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A REVIEW ON THE STUDY OF HISTORICAL STRUCTURES USING INTEGRATED INVESTIGATION ACTIVITIES FOR SEISMIC SAFETY ASSESSMENT. PART II: MODEL UPDATING AND SEISMIC ANALYSIS

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ABSTRACT

This paper represents a continuation of a review on the seismic safety assessment of historical structures using some selected integrated investigation activities. In the first part of this review the dynamic investigation was presented. Here a focus is made on the updating of the numerical models used in the assessment phase and the used techniques in the seismic evaluation.

KEYWORDS: Historical Structures, Model Updating, Seismic Assessment, Safety Evaluation

1. INTRODUCTION

Currently, the usage of numerical models to assess the seismic safety of historical structures is gaining increasing interest. However, these models need a significant amount of information for their preparation. The use of inspection techniques such as coring, flat jack tests, thermovision, sonic tomography, etc. is not enough, in some cases, to obtain all the desirable information due to the variability of the materials and the influence of previous alterations and repairs. In general and especially when the information gathered to build the model is judged too limited, the models have to be validated, at the global level, by comparison with experimental evidence. This validation can be carried out by comparing the predictions of the model with results obtained related to the performance of the structure under known mechanical or environmental actions. This process is called numerical model updating.

Afterwards, the updated model is used in the seismic analysis. In this step, different approaches may be used. The two most common techniques are nonlinear static analysis and nonlinear dynamic analysis, being the former is more widely used than the latter.

To evaluate the structural performance of the assessed historical structure, simple methods like the N2 and the capacity spectrum could be utilized. Based on this evaluation, any necessary strengthening intervention could be proposed. The updated numerical model could be used as a virtual laboratory in which the proposed intervention could be simulated to reveal its adequacy and efficiency before any real implementation.

This research aims at providing a state-of-the-art review on the evaluation of the seismic safety of historical structures carried out using integrated investigation activities. It is presented in two-part paper. In the first part, the reviewed items are the dynamic identification of historical structures, the dynamic monitoring of historical structures, and the modal parameters identification. In this second part, the updating of finite element models of historical construction, and the seismic assessment of historical structures are discussed.

2. UPDATING OF FINITE ELEMENT MODELS OF HISTORICAL CONSTRUCTION

2.1 Introduction

Finite element (FE) method is nowadays widely used to model historical construction. Many relevant references can be found for this method, among them (Rao, 2005; Entwistle, 2001; Huebner, 2001; Zienkiewicz and Taylor, 2000). The method has large capabilities in modeling complex geometries and different materials constitutive models and to perform nonlinear analyses whether static or dynamic. However, creating a reliable FE model of a historical structure is a difficult task due to the challenges usually involved in this class of structures. These challenges are related to different aspects like geometry, construction materials, boundary conditions, existing damage and previous repairs. Therefore, there is a need to update (calibrate) the historical construction FE model against possible modeling inaccuracies and uncertainties. The process of calibration by matching the FE model outputs with the experimentally measured data is called model updating. The experimental data can be obtained from static load tests, dynamic identification tests or a combination of both. This section discusses the different approaches of FE model updating. Some case studies of updating FE models of historical construction based on results of dynamic identification tests are then given and critically reviewed.

2.2 *Philosophy of FE model updating*

When creating a FE model it is usual to make some simplifying assumptions. Likewise, boundary conditions and connections between different structural parts are not modeled with complete certainty. In addition, the FE method is based upon the material properties (Young's modulus, mass density, etc.) and the physical dimensions of the system under test. The shape function of the chosen elements determines the distribution of the mass and stiffness properties, so that the terms in the mass and stiffness matrices can be understood physically. However, alternative elements are available with different shape functions and for that reason the FE models are meaningful but non-unique. Consequently, the analyst may need to examine the sensitivity of the FE model results to changes in the mesh configuration and/or boundary constraints. Ultimately, he settles for a model which will be likely to provide acceptable results according to his engineering judgment. Limitations and errors are associated also with experimental testing. For instance: electronic systems can generally introduce low levels of instrument noise, piezoelectric accelerometers lack linearity at low frequencies, and noise can arise from accelerometer cables. In addition, test measurements usually contain fewer modes than the order of the identified model and therefore are said to be incomplete. Therefore, when comparing experimental and theoretical vibration mode shapes, the latter generally contain more points than those available from the former. The problems introduced by incompleteness are clear in large structures where it is expensive to take measurements at a large number of locations and to process large volumes of data (Mottershead and Friswell, 1993).

The rule of the model updating process is to modify the mass, stiffness and damping parameters of the FE model to obtain better agreement between numerical results and experimental data. One important aspect of FE model updating is that there exists more confidence in the experimental dynamic data than in the FE model itself. It is clear that the improved agreement in results should be achieved by correcting the inaccurate modeling assumptions and not by making other physically meaningless alterations to the model. Several techniques have been developed whereby FE models of structures are altered so that their dynamic characteristics become a closer match of experimentally determined behavior. At the most simple, it is very common to make a small number of changes to the overall properties of a FE model in a number of iterations. This type of process involves a large amount of intervention from an engineer to assess the level of improvement in the dynamic predictions of the FE model (Mottershead and Friswell, 1993; Greening, 1999).

2.3 Methods of FE model updating

The FE model updating methods can be divided into direct methods and indirect (iterative) methods (Ewins, 2000). In the following, these methods are briefly presented. For a more in-depth review, the reader is referred to several available publications on the subject like those of Mottershead and Friswell (1993), Friswell and Mottershead (1995) and Rad (1997). In their specific state-of-the-art research on the applications of FE model updating to the masonry monuments, Atamturktur and Laman (2012) classify the updating approaches into deterministic and stochastic. The deterministic approach is subdivided to manual and automated methods. These are also presented hereinafter.

2.3.1 Direct methods

Direct methods depend on adjusting individual elements in the structure mass and stiffness matrices by direct comparison between measured data and initial predictions of the FE model (Ewins, 2000). Some examples for the direct methods are: the matrix mixing method (Ross, 1971), the Lagranage multiplier method (Baruch and Bar Itzhac, 1978), and the error matrix method (Sidhu and Ewins, 1984).The main advantages of these methods are: assured convergence, less computational time compared with indirect methods and exact reproducing of the reference data set (Rad, 1997). On the other hand, there are two main disadvantages. First, a reduction in the number of DOF of the FE model has to be made because the measured DOF (measurement points) are usually less than the numerical ones. Second, the updating of the structure matrices is performed without involving a physical meaning of the resulting state, and it's difficult to control the results because changes are not directly related to structural parameters (Jiménez-Alonso and Sáez, 2011).

2.3.2 Indirect (iterative) methods

In these methods changes are made to specific physical or elemental properties in the FE model searching for an adjustment which makes measured and predicted data closer. These methods are more acceptable than the direct methods because the parameters which they adjust are physically realizable quantities (Ewins, 2000). Some of the indirect methods are the minimum variance method (Collins and Young, 1972), the inverse eigen sensitivity method (Collins et al, 1974) and the eigen dynamic constraint method (Ibrahim et al., 1989; Lin, 1991). The advantages of indirect methods are that both measured data and FE model data can be weighted, a feature which can accommodate engineering judgment; and that a wide range of parameters can be updated simultaneously (Rad, 1997). The main disadvantages are that the experimental and theoretical modes must be paired from the beginning of the updating process and a faced problem here is that there is no guarantee that all modes can be matched; the FE and identified mode shapes should be scaled correctly because the mass distribution of the FE model and that of the actual structure may be different (Rad, 1997).

2.3.3 Deterministic model updating approach

This approach assumes that both FE model and dynamic investigation measurements are deterministic. It aims at determining the most probable values for uncertain input parameters by comparing FE solutions against in situ measurements. The bridge between FE solutions and measurements are comparative features (like modal parameters: natural frequencies and mode shapes).The model updating is an inverse problem in which ill conditioning is a potential problem if the quality or quantity of the comparative features is insufficient. The success of FE model updating depends not only on selecting the right comparative features but also in updating the right parameters. The updating parameters must be selected according to the combined effects of parameter uncertainty and parameter sensitivity. Parameter uncertainty can be determined from a prior knowledge of the historical structure or from laboratory testing of some specimens taken from it. The sensitivity of the FE model parameters can be determined by a sensitivity analysis which aims at measuring the changes in the model outcomes due to a unit change in the model input. After identifying the comparative features and calibration parameters, the model updating is a matter of changing the updating parameters based on the functional relationships between the measured and calculated comparative features. The FE model inherent properties that can be calibrated are directly related to the quantity, quality and type of comparative features. Successfully and widely used comparative features are the modal parameters because they contain global information about the structure mass and stiffness. Deterministic methods can be classified into manual or automated ones as discussed in the following paragraphs (Atamturktur, 2009a).

Manual FE model updating is a trial-and-error based approach which calibrates selected parameter values based on engineering judgment. This approach can be justified when the initial model is a close representation of reality. In this case, usually after calibration the parameters are only minimally adjusted and they maintain their physical meaning. It is an appealing and convenient approach for calibrating FE model parameters because it incorporates engineering judgment into the updating process. Thus, it keeps the updated model from converging to an unrealistic model. Manual FE model updating proved to be successful when deficiencies arising from imprecise parameters are independent and uncorrelated. However, it is possible that, using manual updating, the existing hidden dependencies between input parameters can be revealed, if these dependencies are strong, this will raise the problem that updating one parameter compensates for imprecision in another. Also, due to its nature, manual updating cannot include uncertainties. As a result, in the presence of several sources of uncertainty, manual updating of material properties will likely compensate for the errors introduced by an inappropriate boundary condition (Atamturktur et al., 2010).

In automated approach the FE model updating is carried out by constructing a series of loops based on optimization procedures or Bayesian inference (Atamturktur and Laman, 2012). An optimization scheme is used through which a number of updating parameters are modified to minimize an objective function. The objective function is a formulation of the differences in dynamic behavior between the experimental data and the FE model. This is recalculated at each stage of the iteration. The method is iterative with changes made to the FE model at each step. Limitations upon the amount of information available from dynamic tests reduce the number of updatable parameters (Greening, 1999).

2.3.4 *Stochastic model updating approach*

This approach is more realistic than the deterministic approach because a FE model contains uncertainty in its input parameters (material properties, dimensions of cross sections, boundary conditions, etc.). Also, dynamic identification tests contain uncertainty in their measurements. The concept of this approach is to reach a statistical correlation between the FE model and dynamic measurements by formulating the FE model input parameters and FE model output response probabilistically (Atamturktur et al., 2010). In general, the treatment of uncertainty and quantification of errors is a two-step process. In the first step, the identification of all uncertainty and error sources is carried out. In the second step, the assessment and propagation of the most significant uncertainties and errors is carried out to obtain the predicted response quantities (Mares et al., 2006). More information about the theoretical background of this advanced technique can be consulted at Beck and Katafygiotis (1998).

2.4 *Experimental and numerical data correlation techniques*

Correlation techniques are a mixture of visual and numerical means to identify the differences between experimental and numerical modal parameters, in specific natural frequencies and mode shapes. Whereas numerical correlation techniques return a numerical value, visual means of correlation are subjective and of qualitative nature. Some of the basic correlation tools include simple tabulation or plotting of measured and predicted natural frequencies. When plotting the relation between experimental and numerical natural frequencies, perfectly matched numerical frequencies should lie on a 45° line. On the other hand, in case that the points scatter around the 45□ line, this means lower matching (Grafe, 1999). In numerical terms, for the mode number (i) , the frequency discrepancy (D_f) between the experimental frequency (f_i^e) and the numerical frequency (f_i^n)can be defined as (Gentile et al., 2009; Gentile and Saisi, 2007:

$$D_f\ (\%) = 100\left|\frac{f_i^e - f_i^n}{f_i^e}\right| \qquad \textit{Equation 1}$$

For mode shape vectors correlation, the Modal Assurance Criterion (MAC) (Allemang and Brown, 1982) is most widely used:

$$MAC = \frac{\left|\sum_{i=1}^{n} \varphi_i^e \varphi_i^n\right|^2}{\sum_{i=1}^{n} \left(\varphi_i^e\right)^2 \sum_{i=1}^{n} \left(\varphi_i^n\right)^2} \qquad \text{Equation 2}$$

where: φ_i^e is the experimental mode shape vector and φ_i^n is the numerical mode shape vector.

Some other assurance criteria include the coordinate modal assurance criterion (COMAC), the frequency response assurance criterion (FRAC), coordinate orthogonality check (CORTHOG), frequency scaled modal assurance criterion (FMAC), partial modal assurance criterion (PMAC), scaled modal assurance criterion (SMAC), and modal assurance criterion using reciprocal modal vectors (MACRV) (Allemang, 2003).

2.5 *Case studies using different updating approaches*

There are many case studies in literature about model updating of FE models of historical construction. The following paragraphs present a summary on some of these case studies.

2.5.1 *Indirect (iterative) approach*

Aoki et al. (2007) used the Inverse Eigen sensitivity Method (IEM) to update a FE model of a historical masonry bridge based on the results of dynamic identification tests. Four experimentally identified modes were used in the updating process. In the first phase of model updating, the FE model was composed of 24 macro blocks to represent in detail the arches, spandrels, fill materials, buttresses, abutments, and piers, Figure 1(a).Two materials were used. The first was for the arch, the spandrel wall and the piers and the second was for the fill materials. Based on tested samples from the stone and the mortar, the Young's moduli for these materials were estimated. The updating parameters were the stiffness' of the macro blocks. In an iterative process, the stiffness' of the macro blocks were adjusted by applying correction factors (Figure 1(c)) to match the experimental and numerical frequencies and mode shapes (Figure 1(d). This resulted in increasing the stiffness of the three arch stones and decreasing the stiffness of the two piers. For evaluating the stiffness correction with more detail and accuracy a second phase of model updating was considered. The arch of each bay was divided into four in length and three in width, the spandrels were divided into four according to length and the piers were divided into five along the height (Figure 1(b)).

As a result, FE model was divided into 83 macro blocks. No changes in material properties or boundary conditions were made. Again the correction factors were applied to the stiffness' of the macro blocks and after updating, the difference between the experimental and analytical frequencies was less than 0,96% for all the modes and the MAC values were more than 0,96 for the second to fourth modes. Some of the physical meanings for the updated stiffness' were the following: (1) to simulate the increased arch thickness that was not considered in the FE model, the stiffness of all macro blocks at the arch stones was increased; (2) the stiffness' of the macro blocks of the piers were reduced perhaps due to the effect of bridge-soil interaction; (3) the stiffness' of the macro blocks of the spandrels near the second pier were reduced probably due to the effect of existing cracks; and (4) probably due to the effect of the boundary condition, the stiffness of the macro blocks at the abutments and spandrels near the abutments was reduced. These interpretations were supported by visual inspection and other testing methods applied to the bridge.

The IEM method was used by Aoki et al. (2008) to update two FE models of a brick masonry chimney. Experimental dynamic analyses and various investigation tests were carried out to assess the structural stability of this chimney (Aoki and Sabia, 2005; Aoki and Sabia, 2006). The first three mode shapes in each main direction of the structure were identified experimentally. The FE models were built using solid, beam and truss elements. One model assumed fixed base and the other used more detailed modeling for foundations using truss elements to consider the rock behavior. For the two models, the updating parameters were the stiffness' of the finite elements. Two updating strategies were followed. The first considered only the natural frequencies and the second considered both of the natural frequencies and mode shapes. After updating the FE models by applying correction factors (as described in the previous case study) it was found that: (1) the stiffness' of the elements at the base for the two FE models were reduced, probably due to the effect of chimney–soil interaction; (2) the stiffness' of some corner elements were increased to simulate the effect of four iron angles at the corners because they were not considered in the FE models; and (3)the influence of considering the mode shapes in the updating process is significant.

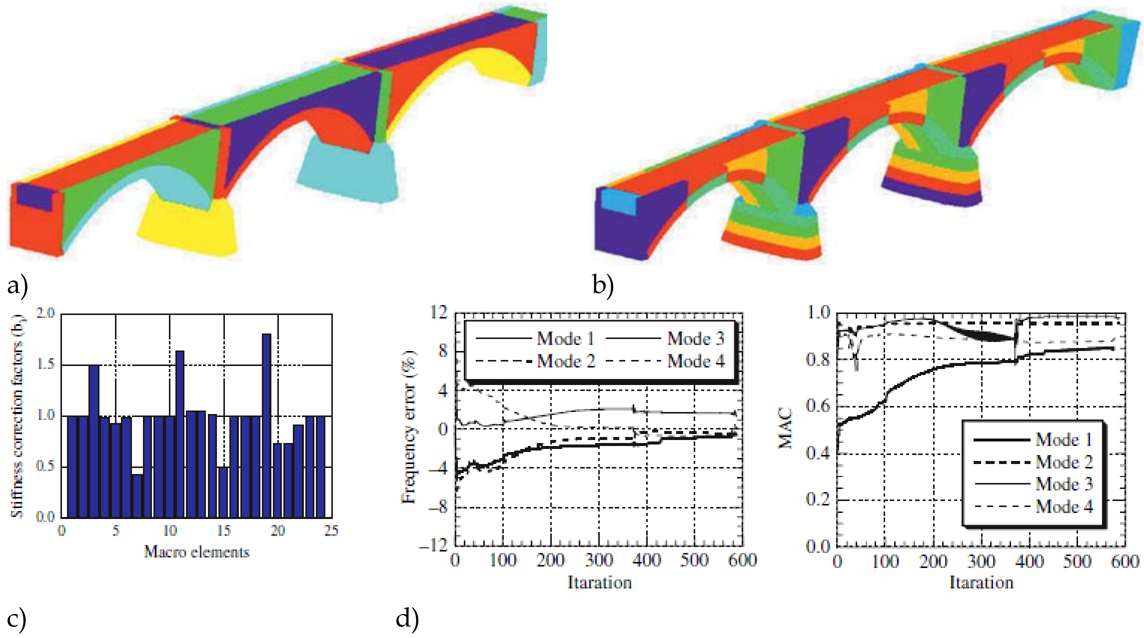


Figure 1. Using IEM in model updating of a masonry arch bridge: (a) 1st FE model; (b) 2nd FE model; (c) Stiffness correction coefficients obtained from model updating of the 1st FE model; and (d) iterative updating of the 1st FE model (Aoki et al., 2007).

2.5.2 *Manual approach*

El-Borgi et al. (2005) carried out a manual updating of a FE model of a historical palace. To determine the compressive and tensile strengths of stone and mortar, some samples were extracted; however, it was allowed to take samples from damaged parts of the external walls only. This induced a bias, because data were based on rather altered materials, and relevant to only a selected part of the palace. Therefore, the estimated compressive strength of masonry was a doubtful parameter, and also the Young's modulus which was taken as 1000 times the compressive strength. AVT was carried out and the first five natural frequencies were identified. The frequencies of the first two modes were used to update the uncertain value of the Young's modulus. To update the model, it was observed that two values of Young's modulus should be used. The first was for the external wall and the second for the internal walls. The updated frequencies were 2.5% and 5.1% away from the experimental values for the first and second modes, respectively. The compressive strengths were then estimated from the updated values of Young's modulus.

El-Attar et al. (2005) updated a FE model of a minaret. The minaret was modeled in details by introducing the internal helical stair, and modeling both the external and the internal limestone walls in addition to the filling materials between them. The base was modeled using springs. The updating parameter was the spring stiffness which depends on the soil modulus of sub-grade reaction (K_s). Three values of K_s were tried: 1K_s, 50K_s, and 100K_s. With the value of 50K_s, three numerical natural frequencies were found to be in a good matching with the measured ones.

Manual FE model updating has been widely used for architectural heritage and many references can be consulted about its application to historic buildings. These include, among many other, the studies performed on a historic church damaged by the earthquake of L'Aquila in 2009 (Casarin et al., 2011); two structures in Verona: the Cansignorio stone tomb and the Arena (Lorenzoni, 2013); two monuments in Cyprus, a church from the 16th century (Votsis et al., 2012) and cathedrals (Caselles et al., 2012; Votsis et al., 2013; Votsis et al., 2012).

2.5.3 *Automated approach*

The FE model of the church of San Torcato was updated using the automatic approach (Ramos et al., 2011; Alaboz, 2009). AVT tests were carried out and the first four modes were identified. Due to the structure complexity, it was not fully modeled. Interface elements were introduced to simulate the stiffness of the missed parts. The soil-structure interaction was defined with interface elements that reproduced the horizontal and the vertical stiffness properties of the soil. The numerical assumptions of the soil properties were based on a previous soil investigation results. The model updating procedure was carried out using four updating parameters; those were the masonry modulus of elasticity, the normal stiffness of the soil-structure interaction interface elements, the normal and shear stiffness of the interface elements of the un-modeled parts. Before the automatic model updating, the effect of each updating parameters on the calibration results were investigated manually. The effect of each parameter was studied independently from the other parameters. For the automatic updating analysis, the Douglas-Reid method (Douglas and Reid, 1982) was used. The lower and upper boundaries of the updating variables were defined based on first manual updating. Completing the updating process, it was found that the average D_f and MAC values were about 1,5% and 85%, respectively.

The automatic model updating using in specific the Douglas-Reid method (Douglas and Reid, 1982) was also used in the updating of the FE model of a masonry tower in Arcisate (Cabboi, 2014; Gentile and Saisi, 2007; Gentile and Saisi, 2004); a stone masonry church (Trujillo, 2009; Lourenço et al., 2012a); a stone tower subjected to AVT before and after repair interventions (Ramos et al., 2010; Ramos, 2007); an Indian masonry minaret (Peña et al., 2010). Other examples of automatic model updating can be checked at Pau and Vestroni (2013) and Rainieri et al. (2013).

2.5.4 *Stochastic approach*

Atamturktur (2009b) performed this type of model updating to Washington National cathedral. She carried out FVT on a typical vault of the structure and identified the first four natural frequencies with the corresponding mode shapes. The uncertainty in experimental measurements was assessed from replicating the dynamic testing. FE model uncertainty was explored via computer experiments in which different values for material properties and springs used for simulating un-modeled parts were tried. From the experimental measurements and the FE modal analyses, the natural frequencies and mode shapes were extracted probabilistically as mean and variance statistics. The uncertain parameters that were candidates for calibration were ranked based on the sensitivity of test-analysis comparative features using a Phenomenon Identification and Ranking Table (PIRT). Five parameters out of thirteen checked parameters were found to have high sensitivity on the matching process between the numerical and experimental results. Sensitivities of the first four frequencies to each one of the five calibration parameters were investigated. It was observed that the first two natural frequencies were highly sensitive to all of the five calibration parameters. The third natural frequency was sensitive to the modulus of elasticity of the lime stone—the cathedral construction material. The fourth frequency was sensitive to one the spring constants. At end, the five updating parameters were found not as a single deterministic value but as a mean value and standard deviation.

This model updating technique is not so common in the literature and only few examples can be found like Prabhu et al. (2014); Atamturktur et al. (2012); De Stefano and Ceravolo (2007); De Stefano (2007); De Stefano and Clemente (2005) and De Sortis et al. (2005).

3. SEISMIC ASSESSMENT OF HISTORICAL CONSTRUCTION

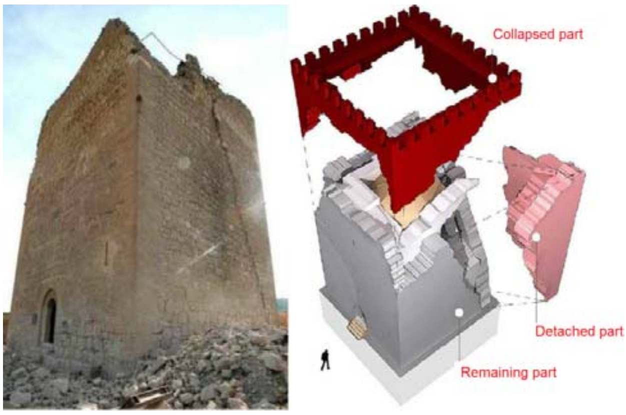
3.1 *Introduction*

In the recent years a number of catastrophic earthquakes occurred in Europe and resulted in significant damage to cultural heritage buildings. Some useful references that addressed the diastrophic effects of these earthquakes on historical structures are Romao et al. (2013), Paupério et al. (2012) and Feriche et al. (2012) who discussed the seismic damage to some churches after Lorca earthquake (Spain) in 11th May 2011. Brandonisio et al. (2013), Lagomarsino (2012), Ceci et al. (2011), Kaplan et al. (2010), Augenti and Parisi (2010), Ceci et al. (2010) showed the effects of L'Aquila earthquake (Italy) in 9th April 2009. Cattari et al. (2013), Sorrentino et al. (2013) and Bournas et al. (2013) discussed the performance of some types of structures affected by Emilia earthquake (Italy) in May 2012. Leite et al. (2013), Moon et al. (2012) and Dizhur et al. (2011) presented the damages resulted from the two recent earthquakes that hit New Zealand, those were Canterbury in 4th September 2010 and Christchurch in 22nd February 2011. The Van earthquake that struck eastern Turkey in 23rd October 2011 and the related damage was discussed in some publications like Akansel et al. (2014), Tapan et al. (2013), Korkmaz and Korkmaz (2013), Korkmaz (2013) and Ozturk (2013). Minarets, similar to other tall structures like bell-towers and spires, present very vulnerable architectural elements to earthquakes. Therefore, their seismic behavior and common collapse mechanisms due to earthquakes are always of concern, and were discussed in some publications like Cakti et al. (2013), Oliveira et al. (2012), Sezen and Dogangun (2012), Dogangun and Sezen (2012), Dogangun et al. (2008) and Dogangun et al. (2007). In Figure 2 some of the damage and collapses to historic structures are shown.

In the same context, it is clearly noted that modern societies are allocating great efforts to protect their culture heritage buildings from earthquakes. Europe, in particular, has carried out a number of research projects on the subject and closely related topics like in-situ investigation, structural monitoring and conservation of historical structures. Among them are Niker (Niker, 2010-2012), Perpetuate (Perpetuate, 2010-2012), Smoohs (Smoohs, 2008-2011), Severes (Severes, 2010-2012), Prohitech (Prohitech, 2004-2008), EU-India (EU-India, 2004-2006), Dias (dias, 2002-2005), Onsiteformasonry (Onsiteformasonry, 2001-2004), Risk-ue (Risk-ue, 2001-2004) and Chime (Chime, 2000-2003).

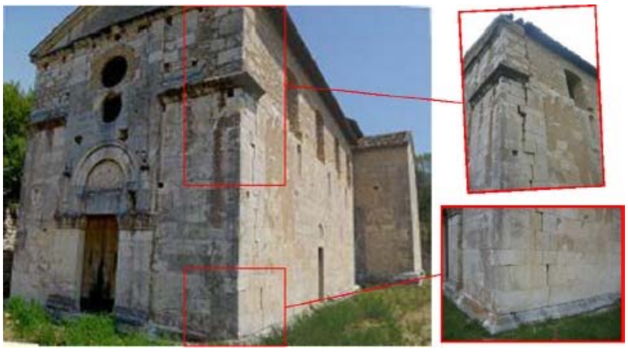
Assessing this vulnerability can be carried out using several techniques varying in complexity and time and resources demands. Using more than calculation method is preferably required to cross check the results and increase the level of confidence in the results. In the following, three techniques are discussed: the FE nonlinear static and dynamic analyses and kinematic limit analysis. To assess the seismic safety, the N2 method is presented. Finally, some case studies are shown and discussed.

a) Collapse of the transept of Santiago church (Romao et al., 2013)



b) Damage of Espolón tower (adapted from Feriche et al. ,2012)

c) Collapse of the gable of San Biago church (Brandonisio et al. , 2013)



d) overturning of façade of San Paolo a Peltuinum church (Brandonisio et al. , 2013)

e) Collapse of historic minarets in Van (Turkey) at the weaker section at balconies (Ozturk, 2013)

Figure 2. Some examples of damage to historical structures: (a) and (b) Lorca earthquake (Spain); (c) and (d) L'Aquila earthquake (Italy); and (e)Van earthquakes (Turkey).

3.2 Nonlinear static (pushover) analysis

3.2.1 General

Pushover analysis is defined by ATC-40 (ATC, 1996) as "an incremental static analysis used to determine the force-displacement relationship, or the capacity curve, for a structure or structural element. The analysis involves applying horizontal loads, in a prescribed pattern, to a computer model of the structure, incrementally; i.e. "pushing." the structure; and plotting the total applied shear force and associated lateral displacement at each increment, until the structure reaches a limit state or collapse condition".

The nonlinear static (pushover) analysis is a relatively simple structural analysis technique that aims at evaluating the expected performance of a structure under earthquakes by estimating its strength and deformation capacities. It accounts in an approximate manner for the redistribution of internal forces occurring when the structure is subjected to inertia forces that no longer can be resisted within the elastic range of the structural behavior. It involves applying a predefined lateral load pattern which is distributed along the structure height. The lateral forces are then monotonically increased in constant proportion with a displacement control at the considered control point (usually at the top of the structure) until either the ultimate condition or a certain level of deformation is reached. The target top displacement may be the deformation expected in the design earthquake (in case of designing a new structure) or the drift corresponding to structural collapse (in case of assessing an existing structure). The pushover analysis can provide valuable information about the seismic response of existing structures which includes estimates of the deformation capacities of elements that have to deform in-elastically to dissipate the seismic energy; consequences of the strength deterioration of individual elements on the global behavior of the structure; the realistic force capacity of potentially brittle elements; identification of the strength discontinuities that can lead to changes in the dynamic characteristics in the in-elastically range; and verification of the completeness and adequacy of load path, considering all the elements of the structural system, all the connections, and the foundation system (Krawinkler and Seneviratna, 1998; Mwafy and Elnashai, 2001).

The pushover analysis has been developed and reviewed over the last four decades by Freeman et al. (1975), Saiidi and Sozen (1981), Kunnath et al. (1992), Bracci et al. (1997), Krawinkler and Seneviratna (1998), Tso and Moghadam (1998), Kim and D'Amore (1999), Antoniou (2002), Themelis (2008) and Vijayakumar and Venkateshbabu (2011), among others.

The pushover analysis is described and proposed as an efficient analysis technique for design and assessment of structures by several modern codes among them: Eurocode 8 (CEN, 2004), the Spanish seismic code (NCSE-02, 2002), the Italian seismic codes (O.P.C.M 3274, 2004; O.P.C.M 3431, 2005; NTC, 2008), FEMA-273 (FEMA, 1997), FEMA-440 (FEMA, 2005), FEMA P440A (FEMA, 2009); ATC-40 (ATC, 1996), ATC-55 (ATC, 2002), IBC (IBC, 2000) and ASCE 41 (ASCE, 2007).

3.2.2 Lateral load patterns

The used lateral load patterns in pushover analysis are intended to represent and bound the distribution of inertia forces in a design earthquake. Clearly, the distribution of inertia forces will vary with the severity of the earthquake and with time within an earthquake. However, an invariant load pattern can be used assuming that the distribution of inertia forces will be reasonably constant throughout the earthquake and that the maximum deformations (obtained from this invariant load pattern) will be comparable to those expected in the design earthquake. The results are near to truth in case that the analyzed structure response is not importantly affected by higher modes of vibration and the structure has only a single load yielding mechanism that can be detected by an invariant load pattern. The use of at least two load patterns that are expected to bound inertia force distributions is recommended because no single load pattern can capture the variations in the local demands expected in a design earthquake. One should be a uniform load pattern that emphasizes the demands in lower elevations compared to the demands in upper elevations of the structure. The other could be a load pattern that accounts for higher modes effects. Nevertheless, none of these invariant load patterns can account for a redistribution of inertia forces occurs when a local mechanism forms and accordingly the dynamic properties of the structure change. Thus, it is attractive to utilize adaptive load patterns that follow more closely the time variant distribution of inertia forces (Krawinkler and Seneviratna, 1998).

Some of the proposed adaptive load patterns which try to establish equivalent lateral load distribution based on a certain theoretical basis mentioned in Jingjiang et al. (2003) are: (1) distribution proportional to the product of the mass and fundamental mode shape, which is used initially until the first yielding takes place, then the lateral forces are determined based on the product of the current floor displacement and mass at each step (Fajfar and Fischinger, 1988); (2) the adaptive distribution, which is varied as the inter story resistance changes in each load step (Bracci et al. , 1997); (3) a distribution based on mode shapes derived from secant stiffness at each load step (Eberhard and Sozen, 1993). These load patterns, however, haven't demonstrated their superiority over the simple invariant load patterns.

For design codes, the Eurocode 8 (CEN, 2004) and FEMA-273 (FEMA, 1997), for instance, recommend using at least two load patterns. The first is a uniform pattern based on lateral forces proportional to mass regardless of elevation. The second is a modal pattern able to account for higher mode effects.

3.2.3 Limitations

The main limitations of this technique can be summarized as: 1) it may not detect some important deformation modes of the structure when subjected to severe earthquakes and it may exaggerate others, i.e., if higher mode effects become important, nonlinear dynamic response may differ significantly from predictions based on invariant or adaptive static load patterns; (Krawinkler and Seneviratna, 1998) ; 2) whatever load pattern is selected, it is likely to advocate certain deformation modes (triggered by the load pattern) and neglect others that are due to the ground motion and inelastic dynamic response characteristics of the structure (Krawinkler and Seneviratna, 1998); 3) none of the invariant load patterns can account for the contributions of higher modes to response, or for a redistribution of inertia forces because of structural yielding and the associated changes in the dynamic characteristics of the structure (Chopra and Goel, 2001); 4) it is unable to account for the progressive stiffness degradation, the change of modal characteristics and the period elongation of a structure subjected to monotonic loading (Antoniou and Pinho, 2004); 5) it provides only a measure of the capacity and has to be combined with a demand measures using methods like capacity-spectrum and N2 to complete the assessment study (Elnashai, 2002).

3.3 Nonlinear time-history (dynamic) analysis

3.3.1 General

When properly implemented, the nonlinear dynamic analysis (also called nonlinear response-history analysis) provides a more accurate assessment of the structural response to strong ground shaking compared to the pushover analysis. Because the nonlinear dynamic analysis incorporates inelastic member behavior under cyclic earthquake ground motions, thus, it explicitly simulates hysteretic energy dissipation in the nonlinear range. Only the damping and other non-modeled energy dissipation need to be added as viscous damping. The analysis output is the dynamic response calculated for the input ground motions, resulting in response history data on the relevant demand parameters. Dynamic analyses for multiple ground motions are necessary because of the inherent variability in earthquake ground motions. Thus, it is possible to calculate statistically robust values of the demand parameters for a given ground motion intensity. The nonlinear dynamic analysis involves fewer assumptions than the pushover analysis; therefore, it is subject to fewer limitations. Nevertheless, the results accuracy depends on the details of the analysis model and the input ground motions, among other factors (Deierlein et al., 2010).

This analysis technique poses several challenges such as the complexity of time-integration algorithms and the difficulties in damping representation which affect the results. In addition to the dependency of the results on the characteristics of the analyzed structure, they are affected also by the nature of each earthquake record which exhibits its own peculiarities, dictated by frequency content, duration, sequence of peaks and their amplitude. The differences between the nonlinear static and dynamic analyses are summarized in *Table 1* (Elnashai, 2002; Mwafy and Elnashai, 2001).

Table 1. Pushover analysis versus nonlinear dynamic analysis (adapted from Elnashai, 2002).

	Static Analysis	Dynamic Analysis
Damping representation required?	No	Yes
Mass representation required?	No*	Yes
Additional operators required?	No	Time integration operations
Input motion required?	No	Yes
Action distribution	Fixed*	Vary in time
Computational time	Usually faster than dynamic analysis	Usually slower than static analysis

* may not be the case for adaptive pushover analysis

3.3.2 Input ground motions

The main important issues to consider when selecting the input ground motions for nonlinear dynamic analysis are (1) the target hazard spectra, (2) the source of ground motions, and (3) the number of ground motions. For the first issue, while the earthquake hazard is a continuum, codes typically define specific ground motion hazard levels for specific performance checks, i.e. the hazard is defined in terms of response spectral accelerations with a specified mean annual frequency of exceedance. For the second issue, there are three sources of ground motions (1) artificial accelerograms, (2) natural records of past earthquakes and (3) simulated accelerograms (Deierlein et al., 2010).

Artificial accelerograms were used in the past because of the lack of natural records of past earthquakes and the need to have seismic input closely representing a specific scenario to match (Iervolino et al., 2009). It was found, however, that some types of artificial accelerograms have shown inadequacy in being a realistic representation of possible ground motions (Bazzurro and Luco, 2003). On the other hand, the recently increasing accessibility to data bases of natural accelerograms recorded during real earthquakes helped significantly in using natural records (Iervolino et al., 2009). Simulated accelerograms are spectrally matched ground motions created by manipulating the frequency content and intensity of natural records to match a specific hazard spectrum (Deierlein et al., 2010). For more information about the debatable subject of selecting and scaling natural records, the reader is referred to O'Donnell et al. (2011); Iervolino et al., (2009); De Luca et al. (2009); Iervolino et al.(2008); Luco and Bazzurro (2007); Iervolino and Cornell (2005); Cornell (2005); Bommer and Acevedo (2004). Regarding the number of ground motions, typical practice is to use seven motions; however, the accurate number of motions is still a topic that needs more research (Haselton et al., 2012).

The input ground motion is doubtlessly the most important variable (more than the analytical model parameters) affecting the results and the amount of uncertainty in seismic design or assessment using nonlinear dynamic analysis (O'Donnell et al., 2011). However, it is found that the recommendations given in seismic codes about selection of input ground motions are generally poor (Iervolino et al., 2008; Bommer and Ruggeri, 2002). Haselton et al. (2012) refers this to the fact that these recommendations are based, in large part, on research of analysis of seismically isolated structures from more than 20 years ago. This occurs, in part because research on the topic is developing fast and at least a few years are required by codes to take it in (Iervolino et al., 2008). The Eurocode 8 (CEN, 2003) regulations are discussed in (Iervolino et al., 2008). This code allows employment of all three kinds of input ground motions previously discussed. It asks for matching of the average spectral ordinates of the chosen set of records to the target code-based spectral shape. To find the mean of the structural response, the set has to consist of at least seven recordings. Otherwise, if the size of the set is from three to six, the maximum response to the records within the sets needs to be considered. Little, if any, prescriptions are given about other features of the input ground motion. It seems that the code requirements have been developed having only spectrum compatible records in mind (Iervolino et al., 2008). In short, there is no general agreement in the earthquake engineering community on how to appropriately select and scale earthquake ground motions for design and seismic performance assessment of structures using nonlinear dynamic analysis (Haselton et al., 2012).

3.3.3 Damping

Any structure has some energy-dissipating mechanisms. Inelastic hysteretic energy dissipation, radiation of kinetic energy through foundation, kinetic friction and viscosity in materials are examples of energy-dissipating mechanisms in structures. Such energy dissipation or capacity is called damping and it is usually assumed to be of viscous type because of its mathematical simplicity (Otani, 1980). However, it has been shown in the literature that the actual mechanism of energy dissipation in real structures is closer to the so-called hysteretic damping than to the viscous damping (Oliveto and Greco, 2002). Damping capacity is not a unique value of a structure, but it depends on the level of excitation. The state-of-the-art does not provide a method to determine the damping capacity based on the material properties and geometrical characteristics of a structure (Otani, 1980). On the other hand, since damping can result from many sources, it is difficult to describe analytically and in a thorough way the complex physical phenomena that determine the energy dissipation (Crandall, 1970).

For masonry historical structures, in specific, there is no information on the nature of inherent damping mechanisms. Since cracking always exists in this type of structures, their damping ratios would be different from those used for modern structures. Moreover, cracking results in reduction in the masonry Young's modulus which would increase the damping level (Elmenshawi et al., 2010c).

3.4 Limit analysis

3.4.1 Background

The limit analysis is a simple tool, yet effective, for estimating the ultimate capacity of masonry structures. The method as proposed by Heyman (1966) includes three basic assumptions: (1) masonry has infinite compressive strength, (2) masonry has no tensile strength and (3) sliding failure cannot occur. The first assumption is not conservative as it may appear because collapse of masonry structures is mostly due to cracking rather than crushing (Betti and Galano, 2012). The second assumption is near to reality since very small tension forces are transferred across mortar joints (Betti and Galano, 2012). These assumptions lead to the definition of the term mechanism in which the structure fails due to the formation of hinges corresponding to disconnections and localized cracking that divide the structure into macro-elements (Castellazzi et al., 2013). Macro-elements can also be proposed based on experience gained from surveys of damage patterns of structures already experienced earthquakes. The reader is referred to the following references for more details about how to propose possible mechanisms of historical Catholic churches (Lagomarsino, 2012), Orthodox churches (Mosoarca and Gioncu, 2013), towers (Sepe et al., 2008), adobe structures (Tolles et al., 2003), masonry buttresses (Ochsendorf et al., 2004), masonry arches (Block et al., 2006), masonry vaults (Huerta, 2001) and other types of historical construction (Jaiswal et al., 2011; D'Ayala and Speranza, 2003; Augusti et al., 2001).

Today, limit analysis is used as a powerful tool able to realistically assess the safety and collapse of structures composed by blocks including arches, vaults, towers, façades and entire buildings. Notwithstanding, it can hardly be used to predict the damage for moderate or service load levels not leading to a limit condition. It should be considered as a complementary tool when performing alternative numerical analyses. Previous studies showed that, regardless the level of sophistication of the used numerical method, it will produce, at ultimate condition, results predictable by means of limit analysis (Roca et al., 2010).

3.4.2 Theorems of limit analysis

The assumptions of the limit analysis enable the application of the three limit theorems of plasticity: the lower-bound (or safe), the upper-bound (or unsafe) and uniqueness. In the first theorem, the structure is safe and the collapse is prevented if a statically admissible state of equilibrium can be found. This occurs when a thrust line can be determined in equilibrium with the external loads and falls within the boundaries of the structure. The load applied is a lower-bound of the actual ultimate load which causes failure. In the second theorem, if a kinematically admissible mechanism can be found for which the work developed by external forces is positive or zero, then the load is an upper-bound of the actual ultimate load. The application of the upper bound theorem leads to the so-called kinematic limit analysis for analyzing masonry buildings. For the last theorem, a limit condition can be reached and the structure will be about to fail if a both statically and kinematically admissible collapsing mechanism can be found. The failure configuration is reached when a thrust line can be found causing as many hinges (Hinges are caused by the thrust line becoming tangent to the boundaries) as needed to develop a mechanism. When this occurs, the load is the true ultimate load, the mechanism is the true ultimate mechanism, and the thrust line is the only possible one (Heyman, 1995).

3.4.3 Calculations of the kinematic limit analysis

The collapse multiplier (α_0) is calculated from Figure 3 and Equation 3):

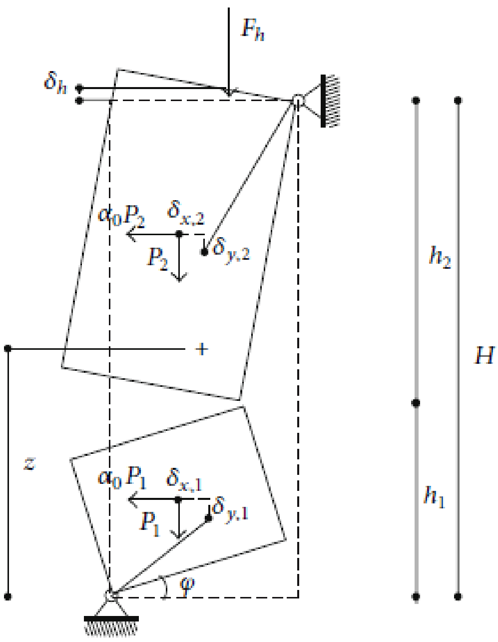


Figure 3. Proposed out-of-plane mechanism of a wall. (Castellazzi et al., 2013)

$$\alpha_0\left(\sum_{i=1}^{n} P_i\delta_{x,i} + \sum_{j=n+1}^{n+m} P_j\delta_{x,j}\right) - \sum_{i=1}^{n} P_i\delta_{y,i} - \sum_{h=1}^{k} F_h\delta_h - L_{fi} \qquad \text{Equation 3}$$

where: P_i is the self-weight of each macro-element part (n entries) composing the kinematic mechanism, P_j is the weight transmitted to the macro-element by adjacent structures(m entries), F_h is the generic external force applied to a macro-element part (k entries), $\delta_{x,i}$ and $\delta_{x,j}$ are the horizontal virtual displacements of each macro-element centroid , $\delta_{y,i}$ and δ_h are the vertical virtual displacements of each point of application of P_j and F_h respectively, L_{fi} is the work done by internal forces, and finally, φ is the given rotation to initiate the mechanism.

The seismic acceleration of the activation of the mechanism (a_0^*) is calculated from (NTC08, 2008):

$$a_0^* = \frac{\alpha_0 \sum_{i=1}^{n+m} P_i}{M^* \cdot FC} = \frac{\alpha_0 \, g}{e^* \cdot FC} \qquad \textit{Equation 4}$$

$$e^* = \frac{g \, M^*}{\sum_{i=1}^{n+m} P_i} \qquad \textit{Equation 5}$$

$$M^* = \frac{\left(\sum_{i=1}^{n+m} P_i \delta_{x,i}\right)^2}{g \cdot \sum_{i=1}^{n+m} P_i \, \delta_{x,i}^2} \qquad \textit{Equation 6}$$

where: g is the gravity acceleration, e^* is the fraction of the participation mass of the structure, M^* is the participating mass in the mechanism and FC is a confidence factor. FC takes a minimum value of 1 when extensive information is available about the structure's geometry, construction details and properties of materials, FC takes a maximums value of 1,35 when very limited information is available about the structure, and FC intermediate value is 1,2.

3.5 N2 method

3.5.1 General

The N2 method (Fajfar, 2002; Fajfar, 2000; Fajfar and Gaspersič, 1996; Fajfar and Fischinger, 1988) is a relatively simple method for the performance evaluation of structures. It combines the pushover analysis of a multi-degree of freedom model with the response spectrum analysis of an equivalent single-degree of freedom system. It is derived in the acceleration-displacement format which allows the visual interpretation of the procedure and of the relations between the basic quantities controlling the seismic behavior.

The results of this method are reasonably accurate when the structure oscillates predominantly according to the first mode. Some other limitations in the method exist and are corresponding to the limitations of the pushover analysis and the inelastic spectra (Fajfar, 2000).

This method is already considered in some codes for earthquake design like Eurocode 8 (CEN, 2004) and the Italian code (O.P.C.M 3274, 2004) and its further modifications (O.P.C.M 3431, 2005). It was successfully applied for seismic performance evaluation of many types of masonry structures (Carpentieri, 2011; Pelá et al., 2009; Aprile et al., 2006; Resemini, 2003; Lagomarsino et al., 2002; Elyamani et al., 2017).

3.5.2 The procedure

The procedure of the method is detailed in (Fajfar, 2002). Here, a brief of the main steps is presented. The procedure is graphically shown in Figure 4. In the first step, the elastic response spectrum in the format of time (T)-elastic acceleration (S_{ae}) (Figure 4, a) is transformed to the displacement (S_{de})-acceleration format (Figure 4, b) for the same viscous damping ratio using the following relation (Fajfar, 2002):

$$S_{de} = \frac{T^2}{4\pi} S_{ae} \qquad \text{Equation 7}$$

In the second step, the capacity curve obtained from the pushover analysis is transformed to the equivalent bi-linear curve using the approximate approach of equal area, i.e., the area under the capacity curve is equal to the area under the bi-linear curve (Figure 4, c). The ratio between the maximum displacement and the yield displacement is the ductility factor (μ).

In the third step, the obtained spectrum in the new format is transformed to the inelastic spectrum Figure 4, d) using as reported by Fajfar (2002) the following relations proposed by Vidic et al. (1994):

$$S_a = \frac{S_{ae}}{R_\mu} \qquad \text{Equation 8}$$

$$S_d = \mu \frac{T^2}{4\pi^2} S_a \qquad \text{Equation 9}$$

where, R_μ is the reduction factor due to ductility and is evaluated from (Fajfar, 2002):

$$R_\mu = (\mu - 1)\frac{T}{T_C} + 1 \qquad T < T_C \qquad \text{Equation 10}$$

$$R_\mu = \mu \qquad T \geq T_C \qquad \text{Equation 11}$$

where, T_C is the characteristic period of the ground motion. The last step in the method is the intersection between the bi-linear capacity curve and the inelastic response spectrum to determine the performance point which defines the performance acceleration and the performance displacement of the structure (*Figure* 4, d).

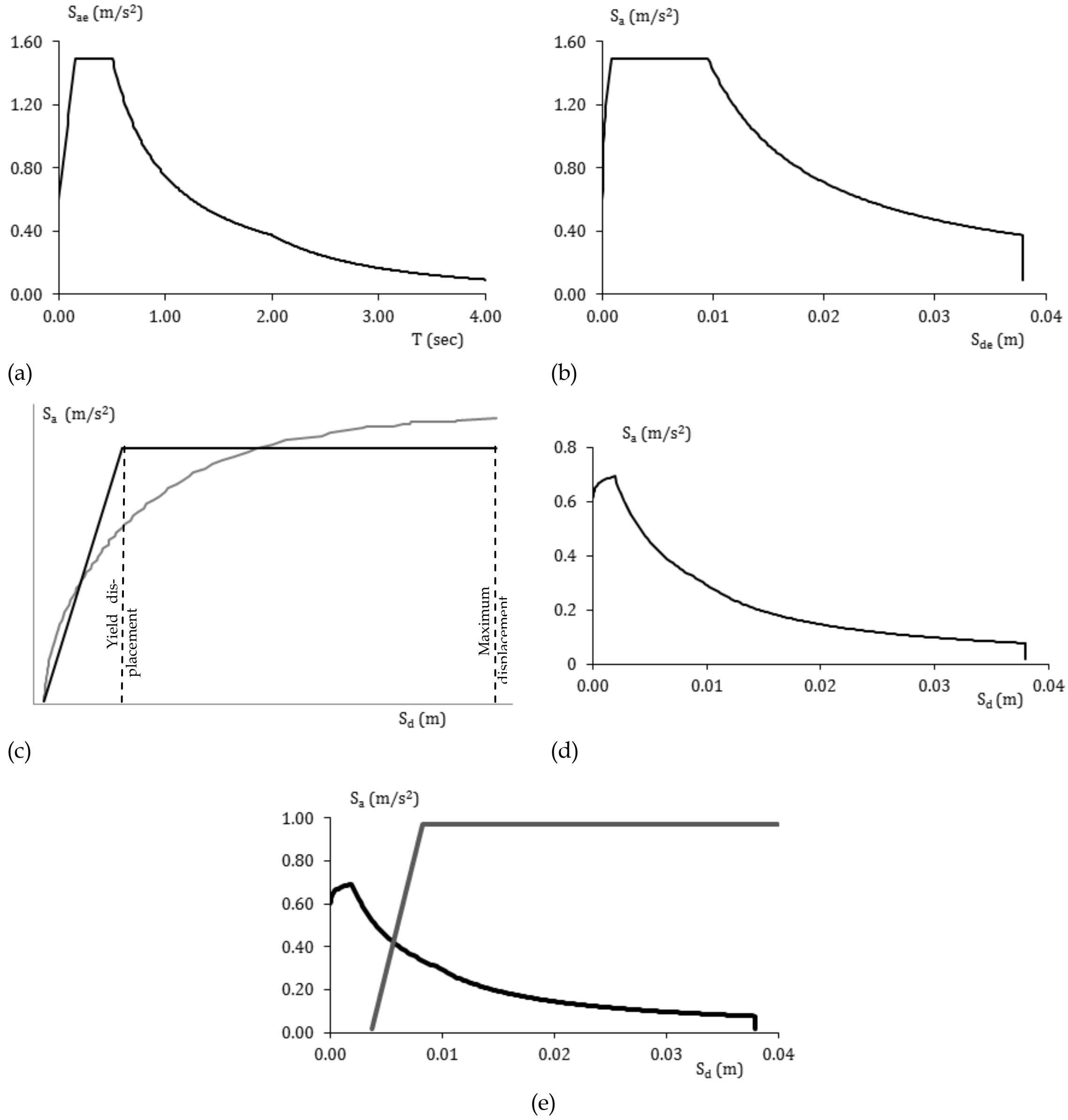
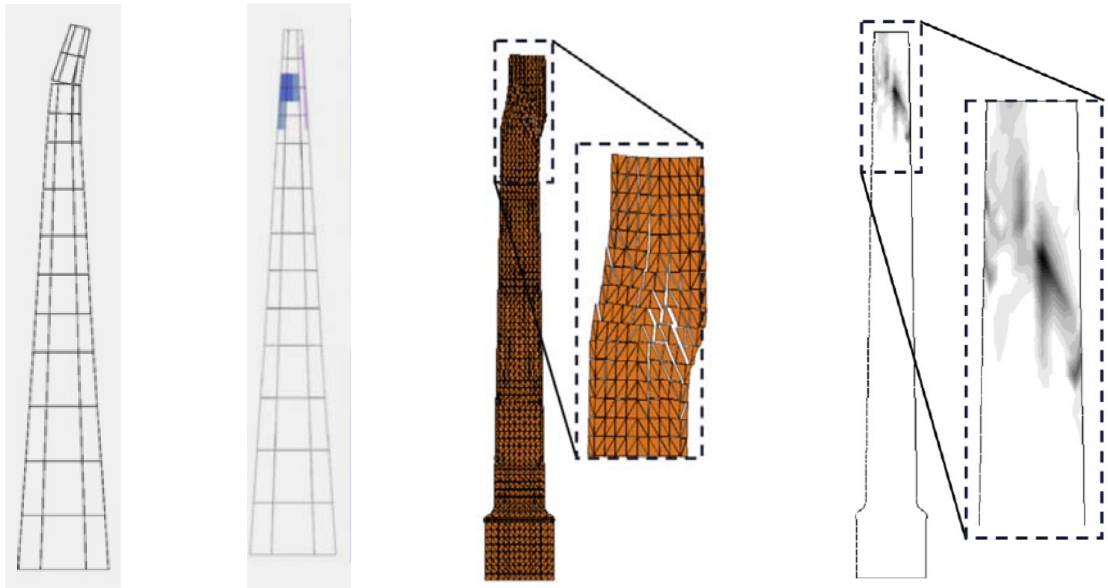


Figure 4. N2 method procedure: (a) elastic response spectrum; (b) elastic response spectrum in AD format; (c) the capacity curve (in grey) and the equivalent bi-linear curve (in black); (d) inelastic response spectrum; and (e) performance point (Elyamani, 2015).

3.6 Case studies

In the literature a large number of publications that addressed the issue of seismic assessment of historical structures can be found. The pushover analysis is widely used and some researchers confirm its results using the kinematic limit analysis; on the contrary, still the usage of the time history analysis is limited. Few case studies have been studied using both of the pushover and the time history analyses.

Peña et al. (2010) used the pushover analysis and the nonlinear dynamic analysis in the assessment of the seismic behavior of a historical minaret in India. The FE model was calibrated based on the results of the AVT (Ramos et al., 2006). The used models were (1) a 3D one that used beam elements to simulate the minaret (Beam model) (2) a 3D one that used solid and shell elements (Solid model) and (3) a 2D in-plane model (Rigid model) based on the Rigid Element Method. Rather similar behaviors were found for the three models. It was observed that the load factor (the base shear/the self-weight) was around 0,21 for the Beam and the Rigid models, while it was around 0,18 for the Solid model. For the collapse mechanism, the three models showed that the minaret materials did not fail by compressive stresses and the structure collapsed by overturning at the base. To study the effect of the used load pattern on the pushover results, two other load patterns were considered: the linear distribution of the displacement along the height and the forces proportional to the first mode shape. It was noticed that the resisted load factor depended very much on the distribution of the forces. The load factor proportional to the first mode was only 35% of the load factor proportional to the mass, while the load factor proportional to the linear distribution was 53%. Moreover, the collapse section changed and moved from the base in case of mass proportional load pattern to the first balcony for the two new load patterns. For the nonlinear dynamic analysis, five synthetic accelerograms compatible with the design spectrum of the Seismic Indian code were used. The Rayleigh damping model was used in which the experimentally identified damping of 2,5% was used and other two values of 5 and 8% were also tried. This analysis showed, on contrast to the pushover analysis, that the last two levels of the minaret were the most vulnerable, especially the last level which presented the highest drift, *Figure* 5-a. The difference in results was attributed to the great influence of the higher modes of vibration on the seismic response of the minaret.



(a) (b)

Figure 5. Nonlinear dynamic analysis of (a) minaret: deformed shape (left) and collapsed part in blue (right) (Peña et al., 2010); and (b) chimney: deformed shape (left) and collapsed part in dark gray (right) (Minghini et al., 2014).

Minghini et al. (2014) in their study on a brick masonry chimney damaged by May 2012 Emilia earthquake (Italy) found that the nonlinear dynamic analysis estimated the same collapse mechanism (the upper part of the chimney, *Figure* 5-b) that was observed in the damage survey carried out after the earthquake. The authors also found that the pushover analysis estimated the collapse of the lower part of the chimney which was not consistent with actual damage.

António et al. (2012) investigated the seismic response of two churches in Pico Island (Portugal) that were damaged by an earthquake in 1998. The authors carried out a detailed survey of earthquake damage, AVT and model updating. For the seismic safety analysis, they used a linear time history analysis and applied the already occurred earthquake. Good agreement was found between the numerical results and the actual surveyed damage.

Ramos and Lourenço (2005) assessed the seismic safety of a typical building typology in Lisbon using the pushover analysis. They studied the influence of the group of buildings in the seismic behavior of the individual buildings that constitute the block, what was called the block effect. It was concluded that this effect was beneficial and increased the safety against earthquakes. Thus, safety analysis of historical buildings belonging to larger compounds can be carried out with isolated buildings, which can reduce the effort and time to great extent. However, it must be stressed that the difference in the results were rather large and, if the isolated building analysis would indicate unsafe condition, it may be suitable and economically justifiable to refine the analysis using the full compound.

A common building typology of stone masonry residential buildings in Lisbon called Gaioleiro was studied extensively using the pushover analysis employing different load patterns, including the adaptive pushover, and the nonlinear dynamic analysis. Moreover a detailed sensitivity analysis was carried out and the influence of the compressive and tensile strengths, the compressive and tensile fracture energies, the damping ratio and the modulus of elasticity of walls and floors was discussed, see Mendes and Lourenço (2013) and Mendes (2012).

4. CONCLUSIONS

The paper has presented the recent state-of-the-art of some of the interconnected activities carried out for the purpose of seismic safety evaluation of historical structures. The following are the main conclusions concerning each of these activities.

On the model updating: creating a reliable FE model of a heritage building is often a difficult task because of the challenges usually involved by this type of buildings such as their complex geometry, and possible existing damage and previous repairs. Consequently, it is always required to validate the historical structure FE model against possible modeling inaccuracies and uncertainties. A common approach is to carry out dynamic identification tests on the structure and extract the experimental natural frequencies and mode shape. Then, by correlating them to their numerical counterparts (using for instance D_f and MAC), it may be possible to update the material properties, in specific the modulus of elasticity, and the boundary conditions could be updated.

There are several methods of model updating. These include the direct methods, the indirect methods, the manual method, the automated method and the stochastic method. For historical structures, the manual method is the most widely used. The automated model updating has been used in few cases, and the indirect methods and stochastic methods have been used in very few cases.

From the reviewed case studies, it can be stated that in case of bridges (and similar structures) and towers (and similar structures), very good correlation in terms of D_f and MAC can be found between experimental and numerical modal parameters. For other types of historical structures, like churches, it is difficult to find a good correlation between numerical and experimental mode shapes.

On the seismic assessment of historical structures: assessing the seismic capacity of historic structures via numerical models is a difficult task and the usage of other simplified methods like the kinematic limit analysis is advisable to cross check the results.

The pushover analysis is nowadays well recognized and adopted in many modern seismic codes. It has no rigorous theoretical base. It has some significant limitations, for instance, in case that the higher modes of vibration become important, the nonlinear dynamic response may differ from the predictions of the pushover analysis.

The nonlinear dynamic analysis is generally preferred to the pushover analysis. However, it has several challenges. Among them are the dependency of the results on the used earthquakes records, the complexity of time-integration algorithms, the difficulties in damping representation, and the large needed of computational and storage resources.

The N2 method is a relatively simple method for the performance evaluation of structures. It is now considered in some modern codes of seismic assessment of structures. It has two differences from the capacity spectrum method. First, the inelastic spectra rather than elastic spectra are used. Second, the demand quantities can be obtained without the need for iterations.

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