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SEISMIC RISK OF HISTORIC STRUCTURES AND MONUMENTS: A NEED FOR A UNIFIED POLICY.

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Abstract. *There is lack for a common approach on the assessment of seismic risk for historic structures and monuments. Performance of interventions for a partial mitigation of seismic risk is yet a challenge. In most cases limitations stemming from the implementation of interventions on historic structures and monuments that are contrary to internationally accepted guidelines, e.g., Carta di Venezia, do not allow fulfilling the safety level of new constructions.*

The challenge of balancing safety versus maintenance of the architectural and artistic features of historic structures remains a pressing issue to address. In addition, new developments on the seismic hazard and proper interventions related to near-source phenomena are scarcely considered in the literature.

Through representative examples, this work attempts to provide a framework that: (i) quantifies the "safe" duration (nominal life) of an intervention on a monument; (ii) considers the effects of near-source phenomena and; (iii) elaborates on the integration of experimental/analytical methods in view of recent developments in the field.

1 SEISMIC REHABILITATION OBJECTIVES AND PERFORMANCE LEVELS

A seismic *Rehabilitation Objective* consists of one or more rehabilitation goals. Each goal consists of the verification of a target *Building Performance Level* with an associated *Earthquake Hazard Level*. By considering not only the structural damage but also the response of non structural elements and artistic assets, performance levels may be defined in relation to different performance targets of the construction, associated to the functionality and the cultural properties of buildings.

The modern seismic codes for the design of new buildings, as well as pertinent recommendations for the evaluation and rehabilitation of existing ones, are based on the performance based assessment (PBA), that is the fulfillment of some target performance levels (limit states) in correspondence to predefined seismic actions [1,2,3,4].

The EC8-3 specifies as T_L=50 years the duration of “nominal life” of a structure and defines three building performance levels (Limit States) considered as appropriate for the seismic protection of ordinary new buildings: (a) Near Collapse (NC); b) Significant Damage (SD); and (c) Damage Limitation (DL). Each one of the limit states is associated to a seismic action defined by the return period (T_{RL}) and a probability of exceedance (p) for T_L=50 years. Specifically for the (a) NC, the T_{RL} is 2475 years and p= 2%; (b) SD, the T_{RL} is 475 years and p=10% and (c) DL, the T_{RL} is 225 years and p=20%.

The Italian code for the monuments (DR14/01/2008)-[5] specifies five limit states: the SLC, SLV, SLD, SLO and SLA. The first three correspond to the (NC), (SD) and the (DL) of the EC8-3, respectively, and the fourth one specifies “almost” no damage conditions. The SLA defines the limit state for structural members/parts and even contents of artistic value. For the first four limit states the probabilities of exceedance are 5%, 10%, 63%, 81%, respectively, which are related to the *reference duration* (V_R). The V_R can be calculated from the *nominal life* of the structure (V_N), which for an ordinary structure is considered to be V_N=50 years, and the importance factor (C_U), that is ($V_{R=}$ V_N x C_U).The code specifies four important classes, assigning to each one a value for the importance factor (C_U).

Currently in Greece there is not yet a code for the seismic protection of monuments. Worth noting are two recent attempts that set the basis to address the pressing issues on the seismic protection of masonry structures and monuments [6,7]. From Ref [7] on monuments, presenting many similarities with the Italian code, it is worth mentioning the proposal to select the design seismic actions as well as the acceptable damage levels on the basis of the importance of a monument, proposing three levels (l_i i=1,,2,3): monuments of universal importance (I_1), monuments of national importance (I_2), and monuments of local interest(I_3).

Also, depending on the exposure to visitors, it proposes that the monuments are classified into three categories (C_i i=1,2,3) assigning a proper $C_{u,}$ that is: C_1: almost continuous presence of public or frequent presence of large groups: (i) inhabited buildings in historical city centers, (ii) monuments used as museums, and (iii) monuments continuously used for worshipping; C_2: occasional habitation or intermittent presence of small groups: (i) monuments visited only under specific conditions, and (ii) remote and rarely visited monuments; C_3: entrance allowed only to service-personnel with visitors standing only outside the monument.

Thus, the earthquake protection of cultural heritage assets can be realized through a preventive knowledge of the seismic risk, in order to plan mitigation strategies and schedule the necessary strengthening interventions for the reduction of vulnerability. Seismic risk is the outcome of three different factors: the seismic hazard (probability of occurrence of an earthquake of a given intensity at a certain site); the vulnerability (predisposition of the building to be damaged by an earthquake); the exposure (related to the conditions of use and presence of the public, but also to the value of the building and the artistic assets that contains). Clearly, the earthquake protection of cultural heritage assets is a task that involves not only the safety and economic impact, but also the conservation of the cultural properties of the assets.

1.1 Seismic Hazard

Seismic hazard requires the selection of the proper ground excitation. It is recommended that the site ground excitation is determined by taking into consideration the following: 1. The response of the monument to past earthquakes, so that, in case of a damaged structure from an earthquake, the action that caused the damages may be the first choice, 2. The soil stratification and the foundation and 3. Topographic amplification effects. For important monuments, the seismic action may be modified by taking into account local soil dynamic conditions, geomorphology, an estimation of the duration of the earthquake and especially the effects of neighboring active faults that could create near-fault phenomena.

For the last parameter it should be stated that, even though there is considerable literature on reinforced concrete and steel structures considering near-fault phenomena, the study of masonry structures and monuments under near-fault seismic excitations is scarce compared to their significance in seismic design [8,9]**.** For this reason a relatively extensive introduction on this issue is presented, followed by an example that demonstrates the significance of near-fault phenomena associated with the presence of an active fault in the vicinity of a monument.

1.2 Near-Fault Strong Ground Motion Characteristics

The increased density of recording stations in the near fault areas has permitted the collection of near-fault ground motion recordings that present characteristics quite different from those of the usual far-field ground motions. These near-fault characteristics are mainly present in the form of large pulses in the ground velocity time history records at sites towards which the fault ruptures, a phenomenon called directivity. The directivity characteristics and effects are briefly presented and explained in the following.

Rupture generally progresses across a fault as a series of individual cracks or sub-events. Each crack produces a dislocation or slip which has a duration called the rise time and a slip velocity or slip rate amounting to 50-150 cm/sec. Each crack creates a velocity pulse with duration equal to the rise time and amplitude equal to the slip rate. These pulses travel along the fault with the velocity of shear waves. At the same time the rupture spreads towards a certain direction with a rupture velocity similar to that of the shear waves. In the rupture direction, rupture cracks and crack velocity pulses travel with the same velocity. Accordingly, in the rupture direction, a phenomenon similar to the Doppler phenomenon takes place. The generated pulses overlap and the waves arrive at a site in the direction of rupture as a large pulse of motion creating a shock wave effect that occurs at the beginning of the record. The phenomenon is called *forward directivity* and the pulse of the ground motion is typically characterized by large amplitude and short duration. At a site located near the epicenter, where rupture propagates away from the site, the arrival of the pulses is distributed in time. This condition, referred to as *backward directivity*, is characterized by motions with relatively long duration and low amplitude.

Research has shown that simplified representations of the velocity pulse can capture the salient characteristics of the response of structures to near-fault ground motions. The simplified pulse representations of velocity time histories are defined by the number of equivalent half cycles, the period of each half cycle and the corresponding amplitudes [11]. The amplitude of the velocity pulse is significantly affected by magnitude, distance and site conditions. The number of half cycles in a velocity pulse affects the spectral amplification, that is, as the number of cycles increases the spectral amplification increases as well [12, 13, 14, 15].

Rupture directivity effects can be present both for strike-slip and dip-slip events. Because of its polarization, the pulse of motion is predominant in the orientation perpendicular to the fault plane. The same phenomenon is present for a dip-slip fault, where forward directivity conditions occur for sites located near the up-dip projection of the fault plane. In this case, also, the pulse of motion is oriented perpendicularly to the dip of the fault plane. Thus, fault parallel spectra are in most cases comparable with design code spectra. Procedures that incorporate amplification of the standard response spectrum to account for near-fault phenomena are available in the literature and are recently incorporated in the Next Generation of Ground Motion Attenuation (NGA) Project [16, 17].

2 INTERVENTION STRATEGIES

Historical buildings may be considered to belong to importance class III or IV leading to large seismic requirements and seismic actions that are characterized by a high return period [4,5]. Thus, their preservation could most likely require invasive interventions in order to meet the “safety standards” for new construction. According to the principles of interventions on historic buildings and monuments less intrusive interventions are imposed, which, nevertheless, would continue to ensure to a certain degree the safety of the monument, e.g.,[18, 19]. In general, interventions on monuments should satisfy the following three principles: (i) Reversibility, (ii) Durability (in-time), and (iii) Feasibility of the proposed solution.

The basic philosophy of codes and comprehensive research efforts, e.g. [5,41], promotes three intervention alternatives: (i) Basic Safety, with a Rehabilitation Objective that practically achieves the dual rehabilitation goals of Near Collapse (NC) and Significant Damage (SD) of [4]; (ii) Limited Rehabilitation, which attempts to improve the seismic safety of the whole structure by means of non intrusive, yet, extensive interventions that are inferior to the Basic Safety objectives and (iii) Local Rehabilitation, which improves the response of the structure through local interventions not affecting the overall behavior of the structure. The last two alternatives appear to be the most appropriate and in fact the most widely used (in many cases “silently adopted”) for the preservation of historic structures and monuments.

2.1 SD Performance through Limited Duration Rehabilitation Measures

This section proposes an alternative approach that handles the Limited Rehabilitation Objective (LRO) in a totally different way. The basic idea is to consider the rehabilitation measures as fulfilling a predefined limit state for a certain time duration, after which the structure with its interventions should be re-evaluated and, if necessary, appropriate measures will be taken at this later time.

The methodology is demonstrated through the following example, applied in order to achieve the SD performance level, as specified in EC8-3, through Limited Rehabilitation measures valid for a certain period of time.

The methodology also requires the introduction of the term "nominal life of an intervention T_Δ", defined as the time for which the intervention ensures a selected performance level, e.g., SD or DL for the probability of exceedance being P_R = 10% or 20%, respectively [4].

The methodology requires the use of either an attenuation relationship or side specific spectra. In this example the following attenuation relations are selected, as forming the basis for the calculation of the a_{gR} of the current EC8-1 design spectra for Greece. As Greece is divided in three seismic hazard zones, Z_i, (i=1,2,3), each equation specifies the corresponding a_{gR} (design ground acceleration) in terms of the return period T_{RL} [20]:

$$\text{Zone Z1: } \log a_{gR} \approx 0.277 \log T_{RL} + 1.579 \qquad (1)$$

$$\text{Zone Z2: } \log a_{gR} \approx 0.264 \log T_{RL} + 1.739 \tag{2}$$

$$\text{Zone Z3: } \log a_{gR} \approx 0.240 \log T_{RL} + 2.015 \tag{3}$$

Adapting a Poissonian distribution for the seismic events, that is the seismic events are statistically independent from each other, the T_Δ is related to the return period T_{RL} and to the probability of occurrence as given by

$$T_{RL} = -\frac{T_\Delta}{\ln(1-P_R)} \approx \frac{T_\Delta}{P_R} \tag{4}$$

It should be noted that the a_{gR} resulting from equations (1) – (3) is decreased by 20% prior to being used in the code spectra [20].

Also, if the seismic action is defined in terms of the reference peak ground acceleration a_{gR}, the value of the importance factor γ_I multiplying the reference seismic action to achieve the same probability of exceedance in T_Δ years as in the $T_{\Delta R}$ years for which the reference seismic action is defined, may be computed from

$$\gamma_I \approx \left(T_{\Delta R} \,/\, T_\Delta\right)^{-1/k} \tag{5}$$

where k is in the order of 3 (EC8-1).

The example considers a historic building located in the seismic zone Z_1 founded on a soil class A.

By implementing the maximum of the interventions that comply with the archeological /architectural restrictions to the monument [18, 19] and after performing the analysis, evaluation of the results indicate that the structure reaches the SD performance level for a_{gRL} = 0.155g.

In order to use equation (1), the a_{gRL} should be divided by 0.8 leading to a_{gRL} = 0.194g. Substituting a_{gRL} in eq. (1) leads to

$$\log 194 \approx 0.277 \log T_{RL} + 1.579 \tag{6}$$

providing the return period T_{RL} = 360.5 years. Consequently, substituting in eq. (4) the T_{RL} = 360.5 years for P_R = 0.1 yields T_Δ = 38 years.

At this point one could consider two possibilities regarding the importance class of the historic structure, that is class III or IV corresponding to γ_I = 1.2 or 1.4, respectively.
Equation (5) results in: $T_{\Delta R}$ = 22 years for γ_I = 1.2 and $T_{\Delta R}$ = 14 years for γ_I = 1.4.

Thus, the implemented interventions can be considered as fulfilling the SD state for the duration of either 22 or 14 years depending on the importance classification of the structure. If the classification of the structure can be considered that it is exclusively related to the "exposure to visitors", then the nominal duration of the interventions could be further extended by setting γ_I = 1.0 leading to a $T_{\Delta R}$ = 38 years.

The Damage Limitation Level can be treated in exactly the same way considering as the probability of exceedance in 50 years to be 20%. In this case for T_{RL} = 360.5 years, use of equations (1), (4) and (5) results in: $T_{\Delta R}$ = 46.5 years for γ_I = 1.2, $T_{\Delta R}$ = 29 years for γ_I = 1.4, and T_Δ = 80 years for γ_I = 1.0.

Therefore, the implemented interventions can be considered as fulfilling the DL state for the duration of 80, 46.5 or 29 years depending on the importance classification of the structure.

It can be concluded that for either the SD or the DL limit states, additional and more invasive interventions can be postponed in time. At the end of the nominal duration of the interventions, a new evaluation of the structure should be performed. At that later time any necessary measures might be implemented making use of a more accurate knowledge of the seismic hazard, the ability to assess more reliably the vulnerability of the monument and the availability of less invasive intervention techniques.

3 METHODS OF ANALYSIS

This section attempts to clarify several important issues related to limitations of [3, 4] specified solution methods as well as the selection of the most appropriate solution method(s) for historic masonry structures and monuments.

In section 4.4.3, EN 1998-3 reports that the seismic action effects may be evaluated using one of the following methods: a) lateral force analysis (static linear), b) modal response spectrum analysis (linear), c) q- factor approach, d) non-linear static (pushover) analysis and, e) non-linear time history dynamic analysis.

Options (a) and (b), are applied with the un-reduced elastic spectrum to a structure that behaves elastically. In addition, according to Annex C of [4] both methods assume conditions of rigid diaphragm and a clear distinction between ductile and failure mechanisms, both requirements are not met in a great majority of masonry structures.
Also, classifying such mechanisms as strictly “brittle” without whatsoever nonlinear deformation capacity would lead to a significant underestimation of the seismic capacity of a masonry structure, and, therefore, to unusable results. It is well documented that a moderate “ductility” has to be recognized also for shear failures. Thus, one may conclude that for masonry structures, both options are impractical, e.g., [28, 29].

According to EC8-3 (4.4.1 (5)) and the informative Annex C, the q-factor approach is practically applicable to masonry structures with rigid diaphragm behavior. The approach proposes either a reduced response spectrum or its equivalent alternative representations, given in [3], that use properly developed time histories of ground motion. Of the two alternatives of linear elastic analysis, clearly the option of time history analysis of modal response and real time superposition of modal contributions is the only accurate option. Despite this, most commercial software for the analysis of masonry structures offer only the possibility of combining the Modal Maxima either through the Complete Quadratic Combination (CQC) or the Square - Root - of the - Sum- of the - Squares (SRSS) option. Both approaches combine maximum responses of different modal contributions which, in time history records, are not concurrent. The danger from these approaches is that they may lead to overly conservative estimates, in light of the fact that several modes need be accounted for before 70% of the total masonry structural mass may be engaged in elastic finite element (F.E.) modal analysis, since, in the absence of diaphragms numerous subordinate modes could be excited in such F.E. models. These modes are occasionally spurious and are not related to the actual behavior of the structure. Overly conservative estimates of response emanating from SRSS may lead to excessively invasive (nonreversible) interventions in masonry structures, and as such, they should be always treated with particular caution.

The EC8-3 specifies the use of the last two alternatives (d) and (e) when the conditions of its section C.3.2 are not met, simply stated: for “irregular” structures with or without rigid diaphragms. First we elaborate on the method (d), i.e., the pushover analysis. We distinguish the combination of two cases: “regularity”, as defined in 4.4.2(1)P and in C.3.2, as well as presence of rigid diaphragm behavior. In the case of rigid diaphragms and regular structures, pushover can be used with two fixed force pattern distributions: modal (mass-proportional) with a force distribution proportional to the first modal shape of the structure; being able to represent the structural dynamic amplification which increases the action on higher stories and, a constant distribution that is consistent with a soft ground storey response. These two distributions may be assumed as bounds for seismic analysis of regular buildings: the actual result, resulting from dynamic analysis can be usually assumed to be within these two solutions. For irregular structures with rigid diaphragms, an adaptive pushover analysis is considered a better approach. In that case the analysis is progressively updated to account for the structural response evolution in terms of stiffness degradation.

The presence of in-plane flexible diaphragms, such as typical wooden floors or thin masonry vaults, is very common in historic masonry structures. Even though connections between walls and floors could prevent to a certain extent local first mode mechanisms, the presence of flexible floors results in a quite complex global seismic response, demonstrating little coupling between the vertical walls that tend to behave independently.

In the presence of plane deformable diaphragms use of pushover presents several issues and difficulties which have not yet been taken into consideration in codes, the choice of the control node being a characteristic example. An acceptable approach in practice could be to analyze separately the in-plane seismic response of each masonry wall, as extracted from the global structure with its supporting loads and inertial masses [21].

For mixed structural systems that are commonly encountered in strengthened existing structures with either steel or concrete members the [22] favors the use of pushover analysis.

Certainly, the most general but complicated option is the option (e), in which case the solution algorithm marches in time satisfying at each time step the conditions of dynamic equilibrium. Clearly, nonlinear F.E. solutions are still hampered today by the convergence and numerical stability problems attributed to cracking in masonry, combined with the absence of the stabilizing effect of steel reinforcement that is available in concrete structures.

Alternative methods that reproduce the nonlinear characteristics of the response, while taking advantage of the stability of the elastic approaches have been developed. An effective option is re-meshing of the structure, while allowing the creation of new boundaries of discontinuity in the structure along the paths of crack formation. By the incremental propagation of a crack, each time a new boundary is created, followed by remeshing and restructuring of the elastic stiffness matrix of the structure, which however, becomes more compliant as the connectivity between adjacent nodes is lost caused by the opening of a crack.

In conclusion nonlinear static analysis and the q-factor approach seem the real options for designers at the present.

4 EVALUATION OF SEISMIC RISK CONSIDERING NEAR FAULT EFFECTS

The church of St. Demetrios, located on the island of Lemnos in the North Aegean sea, has been selected as a case study of a historic structure constructed without seismic regulations in a highly seismically active region. It is a post-Byzantine triple-domed basilica with a timber roof constructed in 1892 [23]. The plan view is nearly quadrangle with three polygonal apses on the eastern façade. The length and width of the church are 25 and 21 meters, respectively. The central aisle has a vaulted ceiling, where a 5.6 m high dome and a Π-shaped outer narthex are incorporated. The height, up to the base of the dome, is 12m. The masonry construction is mixed including rubble masonry with carved corner-stones and is coated with plaster at the exterior facade of the church, while inside it is partly covered with coating and partly with frescoes.

The evaluation of seismic risk involves a thorough condition investigation of the monument as well as the selection of the seismic hazard followed by several analysis in order to specify the type and the extent of the interventions. All steps of the process are briefly presented in the following.

4.1 Condition assessment

Visual inspection as well as a series of in-situ and laboratory tests has been conducted for condition assessment of the church. Main objectives of the testing program were the collection of information regarding the mechanical and chemical properties of the materials as well as the construction details [25, 26, 31].

Geotechnical investigation and soil amplification: At first a geotechnical study was performed in order to investigate the stratigraphy of the soil and the level of groundwater table, followed by a soil dynamic amplification study. The geotechnical study provided the soil parameters needed to estimate the bearing capacity of the soil, the expected subsidence, the modulus of subgrade reaction and the soil category according to [24]. The study of the dynamic soil response aimed at calculating a design spectrum that accounted for the amplification of seismic excitation through the local soil strata.

Three accelerograms from representative Greek seismic events were selected in order to evaluate soil amplification: (a) the 1986 Kalamata earthquake with magnitude M=6; (b) the 1981 Korinthos (Alkyonides) aftershock earthquake at the Corinthian gulf with magnitude M=6.4; and the Athens 1999 earthquake with magnitude M=5.9 recorded at the Sepolia station. All three accelerograms were scaled in order to be compatible with the seismic zone according to [3]. The scaled accelerograms excited the base of the soil column and the amplified spectra were obtained at the soil surface, as depicted in Figure 1.

Visual Inspection: A preliminary assessment of the condition was performed with visual inspection and surveying in order to record damage, i.e., cracking, peeling or other imperfections of the structure. Noticeable cracking, observed almost everywhere on the internal and external facades of the church, was recorded in detail and classified according to its type and size. In Figure 2 the location and the shape of the crack pattern is depicted in all views of the structure. it should also be noted that significant cracking was also recorded at the vaults and the pillars.

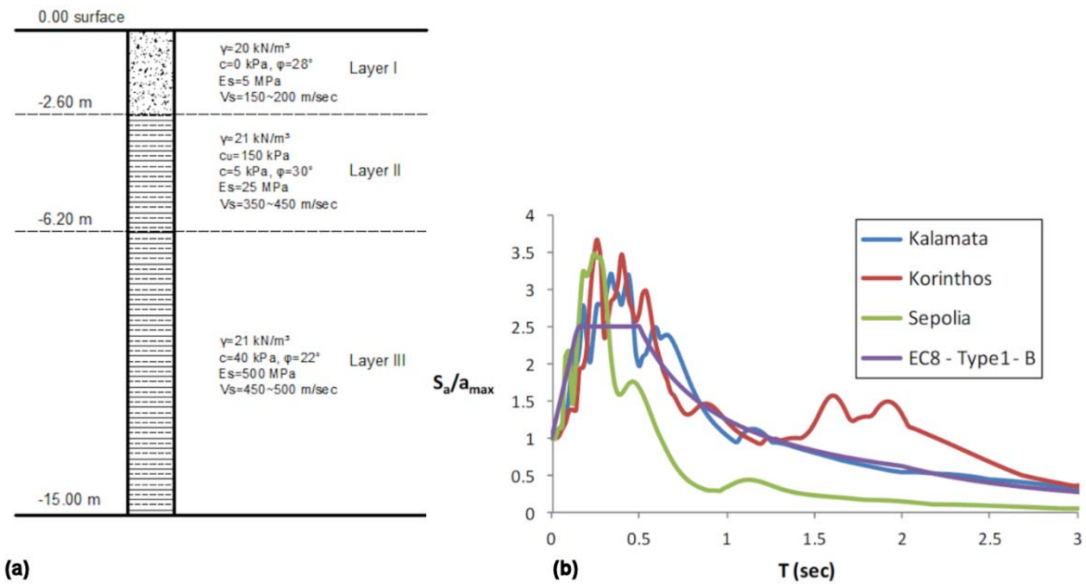


Figure 1: Geotechnical investigation: (a); soil profile; (b) amplified spectra at soil surface.

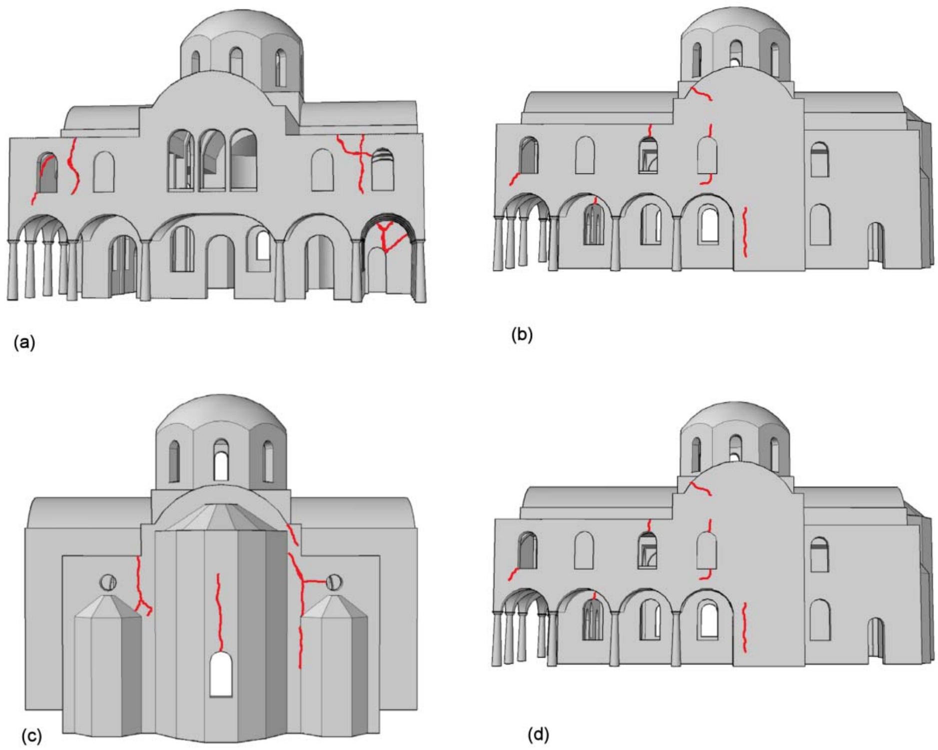


Figure 2: Major cracks in the masonry structure: (a) frontage (west façade); (b) south façade; (c) east façade; (d) north façade.

Measurement of Mechanical and Chemical Properties of Masonry: In order to measure the compressive strength of the stones and the bricks, both laboratory and in-situ tests were performed. Application of rebound hammer tests at selected positions, in conjunction with the laboratory tests provided the compressive strength of the stones (lime stones): f_{bc}=45.8 MPa.

Mortar samples were taken from various positions and their composition was examined in the laboratory. The laboratory work focused on the following issues: (i) nature of mortar (e.g., hydraulic, mixed, etc.), (ii) composition of binder, (iii) type and gradation of the aggregates, and (iv) ratio of raw materials. The following techniques were applied for the characterization of the mortar: (i) optical microscopy, (ii) study of the gradation of the aggregates, and (iii) x-ray diffractometry (xrd).

Infrared thermography: Application of the method revealed areas of humidity in the walls and confirmed testimonies and historic data about the absence of either partial or full closure of openings [27].

Endoscopy: Endoscopy was applied to a total of five positions of the walls of the church. In addition a number of endoscopies were applied at selected positions in order to examine the construction system of the central pillars and the anchorage length of the tie-rods in the masonry walls. It was found that (i) the bearing walls were composed by three-leaf masonry, (ii) there is no uniformity in the layout, since stones of different sizes have been used at several locations and (iii) the tie rods were improperly connected to the walls

Ambient Vibration Measurements: The modes of vibration and the natural frequencies of the church were measured applying ambient vibration testing. In total eight measurements were performed with direction either western-eastern or north-south. The positions included the center of the church beneath the dome at the ground floor and at the roof of the mezzanine as shown in Figure 3 appropriate software was used to calculate the transfer functions and the Fourier spectra at different positions.

(a) (b)

Figure 3: Ambient Vibration testing at indicative positions: (a) ground floor beneath the central dome; (b) roof of the mezzanine.

4.2 Types of Analysis

The mechanical characteristics of the masonry, based on the testing procedures, were applied in subsequent analysis. Two types of analysis were performed: (i) Global analysis considering the structure as a continuum where all the inter-connected individual parts act and deform together during excitations and (ii) local analysis studying parts of the structure that may deform almost independently from the whole bearing body. These parts, also referred as macroelements, were determined with acceptable reliability based on a survey of the structure. At this point it should be emphasized that structures with no diaphragms are mostly vulnerable to local failure mechanisms. The selection and evaluation of different possible collapse mechanisms is an issue of critical importance in order to assess the behavior of a masonry structure [28, 29, 30].

The macroelement approach may be usually followed by either a kinematic limit analysis or a non linear static analysis. The first alternative, which has been selected in this article, involves the proper selection of a collapse mechanism, while the second one uses a finite element model as well as a “blocks and joints” analysis [31, 32, 33].

4.3 Seismic Loads

Greece is the most seismically active region in Europe and among the most seismically active regions on a global scale [34]. The island of Lemnos is located in a seismic zone II that is characterized by a reference peak ground acceleration a_{gR}=0.24 g according to the seismic zone map of Greece [35]. The North Aegean is a well known area for its high seismicity with strong earthquakes reported even from ancient historical era up to as recently as 2014 [36]: M=7.0 in 197 BC; M=7.0 in 1471; M=7.0 on May 14, 1887; and M=6.8 on August 6, 1983. Most recently on May 24, 2014, a M=6.3 earthquake occurred with an epicenter between the islands of Lemnos and Samothraki, as the result of strike-slip faulting at shallow depths beneath the northern Aegean Sea. Faults within the North Aegean trough represent the northern branch of the North Anatolian fault system, the major transform faulting structure in northern Turkey (USGS webpage). In the analysis the seismic loads were determined according to: (i) the current version of [3, 4]; (ii) the first Greek seismic code adopted in 1959 [37]; (iii) spectra accounting for near-fault effects.

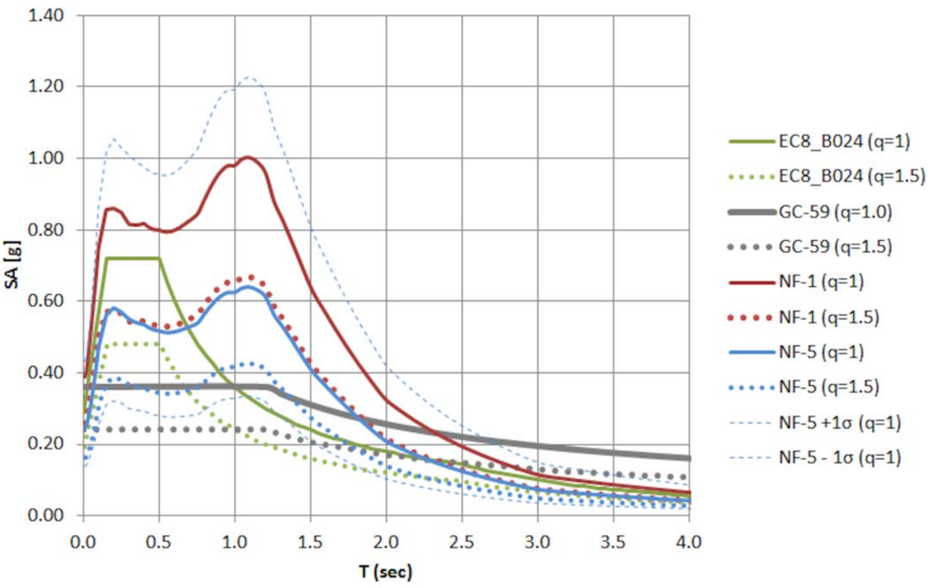


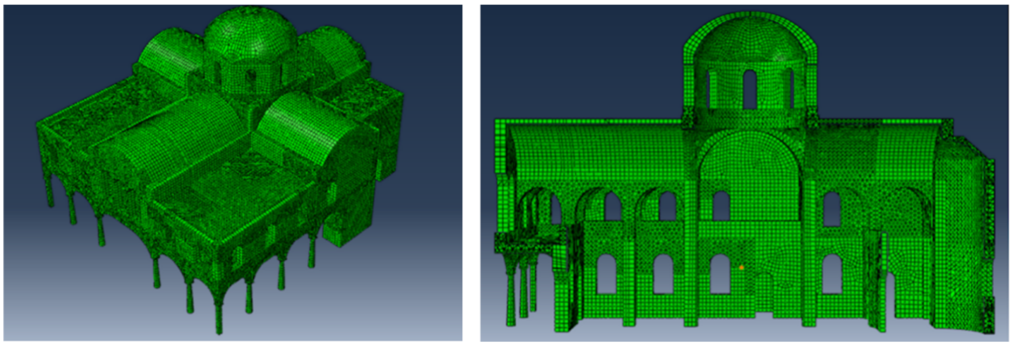
Figure 4: Comparison of different spectra applied in the analyses.

In Figure 4 the different spectra applied in the analyses are shown. The [4] spectra were considered with the following characteristics: reference ground acceleration for seismic zone II; a_{gR}=0.24 g; importance factor γ_I=1.2, characteristic periods T_B=0.15 sec, T_C=0.50 sec and T_D=2.00 sec and soil factor S=1.2. The near-fault spectra determined by applying the procedure proposed in [16] are representative of an M=6.5 seismic event at distance from the surface projection of the fault R=1 km and R=5 km, denoted as NF-1 and NF-5, respectively. The R denotes the minimum distance from the site to the projection of the fault to the surface. For clarification, it is mentioned that R=1 km or R=5 km indicate epicenter distances that could be in the range of 15 to 25 km from the epicenter. The elastic spectra are depicted with a continuous line, dotted lines correspond to inelastic spectra and dashed lines correspond to mean spectral acceleration (±) one standard deviation for the NF-5 scenario. A behavior factor q=1.5 was considered for all cases indicating a structure with small ductility [38]. The results from spectral analysis for the NF-5 scenario are not presented in detail, since as shown in Figure 4, they correspond to spectral values lower than the spectrum [4] for the range of periods considered; thus, it could provoke less damages than the spectra [4].
However, this overall conclusion is valid in average sense only, since consideration of the mean spectrum plus one standard deviation (NF-5 + 1σ) might be more damaging even from the NF-1 scenario, as also depicted in Figure 4.

4.4 Global Analyses and Results

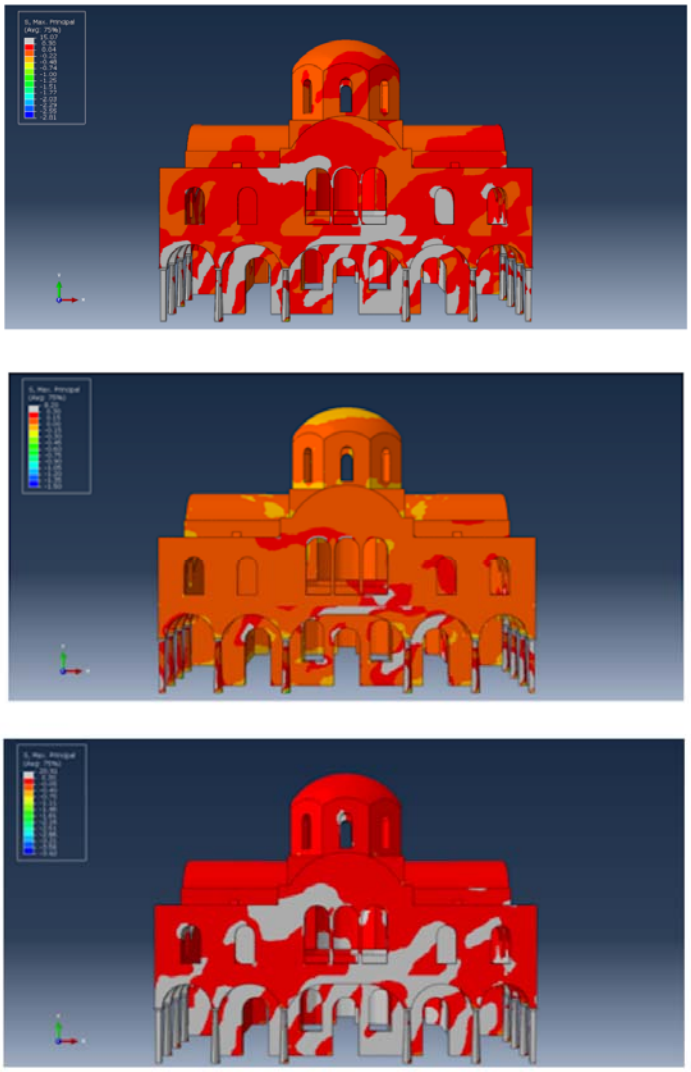
The finite element model, shown in Figure 5, consists of 317929 six and eight-node solid elements. Prior to the response spectrum analysis, an eigenvalue analysis of the model was performed and the results were validated based on the results of the ambient vibration testing [31].

The distribution of maximum stresses, for all the different spectra, is depicted in Figure 6 for the representative west façade. It should be noted that grey colors in the contour diagrams indicate exceedance of the tensile strength. The grey color denotes failures that occurred nearly at all facades and for all cases prior to the interventions on the structure.



(a) (b)

Figure 5: General views of the finite element model: (a) three dimensional view; (b) sectional view along the west-east direction.



(a)

(b)

(c)

Figure 6: West façade - Distribution of maximum stresses in the original model without interventions for the G+0.5Q+Ex+0.3Ey combination: (a) Eurocode 8 spectrum; (b) 1959-Greek Seismic Code spectrum; (c) Near-fault spectrum with R=1 km.

As depicted in Figure 4 the maximum demand is imposed from the near-fault spectrum at R=1 km (NS-1), while the second most damaging loading should be expected for the other spectrum in the near-fault area at R=5 km (NS-5). Indeed the maximum stresses for seismic combinations are noticed for the NS-1 spectrum, as shown in Figure 6, while the NS-5 seismic scenario also results in extended damage as also shown in Figure 6. The capacity checks included: in plane flexural resistance, shear resistance and out of plane flexural resistance parallel to the joints.

4.5 Local Analyses and Results

The determination of possible out-of-plane collapse mechanisms was obtained through a kinematic limit analysis [39]. Given the pattern of the cracks existing on the structure, two collapse mechanisms were examined, that is, the overturning of a part of the left façade and the overturning of the central apse, as shown in Figure 7.

Table 1 shows the results of the kinematic limit analysis performed for the two macroelements as well as the results of the verification with respect to the ultimate limit state according to Eurocode 8 and the NF-1 and NF-5 near-fault spectra. The verification was performed with the spectra shown in Figure 4 considering an importance factor γ_I=1.2. The verifications performed for the collapse mechanisms of the left façade and the central apse yielded different results depending on the spectra. The red color in Table 1 denotes collapse, while the green color denotes fulfillment of the ultimate limit state [3, 4].

4.6 Intervention Scheme and Performance of the Strengthened Structure

The assessment of the current condition of the structure revealed significant problems that may affect the structural behavior including extensive cracking of the masonry walls at all façades.

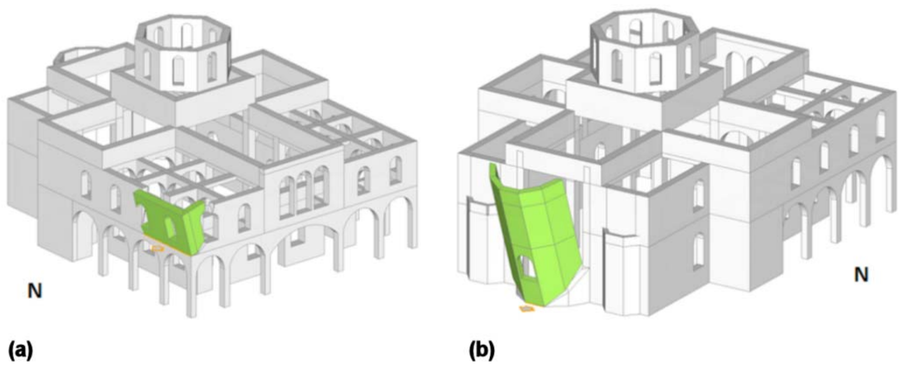


Figure 7: Selection of different local collapse mechanisms: (a) overturning of a structural part in the north façade; (b) overturning of the central apse in the east façade.

Collapse mechanism	a_0	a_0^* (g)	EC8		NF-1		NF-5	
			α^*_{Rig} (g)	α^*_{Def} (g)	α^*_{Rig} (g)	α^*_{Def} (g)	α^*_{Rig} (g)	α^*_{Def} (g)
North façade (mezzanine wall)	0.243	0.185	0.173	0.203	0.233	0.242	0.147	0.163
Central apse	0.162	0.142	0.173	-	0.233	-	0.147	-

Table 1: Results of kinematic limit analyses.

The existence of damage, nearly from the construction period, can be attributed to the limited strengthening measures that have been taken. The analysis of the structure “as is”, revealed that there is a number of significant exceedances in capacity at many positions. Failure is expected for seismic loads according to [4], while more extensive damage should be expected for seismic loads corresponding to near-fault phenomena. Moreover the results of kinematic analyses imply an out-of-plane failure for a wall of the mezzanine at the north façade for the NF-5 spectra and an overturning of the central apse for both the EC-8 and the NF-5 spectra.

In order to overcome these structural weaknesses, a retrofit and strengthening scheme was suggested that included the following measures: (i) replacement of tie-rods with steel bars of a 20mm diameter, and (ii) consolidation of the masonry through grout injections.

At this stage, the analysis demonstrated that SD level was attained for a_{gr}=0.20g. Following the procedure presented in Section 2.1, resulted that the nominal life of the interventions is: $t_{\delta r}$ = 19 years for γ_i = 1.2, $t_{\delta r}$ = 12 years for γ_i = 1.4 and $t_{\delta r}$ = 33 years for γ_i = 1.0.

At this point, given the possibility to perform further “ interventions” it was decided to either use carbon fiber sheets (CFRP) placed at selected locations of the structure in order to increase the tensile strength of the masonry walls or a stainless steel grid of a particular form and size [40]. Further analysis demonstrated that application of the additional interventions increased the strength of the church to meet the SD limit state as specified for a newly constructed structure.

5 CONCLUSIONS

The study: (i) demonstrates the pressing need to develop a common approach for the assessment of seismic risk of historic structures and monuments. It acknowledges the presence of several national codes and attempts for code development and comments on their approaches; (ii) by introducing the notion of “nominal life of interventions” it proposes and demonstrates through an example, a framework that quantifies the duration of an intervention in order to achieve a predefined limit state; (iii) presents fundamental concepts on “near-field” phenomena that should be included in the analysis of monuments; (iv) clarifies numerous issues related to the use and limitations of the most widely used numerical solution methods and, finally; (v) presents a comprehensive evaluation and retrofit study of a historic structure that: (a) combines in-situ and laboratory testing with structural analysis using the notion of “nominal life of intervention”, (b) demonstrates the significance of near-fault effects and, (c) leads to the selection of appropriate intervention measures that use both traditional and modern state-of-the art techniques that respects the historic and architectural characteristics of the monument.

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