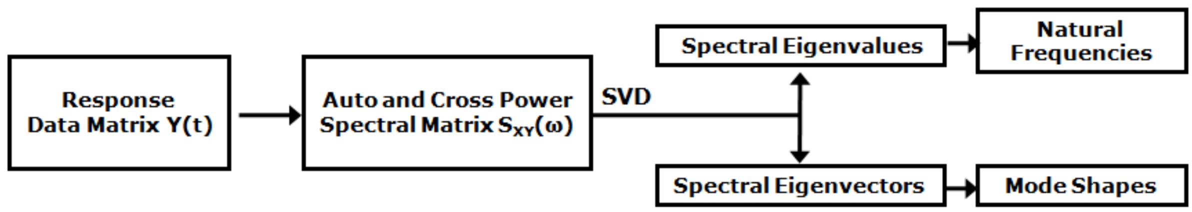


Figure 3.8: Frequency Domain Decomposition (FDD) method

Stochastic subspace identification SSI (Van Overschee and De Moor, 1991) works in time domain and is one of the most commonly applied parametric identification methods. The SSI techniques involve the selection of a mathematical model whose parameters are adjusted to the model so that it fits to the measured data. The goal of this model calibration is to minimize the deviation between the predicted and measured system response. The number of the parameters plays a significant role in the identification process. In case a too small number is defined, the modal parameters may be not modeled statistically correctly. On the other hand, if the number is defined to be too high, then the model becomes over-specified resulting in unnecessary high statistical uncertainties of the model parameters. An advantage of the parametric over the non-parametric techniques is the direct estimation of the system's damping ratio from the identification process.

SSI is based on a state space description of the dynamic problem. In fact, the second order dynamic problem, expressed through the differential equation of motion, is converted into two first order problems, namely the "state equation" and "observation equation". The stochastic state-space model of a discrete–time, linear, time invariant system is mathematically described by the following set of equations (Ewins, 1984; Peeters and De Roeck, 1999):

$$x_{k+1} = Ax_k + w_k \tag{3.5.2}$$

$$y_k = Cx_k + v_k \tag{3.5.3}$$

with

$$E\left[\begin{pmatrix} w_p \\ u_p \end{pmatrix} \begin{pmatrix} w_q^T & u_q^T \end{pmatrix}\right] = \begin{pmatrix} Q & S \\ S^T & R \end{pmatrix} \delta_{pq} \tag{3.5.4}$$

where $y_k \in \mathbb{R}^l$ is the measurement vector at time instant k of the l outputs; $x_k \in \mathbb{R}^n$ is the state vector at discrete time instant k and contains the numerical values of n states; $v_k \in \mathbb{R}^l$ and $w_k \in \mathbb{R}^n$ are unmeasurable vector signals that are assumed to be zero mean, stationary, white noise vector sequences; $A \in \mathbb{R}^{n\times n}$ is the state matrix that describes the dynamics of the system by its eigenvalues whereas $C \in \mathbb{R}^{l\times n}$ is the output matrix. The matrices $Q \in \mathbb{R}^{n\times n}$, $S \in \mathbb{R}^{n\times l}$ and $R \in \mathbb{R}^{l\times l}$ are the covariance matrices of the noise sequence w_k and v_k. The matrix pair $\{A, C\}$ is assumed to be observable, which means that all modes of the system can be observed in the output y_k and can thus be identified. E is the expected value operator and δ_{pq} is the Kronecker delta. During the stochastic identification the parametric model is defined to determine the following parameters:

- the order n of the unknown system
- the system matrices $\{A\}$ and $\{C\}$ as well as $\{Q\}$, $\{S\}$ and $\{R\}$ so that the predicted and the measured output of the model are equal.

The output measurements are gathered and rearranged in a block Hankel (data driven SSI) or Toeplitz (covariance driven SSI) matrix as past (reference) and future blocks. The Hankel or Toeplitz matrix is a matrix where each antidiagonal or diagonal consists of the repetition of the same element respectively. More specifically for the predicted output response time series expressed as a discrete data matrix the block Hankel and Toeplitz matrices are defined accordingly as:

<u>Hankel matrix</u>   <u>Toeplitz matrix</u>

$$H = \begin{bmatrix} y_1 & y_2 & \cdots & y_{n-2s} \\ y_2 & y_3 & \cdots & y_{n-2s+1} \\ \cdots & \cdots & \cdots & \cdots \\ y_s & y_{s+1} & \cdots & y_{n-s-1} \\ \hline y_{s+1} & y_{s+2} & \cdots & y_{n-s} \\ y_{s+2} & \cdots & \cdots & y_{n-s+1} \\ \cdots & \cdots & \cdots & \cdots \\ y_{2s} & y_{2s+1} & \cdots & y_n \end{bmatrix} = \begin{bmatrix} H_p \\ \hline H_f \end{bmatrix} \longleftrightarrow \quad T = H_f H_p^T \quad T = \begin{bmatrix} R_s & R_{s-1} & \cdots & R_1 \\ R_{s+1} & R_s & \cdots & R_2 \\ \cdots & \cdots & \cdots & \cdots \\ R_{2s-1} & R_{2s-2} & \cdots & R_s \end{bmatrix}$$

(3.5.5)

where H $\in \mathbb{R}^{2sl\times(n-2s)}$ and T $\in \mathbb{R}^{ls\times ls}$, s is the number of block rows and n-2s the number of block columns. The subscripts p and f stand for past and future and the matrices H_p and H_f are defined by splitting the Hankel matrix into two parts of s blocks. R_s are the covariance matrices between all outputs and references and are defined as $Rs = E\left[y_{k+s} y_k^T\right]$.

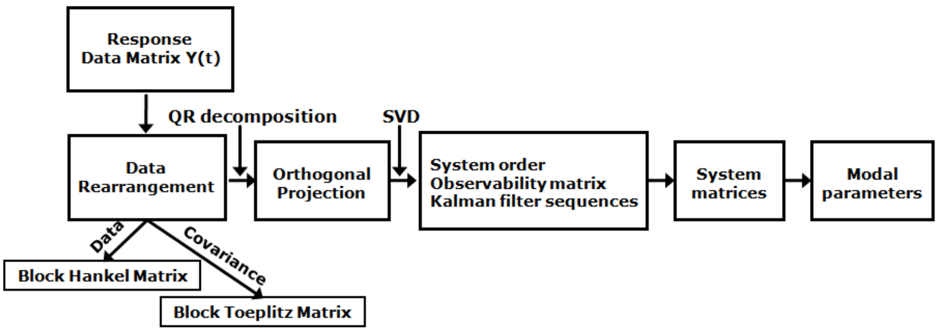


Figure 3.9: Stochastic subspace identification (SSI) method

The key step of SSI is the projection of the row space of the future outputs into the row space of the past outputs (Van Overschee and De Moor, 1996). The idea behind the projection is that it retains all the information in the past that is useful to predict the future resulting to a data order reduction. The main theorem of SSI (Van Overschee and De Moor, 1996) states that the projection can be factorized as the product of the observability matrix (that is based on the matrix pair {A, C}) and the Kalman filter state sequence. The aim of the Kalman filter (Van Overschee and De Moor, 1996; Peeters and De Roeck, 1999; Ljung, 1987; Juang, 1994) is to produce an optimal prediction for the state vector x_{k+1} by making use of the observations of the outputs up to time k and the system matrices combined with the noise covariances. Introducing a QR-factorization to the Hankel or Toeplitz matrix the projection matrix can be computed. Continuously the projection matrix is decomposed into its singular values (Singular Value Decomposition SVD) revealing the order of the system. Finally based on the estimated system order, the observability matrices and the Kalman filter state sequences, the system matrices can be calculated. Knowing the outputs, the system order and the system matrices the identification problem is solved and the modal parameters can be extracted (Figure 3.9).

3.6. Methodological framework for deriving building-specific fragility functions of RC buildings

In the context of seismic vulnerability assessment of reinforced concrete (RC) buildings, the use of field monitoring data constitutes a significant tool for the representation of the actual structural state, reducing uncertainties associated with the building configuration properties as well as many non-physical parameters (age, maintenance, etc.), enhancing thus the reliability in the risk assessment procedure.

A schematic flowchart of the proposed methodological framework for the derivation of building - specific fragility functions of RC buildings based on field monitoring data is presented in Figure 3.10.

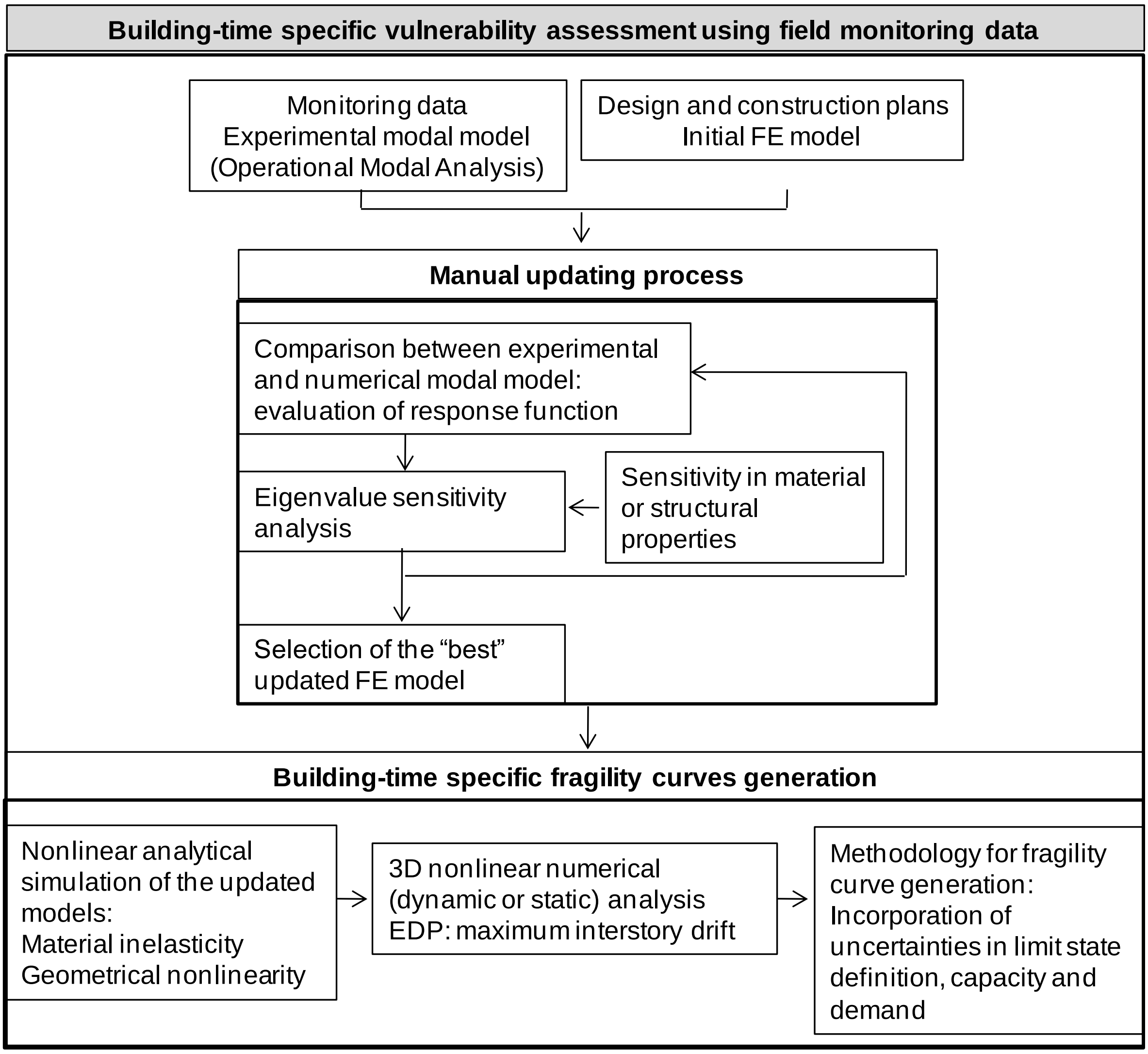


Figure 3.10: Methodological framework for the derivation of building – specific fragility curves of RC buildings.

Ambient noise measurements are used to derive the experimental modal model of the hospital building and identify its modal properties based on operational modal analysis (OMA). The modal identification results are used to update and better constrain the initial finite element model of the building, which is based on the design and construction documentation plans. Model updating aims at the “correction” or “update” of the initial finite element model based on data processing, obtained from measurements conducted on the test structure (Mottershead and Friswell, 1993). The main purpose is to modify iteratively updating parameters to result in structural models that better reflect the measured data than the initial ones. One of the key issues during the updating process is the selection of the appropriate updating parameter. In general, if no major geometrical modifications are identified, structural features such as material or mass properties, are likely to be selected as updating parameters in order to increase the correlation between the observed dynamic response of the structure and the predicted from the numerical modal model (Scodeggio et al., 2012). Other parameters such as soil-foundation-structure interaction, foundation conditions and the connection between structural elements, which influence the modal properties, may contribute in the updating process; however, the process might include high uncertainty levels and additional tests required for their determination (e.g. non-destructive tests).

In the present methodology a manual updating scheme is proposed to be applied considering only a limited number of parameters, which however allows a good observation of the process in order to gain complete insight on the effects of the sensitivity parameters on the structural behavior. The updating procedure consists of an eigenvalue sensitivity analysis of the elastic numerical modal models in order to identify the most sensitive parameters influencing the structural modes of interest, which are used in the manual updating process to define the optimal analytical models that reflect the experimental results. The selection of the best updated finite element (FE) model of the building is made by evaluating an appropriate response correlation function between experimental and numerical results (e.g. Modal Assurance Criterion, Allemang and Brown, 1982). Finally three-dimensional nonlinear numerical (either static or dynamic) analyses of the nonlinear updated models are performed in order to estimate the failure mechanism of the structure and derive the building-specific fragility functions.

IDA is an emerging analysis method which involves performing a series of nonlinear dynamic analyses under a suite of multiply scaled ground motion records whose intensities should be ideally selected to cover the whole range from elasticity to global dynamic instability (Vamvatsikos and Cornell, 2002).

In order to perform IDA a representative set of accelerograms needs to be selected. The set of seismic records can be selected using existing strong ground motion databases such as the European Strong-Motion Database (http://www.isesd.hi.is) based on a reference acceleration spectrum that reflects the site-specific hazard conditions. Additionally the soil classification of the site should be defined as well as the moment magnitude Mw and epicentral distance R range of interest. In order to eliminate potential source of bias in structural response, the selection of pulse-like records should be avoided. The primary selection criterion is the average acceleration spectra of the set to be of minimal “epsilon” (Baker and Cornell, 2005) at the period range of interest with respect to the defined reference spectra corresponding to the median of the Mw and R selection bin. Epsilon is computed by subtracking the mean predicted logarithmic spectral acceleration that corresponds to the fundamental structural period $\ln S_a(T_1)$ from the record’s $\ln S_a(T_1)$ and dividing it by the logarithmic standard deviation as estimated by the prediction equation (Baker and Cornell, 2005). In order to achieve minimal “epsilon”, the set mean spectrum is constrained to match the mean S_a prediction with a tolerance dependent on corresponding variance of S_a. The selection procedure can be optimized using the REXEL software. (Iervolino et al., 2010).

IDA curves of the structural response, which provide the relationship between a damage measure quantity (i.e. engineering demand parameter EDP) and a scalable intensity measure (IM) of the applied scaled accelerograms, can be constructed by interpolating the resulting EDP-IM discrete points. Within the framework of this methodology the most commonly used EDP is the maximum interstorey drift ratio maxISD, which is known to relate well to dynamic instability and structural damage. More specifically, the maximum peak SRSS drift in the two principal directions is selected (Wen and Song, 2002) which represents the maximum over all stories of the peak of the square-root-sum-of-squares of each storey’s drift. The seismic intensity on the other hand is usually described using either the spectral acceleration corresponding to the fundamental mode of the structure $S_a(T_1, \xi\%)$ or the peak ground acceleration PGA. It should be noted that the fragility curves are derived using building-specific damage state limits that are defined based on the results of the analyses. In particular two limit states are defined in terms of interstorey drift ratio, representing the immediate occupancy (IO) and collapse or near collapse prevention (CP) performance levels. The first limit state, namely the Immediate Occupancy corresponds to the yielding point where the elastic branch gives place to the post-elastic branch. The second limit state is assigned at a point where the IDA curve is softening towards the flat line, but at low enough values of maxISD so that we still trust the structural model (Vamvatsikos and Cornell, 2004).

The results of the IDA (IM-EDP values) are used to derive the fragility curves expressed as two-parameter lognormal distribution functions:

$$P[DS/IM] = \Phi\left(\frac{ln(IM) - ln(\overline{IM})}{\beta}\right) \qquad (3.6.1)$$

where, Φ is the standard normal cumulative distribution function, IM is the intensity measure of the earthquake, $\overline{IM}$ and β are the median values (in units of g) and log-standard deviations respectively of the building fragilities and DS is the damage state.

The median values of IM corresponding to the prescribed performance levels are determined based on a regression analysis of the nonlinear IDA results (IM – EDP pairs) for each structural model. More specifically, in accordance to previous studies (e.g. Cornell et al., 2002), a linear regression fit of the logarithms of the IM - EDP data which minimizes the regression residuals can be adopted for the particular analysis cases.

The various uncertainties are taken into account through the log-standard deviation parameter β, which describes the total dispersion related to each fragility curve. Three primary sources of uncertainty contribute to the total variability for any given damage state (NIBS, 2004), namely the variability associated with the definition of the limit state value, the capacity of each structural type and the seismic demand. The uncertainty in the definition of limit states is defined on the IDA curves while the variability of the capacity is assumed to be 0.25 and 0.3 for the high and no/low code structures respectively (NIBS, 2004). The third source of uncertainty associated with the demand, is taken into consideration by calculating the dispersion of the logarithms of IM - EDP simulated data with respect to the regression fit. Under the assumption that these three variability components are statistically independent, the total variability is estimated as the root of the sum of the squares of the component dispersions.

4. Case study: Philosophical Faculty Building of AUTH

4.1. Data collection

According to the information provided by AUTH, the building of the Philosophical Faculty was designed and constructed in the 1960s. The main façade of the existing building (a) as well as original design drawings, including the plan of the ground floor (b) and the longitudinal cross-section (c) are shown in Fig. 4.1.

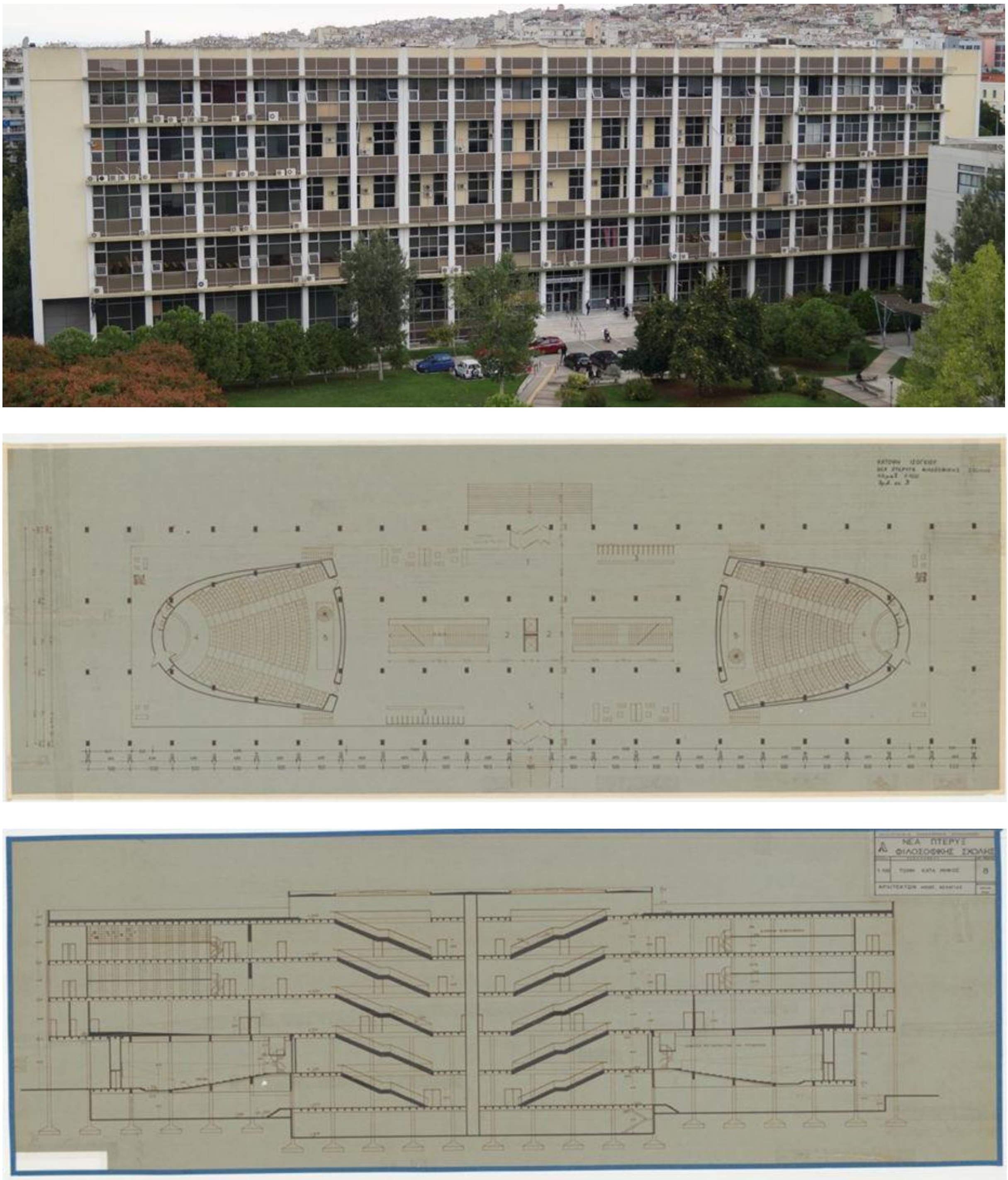


Figure 4.1: Main façade (a), plan of the ground floor (b) and the longitudinal cross-section (c) of the building of the Philosophical Faculty of AUTH

The construction documentation provided by the AUTH partners included some architectural drawings and limited specifications, containing information about the building and its parts. However, important to note, that the available documentation was not complete (in particular, information about some structural details and properties of materials was not available) and moreover (as it was found out in the course of the in-situ measurements), a considerable part of the preliminary available information was incorrect.

The measurement campaign in Thessaloniki (which included investigation of two buildings in the campus of AUTH) was conducted in September/October 2015 jointly by the groups of AUTH, GFZ-Potsdam and TU-Berlin. The activities included visual survey and inspection of the structures and were accompanied by short-term ambient vibration measurements.

As mentioned above, some considerable discrepancies (in particular, in cross-sectional dimensions of structural elements) in comparison with the original drawings were detected during the conducted in-situ measurements. Moreover, one additional floor was detected on site in comparison with the original drawings (Fig. A1, c). It was found out later that the building was modified (allegedly in the 1970s); however there is no related documentation.

During the survey, for the geometry measurements (including both overall dimensions of the building and member sizes) we used conventional measure tapes and a laser distance meter (Fig. 4.2).

Figure 4.2: Laser distance meter HILTI PD5

For investigating details of the existing concrete elements for scanning concrete members to detect the location of reinforcement bars and determine the depth of concrete cover we used the HILTI PS 50 Multidetector (Fig. 4.3).

Figure 4.3: Detecting of ferrous metal in a concrete column with the use of HILTI PS 50 Multidetector

The vibration measurements were performed using 38 CUBE digitizers connected to 4.5Hz geophones, shown in Figure 4.4.

Figure 4.4: Geophone and data logger

Considering the complex structure of the building, in particular, irregular spatial distribution of supporting columns in the side units (Fig.A1, b), for the simplified structural analysis we decided to focus on the central unit only. Keeping this purpose in mind, the measurements were concentrated on the central unit (monitored with sensors placed at the four corners of 1st, 2nd, 3rd, and 4th floors; two sensors were installed on the roof, and two in the underground basement). The two side units were also monitored, but less densely (the sensors were installed at the four corners of the 1st and 4th floors, and one sensor in the semi-basement). The spatial arrangement of the system of installed sensors is presented schematically in Fig. 4.5.

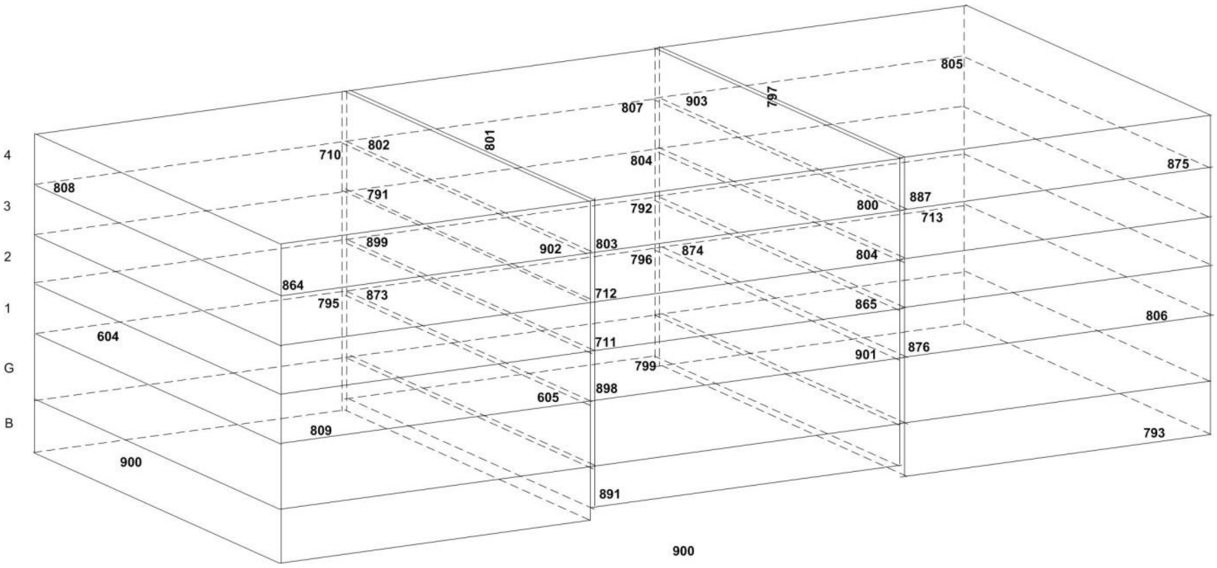


Figure 4.5: Schematic representation of the instrumentation layout in FB

The building was monitored for about 20 hours. Such rather long duration of vibration measurements is generally not necessary for the modal analysis. In this case it was done purposing to record and compare the vibrations in the day and night periods. Commonly, for the operational modal analysis of normal buildings only few minutes of recording would be quite sufficient.

The recorded ambient vibrations were processed by GFZ partners and provided in miniSEED format. Further processing and modal analysis were implemented with the help of MACEC 3.3 software (Reynders et al. 2014) using stochastic subspace identification approach (Reynders, 2012). Several three-minute-long recordings (in the different day and night periods) were analyzed and compared. The calculations for all the analyzed recordings gave very similar output.

The three modes obtained from the operational modal analysis are presented in Fig. 4.6. The obtained fundamental frequencies for two bending modes are equal to 1.60 Hz (in the lateral direction) and 1.72 Hz (in the longitudinal direction). The frequency corresponding to the torsional mode is 1.76 Hz. The torsional mode is, however, less relevant for the seismic vulnerability assessment due to the symmetry of the considered structure. In the further analyses we focus on the central unit and consider first of all the bending modes in the structural model described below.

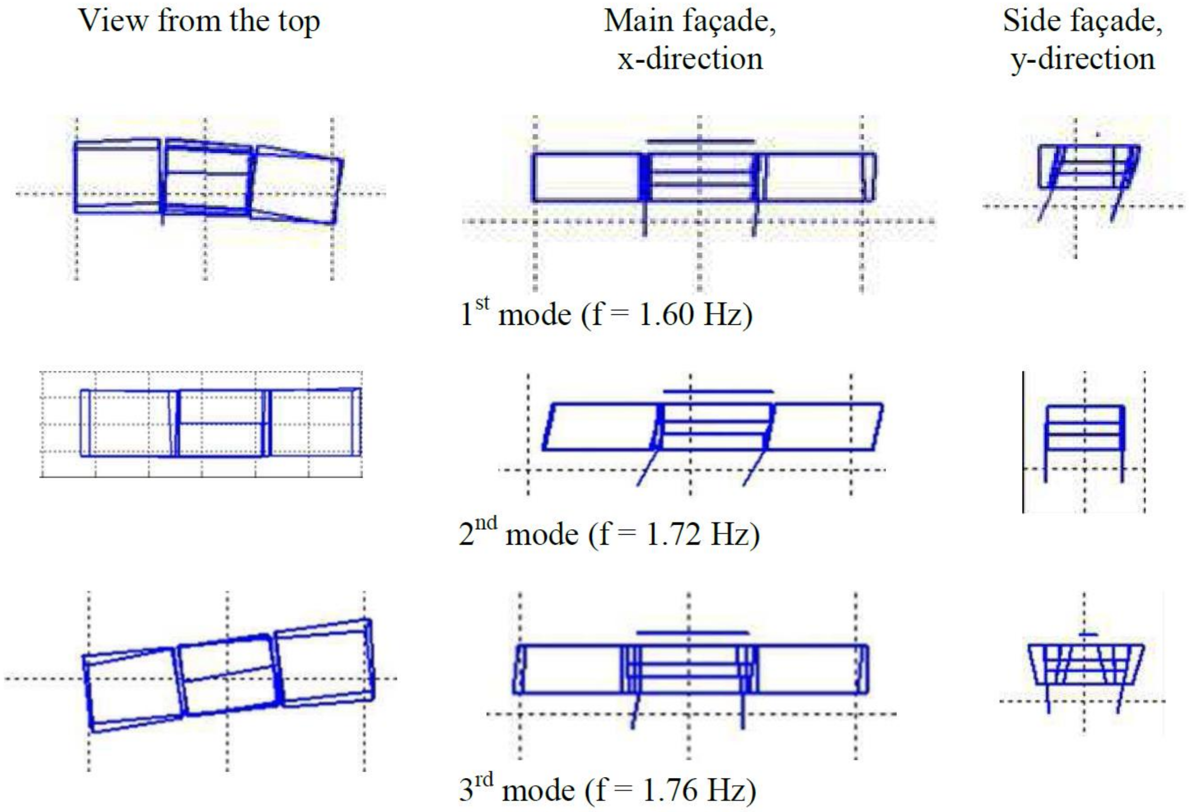


Figure 4.6: Results of operational modal analysis for the building

The collected information, required for structural modelling and seismic vulnerability assessment is briefly summarized in Table 4.1, where the used data sources are also indicated (CD – construction documentation, IM – in-situ measurements, SD – simulated design).

As describe above, an initial part of the collected information was taken from the partially available construction documentation, which, however, contained outdated and, therefore, wrong information, and was further considerably updated and modified using the actual data obtained immediately from in-situ measurements. The lacking information (concerning, first of all, structural details and material properties) was updated using simulated design; that is the required input parameters were taken according to the usual design practice for such buildings. Of course, this fact introduces an essential uncertainty into the model, which can be generally refined by comparing the measured and calculated modal characteristics and tuning the input parameters, correspondingly.

Summarizing the above, one may conclude that the amount and quality of collected input data in the considered case corresponds to the knowledge level KL1.

Table 4.1: Data collected for structural modelling and vulnerability assessment

NN	Data	Brief description	Sources used
1	Lateral load-resisting system and material of bearing structures.	Moment resisting RC (reinforced concrete) frames. The RC elevator shaft located in the center of the building.	CD, IM
2	Overall dimensions and shape of the building (including the presence and location of separation joints)	The rectangular building consists of 3 units divided by separation joints. Only the central part is analyzed.	CD, IM
3	Presence of irregularities (physical or geometrical) in plan or in elevation	Reduced stiffness of the ground floor in comparison with upper floors (possible soft-story effects). Structural imperfections due to modernization (resulting from the added top floor)	CD, IM
4	Dimensions and location of structural components (columns, walls, braces, shafts, slabs)	Collected from the drawings and partly measured on site	CD, IM
5	Cross-sectional (shape, reinforcement ratio) and the material properties (concrete and steel strength values, elastic moduli, specific density) of the structural members	Collected from the drawings and partly measured on site; updated using simulated design	CD, IM, SD
6	Presence of non-structural elements and other building components, which can contribute to the stiffness and/or mass distribution and their characteristics	Thick masonry walls (non-structural) on the upper floors	CD, IM
7	Year of construction (and modification) of the building and its previous and current occupancy (as well as importance class)	Constructed in 1960s; modified in 1970s	CD, IM
8	Current state of the preservation and physical condition of structural elements	In operational use; normal physical condition of structural members	CD, IM

4.2. Structural analysis and seismic evaluation

As mentioned above, it was assumed that the separation joints, dividing the whole building structure into three units, allow independent consideration of those units (Fig. 4.1). Therefore, only the central part of the building is considered for the structural modelling and analysis.

For constructing the building model we take into consideration that the structural system is presented by moment resisting frames, composed of reinforced-concrete columns and beams. The lateral-load resisting system is complemented with the reinforced-concrete elevator shaft located in the center of the building. There is also a system of masonry walls, defining the internal functional arrangement of the building. The walls do not belong to the support framework, however, they contribute to the stiffness and mass of the structure; therefore they have also been included to the structural model. The structural system is completed with reinforced concrete floor slabs, providing the spatial integrity of the building as a whole (Fig. 4.7). The modeled system is considered fixed at the ground level and, for the sake of simplification, we do not take into account possible influence of the underground part of the building and neglect possible ground-structure interaction effects.

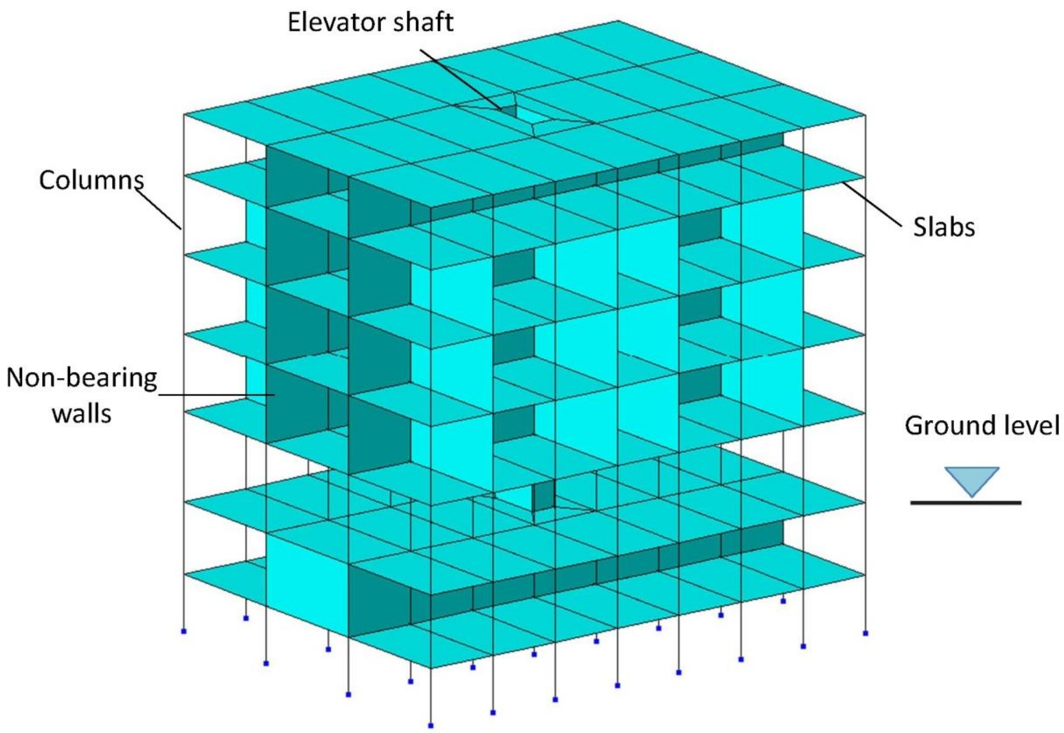
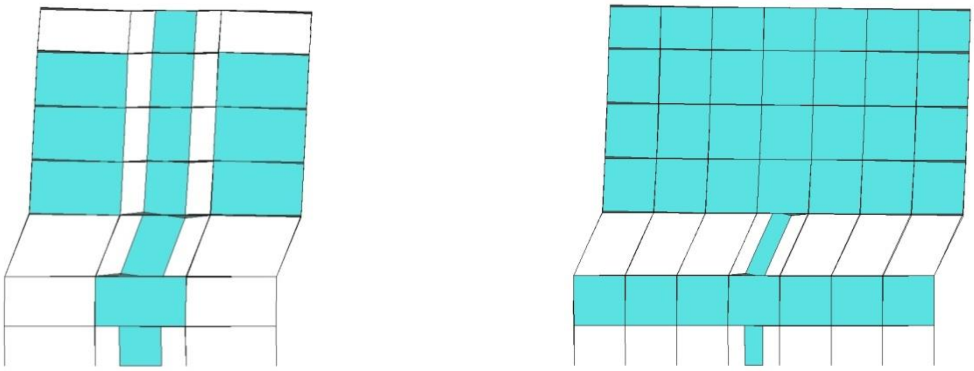


Figure 4.7: Structural model of the central unit of the building

Fig. 4.8 shows the deformed state of the building under the horizontal loading in two planar directions, where the main lateral-load-resisting elements of the structure can be seen, as well as the non-structural inner walls contributing to the stiffness of the whole system. One can also see that the presence of the weak ground floor (with reduced stiffness) generally dictates the global behavior of the building.



Transversal direction Longitudinal direction

Figure 4.8: Deformation scheme of the building

On the basis of the collected geometry, material and structural data the SISM model for the building was constructed, considering two planar directions. The torsional stiffness and, correspondingly torsional mode of vibration, is not considered.

Two bending modes, calculated from the eigenvalue problem for K and M, are shown in Fig. 4.9. They match qualitatively well the measured bending modes of the building. The calculated natural frequencies (f_{c1}=1.59 Hz, f_{c2}=1.71 Hz) are very well comparable with the measured ones (f_{m1}=1.60 Hz, f_{m2}=1.72 Hz). The relative differences with respect to the measured frequencies are negligible in view of large uncertainties mentioned above and a quite simplified modeling procedure.

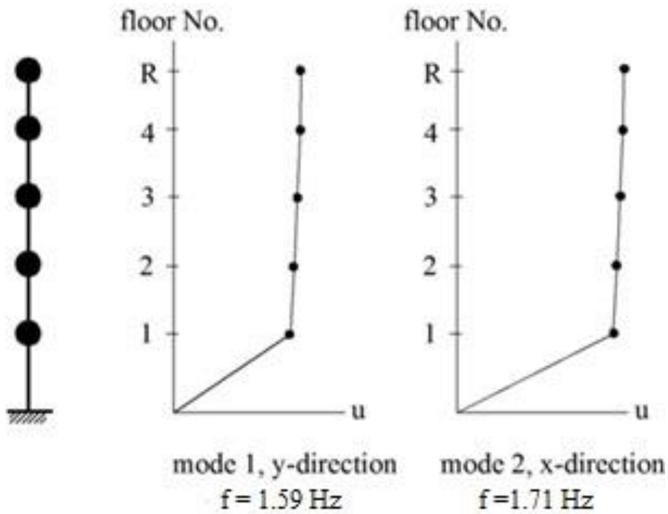


Figure 4.9: First bending modes of the SISM

According to Eurocode 8, the seismic base shear force F_b is determined using the following expression

$$F_b = S(T_1) * m * \lambda$$

where

$S(T_1)$ is the ordinate of the acceleration spectrum at the fundamental period of vibration of the building for lateral motion in the considered direction ($T_1 = 1 / f_1$);

m is the total mass of the building, above the foundation or above the top of a rigid basement;

λ is the correction factor, the value of which is equal to: λ = 0,85 if T1 < 2 TC and the building has more than two stories, or λ = 1,0 otherwise.

For calculation of seismic load we used the acceleration spectrum provided by AUTH-partners (Fig.4.10), from which the level of seismic acceleration corresponding to the fundamental vibration period of the building can be determined.

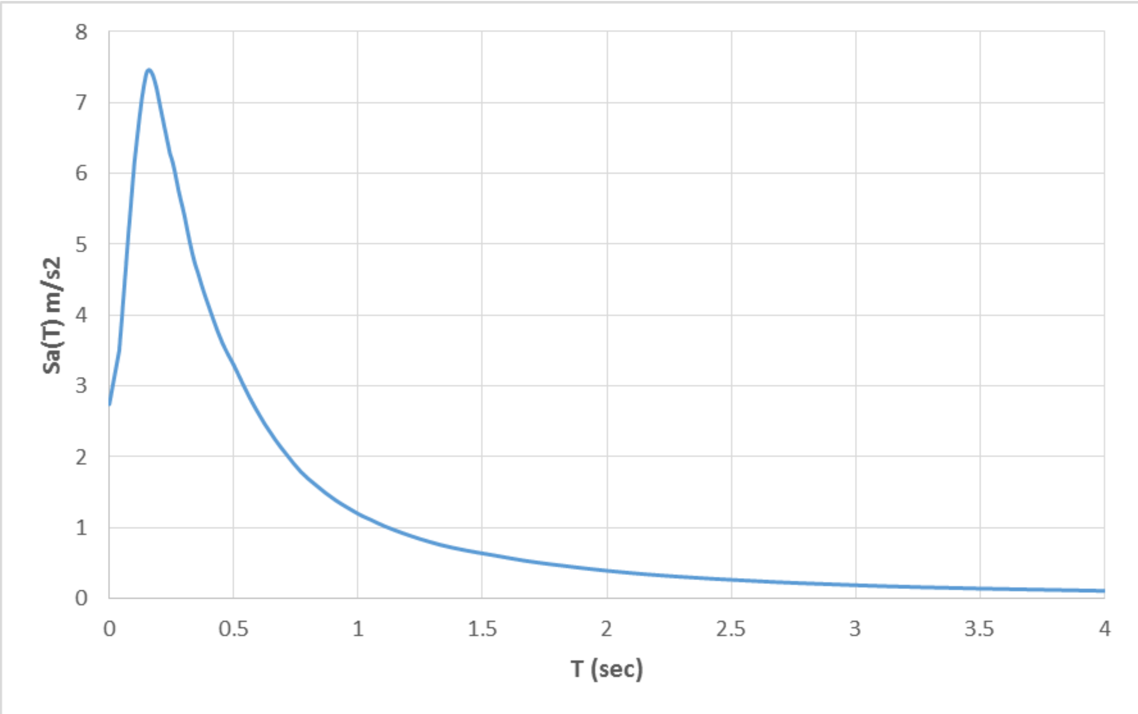


Fig.4.10. Acceleration spectrum for the investigated site

This input seismic data along with collected information about the building were used for the structural analysis and seismic assessment of the building with the help of the SISM-tool.

4.3. Findings and conclusions

During the in-situ inspection in the Philosophical Faculty building considerable discrepancies with the original design documentation were detected. First of all, the existing building (Fig.4.1, a) has one additional floor in comparison with the original design drawings (Fig.4.1, c). The measured total height of the building (from the finished floor level to the roof) is about 5 m higher in comparison with the height value specified in the design drawings. Furthermore, sample measurements of the cross-sectional dimensions of structural elements (columns and beams) made in the course of in-situ inspection, showed considerable changes in comparison with the data from the available drawings. For example, the cross-section of all the external raw columns of the ground floor in the drawings is indicated as 40 x 60 cm, while actually the existing columns are considerably thicker (the measured dimensions of the rectangle cross-sections of those columns range from 49 x 74 to 50 x 76 cm).

Unfortunately, the scanning of the structures with the multi-detector did not allow identifying the exact reinforcement schemes of the considered structural cross-sections (the device showed presence of ferrous metal elements though at deeper layers than indicated in the drawings). The lacking information about the reinforcement ratio and mechanical properties of the structural and non-structural components was updated using simulated design according to the usual design practice for such buildings.

Considering the available information, one may conclude that after completion of construction, the building was redesigned and structurally modified (one floor was added, in consequence of which the bearing structural elements were strengthened). In this situation, an extended in-situ inspection would be necessary to collect the information required for the structural modeling of higher knowledge levels.

Obviously, such essential structural modification would not only considerably change the vibration parameters of the building, but also influence its seismic vulnerability and, therefore, any computational analyses solely based on the original design drawings of the building, neglecting the structural modifications made after the construction, would produce misleading results and, hence, inadequate decisions. This fact emphasizes the crucial importance of in-situ inspections for assessment of actual structural vulnerability of existing buildings.

The results of the operational modal analysis show that the separation joints are only in part efficient and there is still certain coupling between the central and side units of the building at the level of the added floor (these effects, in particular, can be seen from consideration of the higher modes). Most probably, this can be the consequence of the structural modernization of the building, which is prompted also by the cracks found at the locations close to the separation joints. More detailed investigation of this structural imperfection would be required.

Summarizing the above, one can say, that in the considered case, the available and collected information corresponds to the knowledge level KL1. For this knowledge level for building assessment we used the developed SISM-based approach and the Excel tool. The numerical results obtained with the use of the Excel tool show that at the given level of seismic hazard at the level of the ground floor one may expect occurrence of the limit state 3 (failure of the structure), while the upper floors perform much better. In particular, the results obtained for the X-direction show that only the LS1 (cracks in concrete) is achieved in all upper floors, while for the Y-direction the LS2 (yielding in reinforcement bars) is achieved of the second and third floors and LS1 on the fourth and the fifth floors of the building (Fig 4.11).

Limit state assessment

X-direction

Story	EQ Force LS1 [MN]	EQ Force LS2 [MN]	EQ Force LS3 [MN]	LS1 Force [MN]	LS2 Force [MN]	LS3 Force [MN]
1	246.1182413	94.4290395	29.03190683	4.520715377	21.4232546	22.75822563
2	205.5056892	77.39480153	23.47226757	28.14041007	296.2669694	1074.037221
3	157.3586695	58.40913941	17.52149768	23.87799179	283.7937701	1293.642951
4	104.8435583	38.53019631	11.46952807	19.61557341	271.4026172	1592.061831
5	50.50540804	18.46710774	5.476084829	24.53039057	423.8126362	3270.811672

Y-direction

Story	EQ Force LS1 [MN]	EQ Force LS2 [MN]	EQ Force LS3 [MN]	LS1 Force [MN]	LS2 Force [MN]	LS3 Force [MN]
1	290.3190683	157.7165873	81.36970425	6.87046925	33.16692626	34.90576826
2	250.8568525	134.6503997	67.90313338	19.68146187	133.9094731	313.1754795
3	197.1905844	104.9808544	52.00672968	16.74463876	124.0534153	385.7307694
4	133.7174304	70.84212122	34.66057665	13.80781562	114.2411647	502.4295424
5	64.91948986	34.30243282	16.67741521	18.33726608	187.1751682	1280.781388

Figure 4.11: Results of building assessment with the use of the SISM-Tool

Having summarized the collected in-situ information and obtained numerical results one may conclude that the presence of the weak ground floor makes the building seismically vulnerable; therefore strengthening of the bearing structural elements would be recommended.

One should keep in mind that in our simplified assessment only the central part of the building was considered, neglecting possible influence of the side units of the building on its performance as a whole. Furthermore, we have to take into account considerable uncertainty in the input parameters inasmuch as we had to use simulated design due to the lack of reliable information about actual parameters of the building. Nevertheless, our estimation obtained with the use of simplified approach reflects the in-situ findings, in particular, the presence of soft-story and the predicted damage mechanism in case of a strong earthquake seems to be plausible.

For the case of higher knowledge level (KL2), additional investigations should be conducted aimed at collecting more detailed information about parameters of the building and more sophisticated methods of analysis should be used. Some results of such calculations obtained for the building of the Philosophical Faculty are presented in Deliverable DC4.

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