Rapid Seismic Assessment Procedure of Masonry Buildings with Historic Value

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Abstract In light of the increasing interest in rehabilitation of heritage neoclassical buildings of the 19th and 20th century in Greece, often restricted by international treaties for non-invasiveness and reversibility of the intervention and given the practical requirements for the buildings' intended reuse, the present study focuses on the investigation of the parameters that influence the outcome of their seismic assessment through simulation. This class of load-bearing masonry buildings, which is also present in many European countries, are marked by carefully engineered configuration (layout in plan and elevation, systematic location of openings) that can lead to a specific type of seismic response. This study presents a relatively simple and rapid analysis procedure that, for this special class of buildings, can produce very dependable results compared to those obtained from time-consuming dynamic analyses, in a much easier and fast way. The accuracy of the introduced methodology is evaluated through comparison of the results calculated from the proposed method with calculated seismic responses obtained from dynamic time-history analysis using as case studies two representative historical buildings located in the seismically active region of Thessaloniki. For the study a total of ten strong ground motion records are considered, five of which had near-field characteristics.

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Also presented is a qualitative comparison of the location and extent of anticipated damage, as estimated from the proposed rapid analysis procedure, compared with post-earthquake reconnaissance observations.

Keywords Seismic assessment · Heritage buildings · Pushover analysis · Unreinforced masonry structures (URM)

1 Introduction

Historic neoclassical buildings of the 19th and 20th century are a significant part of the built environment in many cities across Europe. Having a lifetime of more than a 100 years, load-bearing masonry buildings of this class are a living part of the European history and they define the ambiance of many of its cities. For this reason, they are protected by international treaties and organizations. Over the several decades of their service life most of those buildings have suffered structural damages of different severity, especially in countries of the Mediterranean basin due to the high seismicity. Yet, even today they remain in good condition, being operational in many cases (Fig. 1). Recognizing the historical importance and significance of neoclassical urban buildings as examples of an architectural school of thought, an increasing interest for their rehabilitation has recently emerged, often regulated by international treaties for noninvasiveness and reversibility of the intervention, combined with the practical requirements for the buildings' modern day intended reuse.

In the effort to assess the residual strength of historic and heritage buildings, reduced from a vague undetermined value which represents the initial state and in designing the appropriate retrofit measures for upgrading, sophisticated finite element analysis programs combined with powerful computing means have become a valuable tool for Structural Engineers. Yet, despite the capabilities which can derive from the use of modern technology, the obtained results are not necessarily reliable, as they often fail to recognize or reproduce important structural phenomena in the modeling process, or due to lack of convergence owing to inherent limitations of the analysis algorithms. As a result, in the process of seismic assessment of historic or heritage buildings of the 19th and 20th century the residual strength of the corresponding structure can easily be underestimated, which could lead to rather invasive choices of rehabilitation methods that can alter or destroy the unique historical or architectural features of the building in the interest of perceived needs for strength increase of the structure.

The objective of this paper is to present a rapid, yet efficient procedure for the determination of the seismic demand of this class of unreinforced masonry buildings. The introduced methodology requires little computational effort and is easy to use by practitioners that have a working skill in standard Finite Element analysis for

Fig. 1 Heritage buildings of the 20th century located at the center of Thessaloniki, Greece

gravity loads. Yet, it produces results of comparable accuracy and reliability with those derived from a more complex and time-consuming time-history dynamic analysis. To illustrate this point, results obtained from application of the proposed rapid seismic assessment procedure on two Neoclassical buildings of the 19th century are compared with responses calculated from detailed time-history analyses.

2 Practical Difficulties in Computer Modelling of URM Structures

Eurocode 8-III [1] provides guidance for assessment of existing structures, which rides on analytical estimations of seismic demand that may be calculated from a number of analytical alternative representations of the structure. Those representations are ranging from equivalent single degree of freedom systems to detailed three-dimensional modeling of the geometrical details with consideration of the regions of nonlinearity. The seismic hazard may be represented through an acceleration spectrum, or alternatively by the acceleration time-history, which requires step by step integration through time. Spectral representation of the seismic hazard lends itself to modal superposition, provided that the structural model is linearly elastic. It may also be combined with an independently established “static” pushover in order to determine a “performance point” for the structural system. These two general options regarding choices of representation of the structure and the load, involving different degrees of complexity, when combined in all possible ways yield an array of several different possible alternative methods that may be used for demand estimation, all more or less acceptable in practice. This variety rides on the assumptions that (a) where needed, available computer software supports nonlinear modeling of the individual member components and (b) no premature, brittle modes of failure that could macroscopically cause a post-peak softening branch would occur over the range of calculated seismic response; such an occurrence would be identifiable trough a negative or zero pivot in the stiffness matrix of a structure.

The above assessment code builds on established computer modeling technologies for lumped systems, mostly frames, for which most commercial codes enable modeling of lumped or spread nonlinearity and detailed time-history calculations. And because it calls on concepts that are general in principle, it is considered applicable and easily extendable to all types of structures, including URM buildings. However, when attempting to practically apply the above ideas to the simplest of these structures, a number of stumbling blocks may be encountered. For one, the state of the art in structural software today does not address the requirement (b) above: URM is brittle and thus maintaining a positive definite stiffness of pier members after cracking is not possible, particularly in tension-controlled modes of failure. Furthermore, most of the available commercial software packages today do not offer the option for 3-D analysis using nonlinear shell elements, which are needed to model masonry wall behavior. So, accounting for nonlinearity in this class of structures is restricted to either one-dimensional elements (beams, trusses, springs and gap elements) that can be used to modeling secondary elements (such as timber beams in diaphragms and roof trusses) or points of contact (such as unilateral contact at the point of embedment of a timber beam in a masonry wall using gap elements and the contact between foundation masonry with the surrounding soil, modelled using springs with asymmetric properties, etc.). Last, most commercial software packages today do not offer complete options for nonlinear dynamic response estimations, except for combinations of modal response maxima (which, being based on the principle of superposition, precludes the option for even considering secondary sources of nonlinearity). An added difficulty emanates from the distributed character of URM structures. As the number of modes generated is proportional to the total number of degrees of freedom in the structure, there is no clearly prevalent "first" or "fundamental" mode. The mode with the highest period is oftentimes associated with vibration of a single secondary component (such as a diaphragm timber beam), with insignificant ratio of mobilized mass. Previous studies by the authors and co-workers [2, 3] have illustrated that in some cases with flexible diaphragms several hundreds of modes need be included in the calculation just so as to mobilize 70 % of the total mass in lateral translation. This numerical circumstance in practice nullifies the so called equivalent single degree of freedom representation of the structure, which, combined with the pushover analysis methods, forms the backbone of modern code methods for seismic assessment and design (see EC8-I, Appendix B [4]).

Special, research-oriented software may be used instead to conduct detailed time-history analysis of URM structures (e.g. ABAQUS, DIANA, etc.); the effort required is disproportionately higher than the degree of confidence in the actual values of the input parameters concerning both the materials and the description of the seismic hazard, violating a fundamental principle of modern simulation. Therefore an urgent research need is facing the earthquake engineering community, regarding formulation of a simple framework for seismic assessment of URM structures that could also be used to guide seismic retrofit. This objective is the motivating interest in the present paper: a seismic assessment method that produces results of equivalent accuracy to detailed time-history dynamic analysis-based assessment procedures, yet requires significantly shorter computational time is presented and specifically tailored to the morphology and particularities of older URM structures. Here, the seismic hazard is specified in its spectral format (total acceleration and relative displacement) so as to render the methodology compatible to design code formats.

To deal with the uncertainty associated with a dependable estimation of the structural period, all structures up to two-storey high (the most common sample of the URM heritage building population in southern Europe) may be evaluated at the end of the constant acceleration range of the spectral plateau. The principles of generalized single degree of freedom representation of complex distributed systems are used to convert the structure to an ESDOF system consistent with the established code procedures—the fundamental response shape is almost a heuristic approximation of the fundamental translational mode of vibration, used as a tool for a global to local transformation of displacement demands thereby identifying locations of anticipated damage. The behavior factor q is obtained from the peak ratio of demand to supply in terms of out-of-plane moments of the free standing walls of the structure and is subsequently used to modify spectral displacement estimates through pertinent q-μ-T relationships.

Application of the proposed method provides information about the condition assessment of the structure and the anticipated damage localization at the state of the building's maximum seismic response, based on the translational modal characteristics of the building. As illustrated in the presented example analyses of two neoclassical buildings of the late 19th century, the proposed method can lead to equally dependable estimates as the results obtained from complicated and time-consuming dynamic time-history analysis.

3 Procedure for Seismic Assessment of Historical Buildings Based on Their Fundamental Mode Shape

The significance of the fundamental response shape as a diagnostic tool for seismic assessment of existing structures has been illustrated in recent studies in the field of seismic assessment [5, 6]. The fundamental translational shape is a compound property that conveys information about the tendency for localization of deformation demand in the structure. Therefore, the fundamental shape of a structure can be used to identify likely points of concentration of anticipated damage through the distribution of relative drift, while at the same time identifying lack of stiffness and the relative significance of possible mass or stiffness discontinuities (Fig. 2).

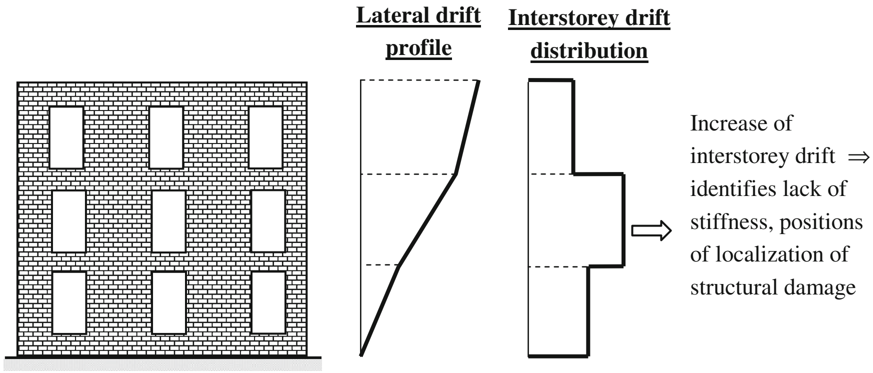

Fig. 2 Use of the building's deformed shape in the identification of the developed structural damage points

Furthermore, the fundamental response shape of any structure with established diaphragm action at the floor levels has been shown to correlate very well with the structure's displacement profile at the state of maximum roof displacement [7]. This conclusion was derived from the results of parametric dynamic analyses of several R.C. building models, both simplified and detailed, accounting for different types of geometric configuration that were subjected to ground accelerations of different characteristics. It was shown that the deformed shape of any multi-storey structure in the presence of diaphragms resembles the fundamental mode shape (i.e. the mode shape that mobilizes the greatest percentage of its total mass) at the point of maximum roof displacement, especially when the period of a building's fundamental mode is in the range of the predominant period of the seismic vibration. This conclusion was further extended to nonlinear systems, where the fundamental shape refers to the eigen-mode associated with the secant stiffness matrix at the instant of peak response.

Given the fact that some floor types used in historical or heritage masonry buildings of the 19th and 20th century provide adequate diaphragmatic response, the fundamental mode shape of those buildings has also been used to identify potential damage locations under earthquake; recent studies (Karantoni et al. [3], Pardalopoulos and Pantazopoulou [2], Kontari [8]) have tested application of the same concept in URM buildings with flexible diaphragms, with good success. To do so, a three-step procedure for seismic assessment of the URM historical buildings is presented, which produces results of comparable accuracy with time-history dynamic analyses. The three steps of the proposed procedure are:

1. *Determination of the fundamental translational mode of the URM building* The fundamental translational modes along the primary plan directions of a building may be estimated by subjecting a three-dimensional finite element model of the building to a notional gravitational field that is taken to act horizontally, in one of the directions of interest (i.e. along the longitudinal and transverse directions

of the plan geometry). Considering that gravitational forces are proportional to the mass of the structure whereas the restoring forces in free vibration are equilibrating these mass-proportional inertia forces, the deflected shape of the structure obtained from this solution is thought to be the closest approximation to the translational mode of vibration, since the associated natural frequency would result from the ratio of the work-equivalent inertia force and restoring force [e.g. Rayleigh's approach, recommended in EC8-III [1] Appendix B, and described in detail in [9]]. This procedure is more suitable for the determination of the fundamental translational modes of load-bearing masonry structures as in systems with distributed mass modal analysis using F.E. will lead to a large number of very similar translational modes with closely spaced periods, each having a small participation factor, thereby leading to the requirement of inclusion of several modes in the calculation in order to mobilize a respectable fraction of the structural mass. On the contrary, for this class of buildings, the fundamental translational modes that are estimated with the use of a notional gravitational field, as described above, mobilize significant mass, similar in magnitude to that which is calculated at peak seismic response from step-by-step time history analysis. Taking into consideration the brittle response of URM, which cannot secure a positive definite stiffness of pier members after cracking, the examined building can be simulated as a linear finite element model, with localized points of non-linear response (i.e., nonlinear elements at points of contact or in modeling secondary elements).

2. *Calculation of the seismic response of the building at the state of its maximum seismic response* Based on the postulated proportionality between the fundamental translational mode of a structure and the corresponding deformed shape at the state of its maximum seismic response [7], this response can be estimated from the spectral demand [4]. For this reason, the amplification factor f_i is introduced:

$$f_i = S_{d,i}(T) \big/ U_{Roof,i} \tag{1}$$

where, $S_{d,\,i}(T)$ is the spectral relative displacement demand in plan direction i (x, or y) and $U_{Roof,\,i}$ is the horizontal translation at the roof level of the building in the corresponding direction, i, as calculated in the previous step of the proposed procedure. The same scaling (through f_i) maybe applied in the estimated member forces from the analysis of Step 1, in order to obtain a rough estimate of peak member forces/stresses during the ground excitation. The building's fundamental period, T, that is used with the design spectrum to obtain the demand $S_{d,\,i}(T)$ is approximated by (EC8-I [4]):

$$T = C_t \cdot H^{3/4} \tag{2}$$

In Eq. (2), H is the total building height, in m, measured from the level of foundation or the level of rigid basement and $C_t \approx 0.05$ [10]. To keep matters simpler, the displacement demand alternatively may be associated with the end of the plateau region of the Design Code Type I spectrum (EC8-I [4]).

3. *Determination of local seismic demand and application of acceptance criteria* Bearing capacity in URM historical structures can be best identified by the amount of deformation occurring in the various components of the structure. The use of deformation demand for the purpose of seismic assessment is more meaningful than force demand estimation—based on the equal displacement rule, elastic displacement demands are close to the inelastic ones, whereas forces in the nonlinear analysis are vastly different from the elastic values. Performance criteria are also specified in terms of relative drift ratio—the drift capacity may refer either to URM piers deforming laterally so that drift refers to the relative deviation of the pier ends from vertical, or alternatively, it may refer to URM facades deviating from their horizontal initial orientation. These parameters are referred to as relative drift ratios in height and in plan of the examined building, θ_{height} and θ_{plan} respectively. θ_{height} is defined as the horizontal displacement difference that occurs between the top and the bottom of each of the vertical structural elements (i.e. piers and walls) at each storey of the building, divided by their vertical length, whereas θ_{plan} is defined as the relative lateral displacement of any two points of the plan perimeter, divided by their horizontal distance. In this regard, the most meaningful pair of points to be used at the crest of the building (or at the floor levels) is the point of peak outwards deflection in the wall orthogonal to the earthquake action and the point at the corners where transverse walls are intersected by walls parallel to the earthquake. Deformation measures calculated above can be used to determine the performance level (characterization of damage level) attained by the structure in response to the design earthquake. Cracking rotations (drift ratios) in masonry elements are in the order of 0.15 %, but the available ductility capacity varies depending on the type and reinforcement (e.g. timber lacing) of the URM walls. In well-constructed masonry a drift capacity of 0.5 % (drift ductility of 3.5) may be attainable, whereas for timber-laced or adobe masonries even larger values may be depended upon. But plain unreinforced masonry without timber lacing is unlikely to be able to support rotation or drift ductility in excess of 2 (a drift ratio of 0.3–0.4 %, EC-8 Part III [1]).

4 Demonstration of Proposed Method in Seismic Assessment of Two Neoclassical Buildings of the 18th Century

To demonstrate the accuracy of the proposed analysis procedure as compared to the corresponding results obtained from detailed time-history dynamic analysis, a series of test analyses have been performed in three-dimensional finite element models of two neoclassical building located in Thessaloniki, Greece. The buildings were constructed in the end of the 19th century, according to the designs of Ernst Ziller and they housed at the time the Hellenic high school and the Hellenic consulate of Thessaloniki, respectively. Both buildings operated continuously for more than 80 years until June of 1978, when they suffered damages by a strong earthquake of 6.5 M_w that struck the city of Thessaloniki (the epicenter was located about 40 km North-East of Thessaloniki, in the Volvi lake region).

4.1 Description of the Examined Buildings

The *Hellenic High School* of Thessaloniki is a two-storey building with a basement and a timber-framed roof (Fig. 3a), constructed in 1893 [11].

The building has a 20.85 m × 19.58 m plan, symmetrical with respect to a main corridor that is spanning from the northern side of the building to the southern, whereas the external building height, from ground level to the roof top, is 14.20 m. Initially built dividing walls form an integral part of the building's structural system. The walls of the basement are made of stone, having a thickness equal to 0.75 m in the perimeter of the building and 0.65 m in the inner plan. Walls of the first and the second storey were built of solid brick. Perimeter walls are 0.50 m width, whereas internal walls are 0.40 m thick. Floors of the first and the second storey were made of double T iron beams having a 60 mm × 180 mm cross section, spaced at 0.70 m along the small sides of the rooms and the corridors (i.e. having an E-W orientation over the building's corridors and a N-S orientation over the building's halls), whereas brick-arches spanning in the transverse direction between successive iron beams were encased between the upper and lower flanges of the double T beams. The total thickness of building's floors (including the finishing) is 0.33 m at the location of the iron beams and 0.25 m at the highest point of the arches. The last storey is covered by a roof made of timber trusses spanning in the east to west direction of the building.

The *Hellenic Consulate* of Thessaloniki is another two-storey sample of the same period and type of construction (Fig. 4a), built in 1898. The building operated continuously until 1978, when it suffered heavy damages from the earthquake. Building plan dimensions are 19.40 m × 15.21 m. External building height, including the roof, is 14.45 m. Initially built dividing walls form an integral part of the building's structural system.

The walls of the basement are made of stone, having a thickness equal to 0.65 m in the perimeter of the building and 0.55 m in the inner plan. Walls of the first and the second storey were built of solid brick. Perimeter walls are 0.55 m thick, whereas internal walls' thickness varies from 0.10 to 0.45 m. Floors of the first and the second storey were made of double T iron beams, whereas brick-arches spanning in the transverse direction between successive iron beams were encased between the upper and lower flanges of the double T beams. The last storey is covered by a roof made of timber trusses spanning in the north to south direction of the building, whereas a penthouse was built in the North-East corner, having a concrete roof.

Fig. 3 The *Hellenic High School* of Thessaloniki: **a** North view, **b**–**d** plan views of the building's basement, 1st and 2nd storey, respectively [12]. The building design followed the archetype designs of Ernst Ziller, modified by Architects Kambanakis and Kokkinakis and sponsored by A. Syggros

4.2 Modeling and Analyses Details of the Two Examined Buildings

The seismic response of the two buildings to various earthquake excitations was examined using three-dimensional finite element analysis (Fig. 5, [13]). In all building models walls were idealized using four-node shell elements (6 d.o.f. per node, supporting nodal forces and flexural moments). Floors were modelled using linear elements for the iron beams and shell elements to represent the brick arches spanning between steel beams. Linear elements were used at the roof level, accounting for the horizontal timber beams of the roof trusses. In all models, the response of the shell and the linear elements was considered elastic. The modulus of elasticity of stone and bricks was considered 1000 times the value of the corresponding compressive strength, f_k; this variable was taken equal to the following:

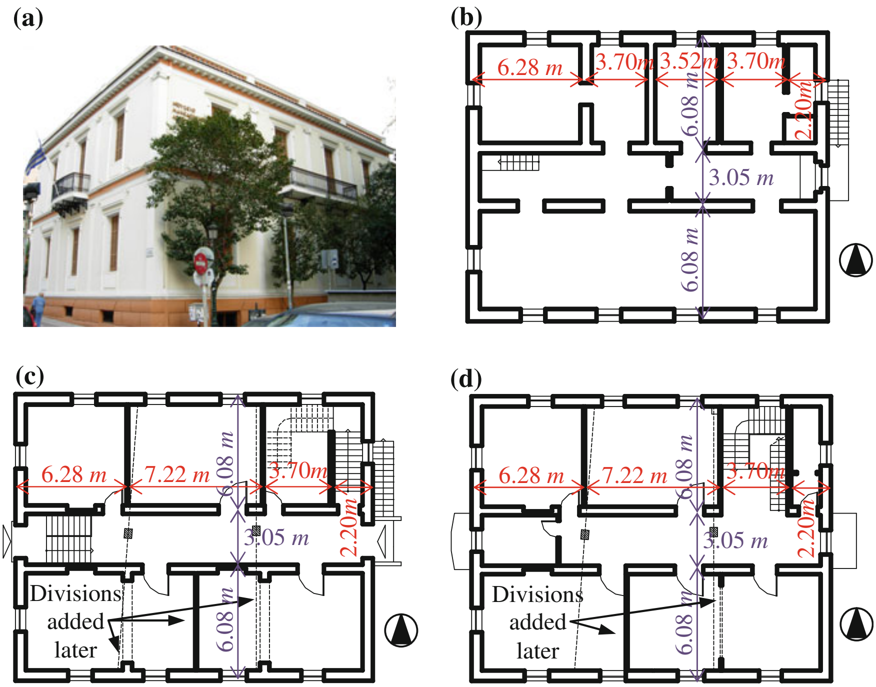

Fig. 4 The Hellenic Consulate of Thessaloniki: **a** South-West corner of the building, **b**–**d** plan views of the building's basement, 1st and 2nd storey, respectively. Built in 1898 according with the designs of Ernst Ziller; sponsored by A. Syggros to house the Hellenic Consulate of Thessaloniki while the city was still under Ottoman rule. After the annexation of Thessaloniki to Greece, the building operated as a primary school [11, 12]

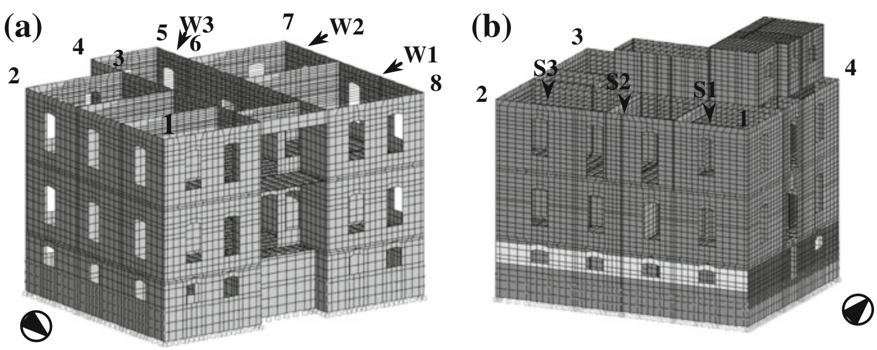

Fig. 5 Three-dimensional finite element models and identification of the edges and the wall points of **a** the *Hellenic High School* and **b** the *Hellenic Consulate* of Thessaloniki

(a) in the case of the *Hellenic High School* for stone f_k = 5.5 MPa, for solid bricks f_k = 4.0 MPa and for voided bricks f_k = 1.5 MPa, whereas in the case (b) of the *Hellenic Consulate,* for stone f_k = 6.3 MPa and for solid bricks f_k = 5.2 MPa. In the case of frame (iron and timber) elements, the modulus of elasticity was taken equal to 150 GPa for the iron beams, 10 GPa for timber in the longitudinal direction of the beams and 1 GPa in the other two sectional directions. In all cases self weight of the building was calculated according to the material density; this was taken equal to 28.5 kN/m^3 for stone, 18 kN/m^3 for solid bricks and 14 kN/m^3 for voided bricks. A roof weight equal to 1.5 kN/m^2 was assumed, uniformly distributed along the linear elements of the roof trusses according to their tributary area. Service loads were considered equal to 2.50 kN/m^2 for the roof and 3.50 kN/m^2 for the floors of both buildings. Masses considered in the dynamic analyses were automatically calculated by the program, by multiplying each element (shell or linear) volume by their respective density.

To account for the effect of the ground motion parameters a suite of ten earthquake records was used in conducting time-history dynamic analyses. All of the acceleration records derived from past strong earthquakes that have occurred in Greece between 1978 and 2014, which caused different types and extents of damages in numerous documented cases of URM buildings of the same type as those examined herein and of the same period of construction (19th and early 20th century). Recorded datasets were selected from the ITSAK earthquake database [14]. From among the ten earthquake records considered in the analyses, five were recorded in the near-fault zone (i.e. within 20 km from the rupture fault), whereas the other five datasets were recorded on sites with a distance from rupture fault ranging from 22 to 40 km. From among the three components recorded for each earthquake case (two horizontal and one vertical) the record used in dynamic analyses corresponds to the horizontal component with the maximum recorded absolute peak ground acceleration (*PGA*). These components were then applied separately in each of the two principal directions in plan of the examined buildings. Table 1 presents the characteristics of the ten earthquake records used in the dynamic analyses, whereas their absolute acceleration and relative displacement response spectra, S_a and S_d respectively, as those were calculated considering a viscous damping equal to ξ = 5 %, are presented in Fig. 6.

4.3 Comparison Between the Results of the Rapid and Time-History Analyses

To demonstrate the accuracy of the results obtained from the proposed rapid analysis procedure as compared to the time-history response of URM buildings at the instant of peak displacement, first the distribution of the horizontal deformations developed height-wise in the examined buildings is investigated. Figure 7 depicts the lateral displacement profiles of the four edges of the Hellenic Consulate (for the location of each edge see Fig. 5b) when the earthquake excitations are imposed in

Table 1 Earthquake cases considered in the analyses and their characteristics

Earthquake	M_L	Station	Epicentral distance (km)	Record type	Record component	PGA (g)
Kythera, 08/01/2006	6.4	KYT1	>20	FF	L	0.122
Limnos, 24/05/2014	6.3	LMN1	>20	FF	E	0.106
Kefalonia, 03/02/2014	6.1	CHV1	8.85	NF	E	0.755
Kozani, 13/05/1995	6.1	KOZ1	16.38	NF	L	0.216
Volvi, 20/06/1978	6.0	THEA	26.35	FF	T	0.150
Lefkada, 14/08/2003	5.9	LEF1	<10	NF	T	0.417
Alkyonides, 25/02/1981	5.9	KORA	26.55	FF	T	0.137
Aigio, 15/06/1995	5.6	AIGA	21.56	FF	T	0.517
Kalamata, 13/09/1986	5.5	KALA	12.30	NF	T	0.297
Athens, 07/09/1999	5.4	ATH2	19.63	NF	T	0.159

NF Near Fault record, *FF* Far Fault record

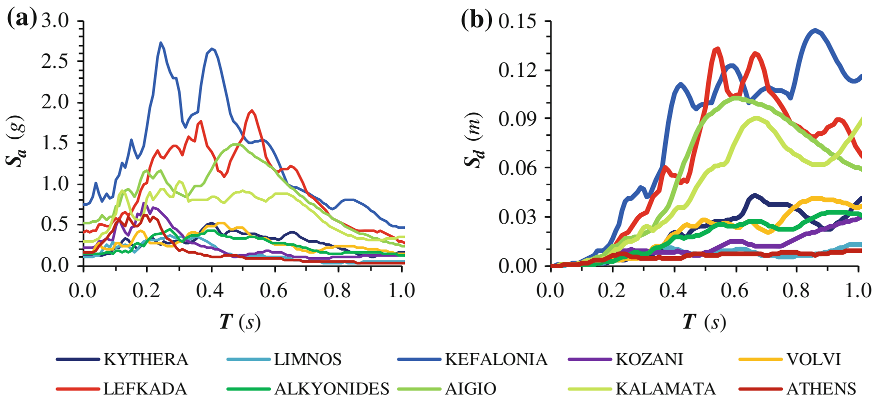

Fig. 6 Response spectra of the earthquake records used in the analyses, calculated for 5 % damping: **a** absolute acceleration response spectra, **b** relative displacement response spectra

the East-West (E-W) and in the North-South (N-S) directions, respectively. In all graphs the black continuous lines represent the deformed shapes of the building's edges, as those derived after application of the proposed simplified analysis procedure in the respective directions. Also plotted in the same figure with coloured continuous lines are the normalized deflected shapes of the building's edges in the corresponding plan directions at the instant of maximum roof displacement during the ten different earthquake excitations, obtained from time-history dynamic analyses. All deformed shapes are normalized with respect to the displacement value at the roof level of the corresponding edge.

As illustrated in Fig. 7, the lateral displacement profiles in both E-W and N-S directions obtained from the proposed simplified analysis procedure and those

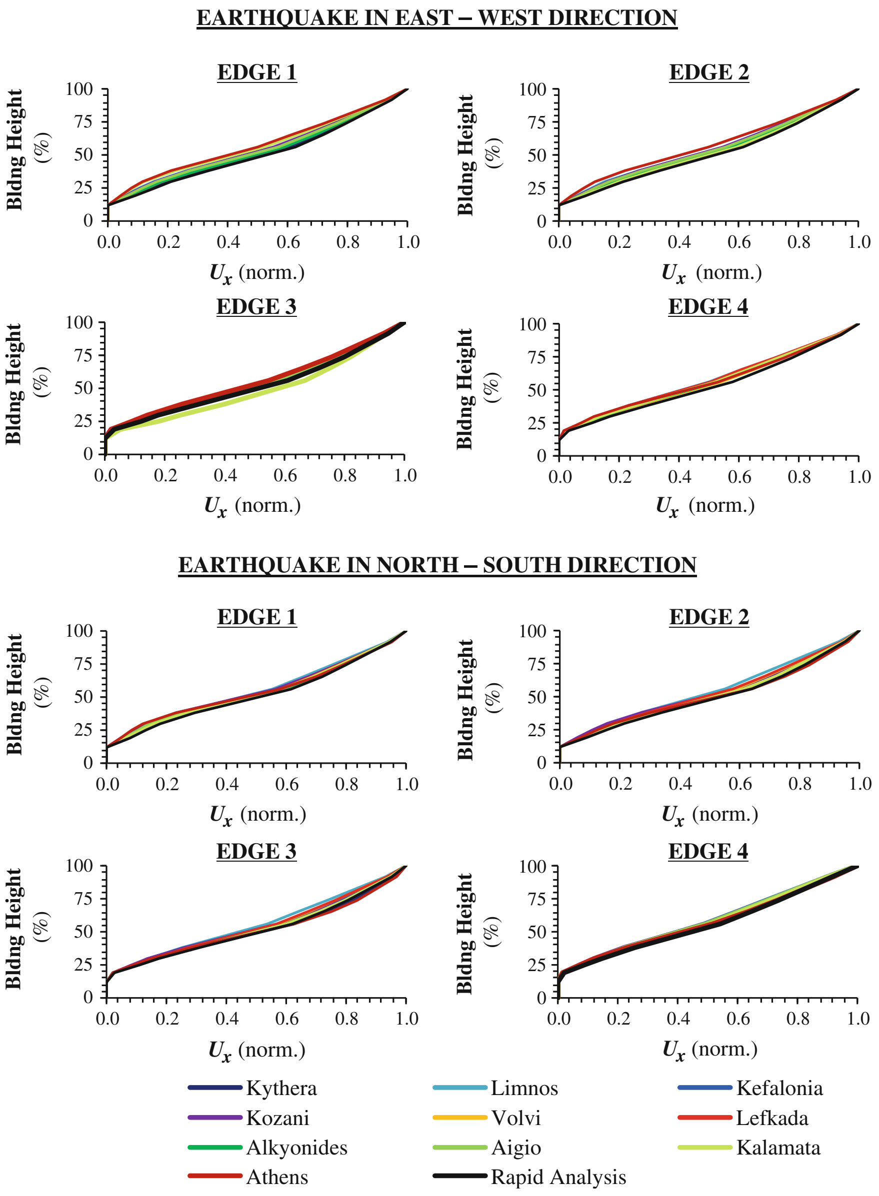

Fig. 7 Lateral displacement profiles along the four edges (see Fig. 5b for identification of location) of the Hellenic Consulate in the East-West and North-South directions at the instant of maximum roof displacement, when the building is subjected to earthquake excitation in the same direction

calculated at the instant of peak horizontal roof displacement from time-history dynamic analyses are closely matching. Variation between the lateral displacement profiles obtained from the two alternatives range between 0 and 9 % when the examined direction is the East-West and between 0 and 13 % when the North-South direction is considered, with a mean value equal to 2.3 and 3.4 % in E-W and N-S directions, respectively.

The same conclusions derive from the examination of Figs. 8 and 9, which illustrate the corresponding comparisons in the case of the seismic response of the Hellenic High School (see Fig. 5a for identification of nomenclature). Again, variation between the lateral displacement profiles obtained from the two alternative approaches range between 0 and 15 % in E-W and between 0 and 12 % in the N-S directions, respectively, with mean values of 3.5 and 2.9 % in the two directions, respectively.

Therefore, in terms of determination of the lateral displacement profiles, application of the rapid analysis procedure yields results of the same accuracy as a time-history dynamic analysis. Note that this accuracy also applies to the displacement profiles considered in the midspan of walls oriented orthogonal to the direction of seismic excitation. Yet, application of the introduced rapid analysis procedure requires significantly smaller computational time and means (the executed time-history analyses cases produced output files whose volume ranged between 13 and 65 GB and their execution time ranged between 3 and more than 30 h, whereas execution of the rapid analysis procedure in the same 3-D finite element models required less than a minute for each plan direction of the examined buildings and the volume of the produced output files ranged between 300 and 440 MB), much simpler to process and handle; this renders the rapid approach a useful tool in the hands of practitioners in the field of seismic assessment.

Note that due to the absence of well established diaphragm action across the floors of the two examined buildings, owing to the construction practices used in the period of their construction, peak lateral displacements of all perimeter points occurred nearly simultaneously in all earthquake inputs considered. Whether all points vibrated completely in phase or not, depends on the dynamic characteristics of the vibrating structure and the ground excitation. Figure 10 depict the time-histories of the calculated base shears and the horizontal displacements of selected points at roof level of the Hellenic High School in E-W direction for the ten ground motion cases acting in the same plan direction, whereas the same indices are depicted in Fig. 11 regarding the seismic response of the Hellenic Consulate in N-S direction. In these figures the black continuous line represents the waveform of the base shear value, Q, calculated from finite element analysis program [13] exactly 1 s before and after the point of maximum base shear response. The grey dashed lines represent the waveform of the horizontal displacements, U, of selected edges at the roof level of the two buildings in directions parallel to the imposed earthquakes, whereas the grey doted lines show the same response of selected points at the midpoint of the exterior transverse walls orthogonal to the seismic action (the location of each selected point of the Hellenic High School and the Hellenic Consulate is illustrated at Fig. 5a, b, respectively). In order to facilitate the

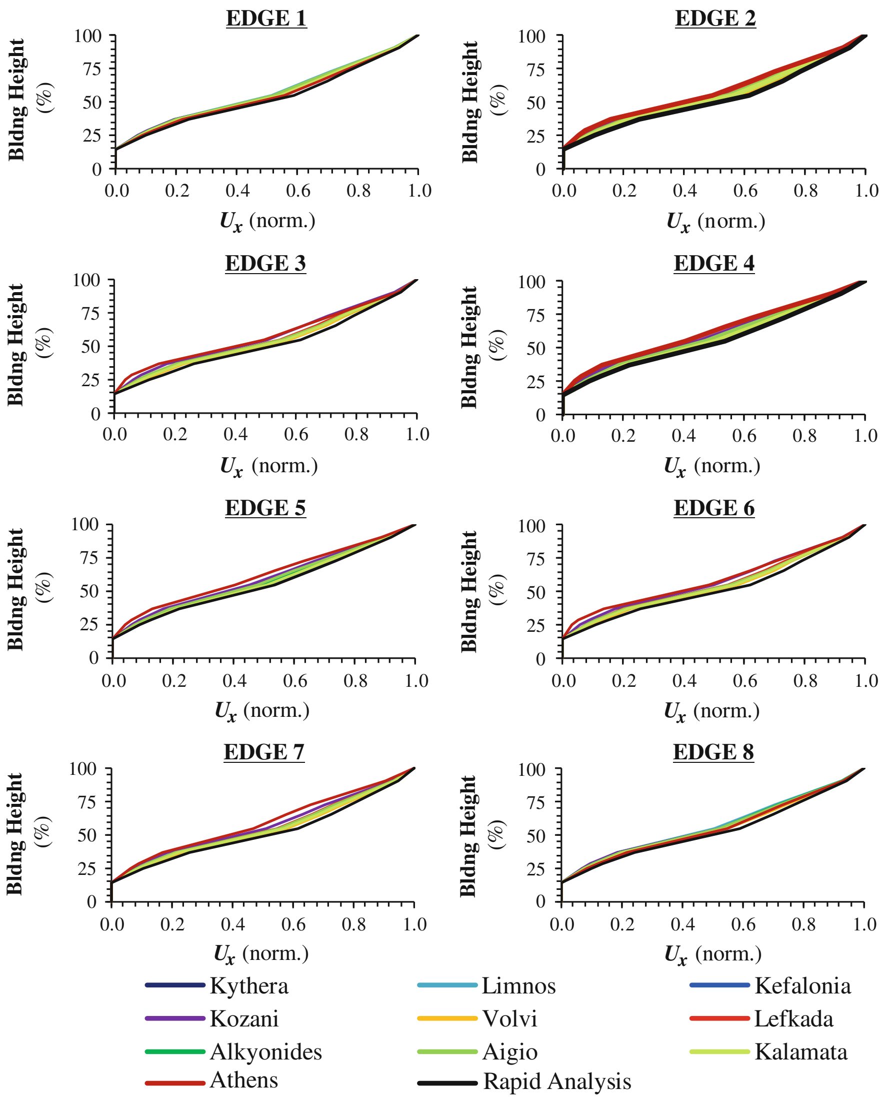

Fig. 8 Lateral displacement profiles of the eight edges of the Hellenic High School (for identification of location see Fig. 5a) in the East-West direction at the instant of maximum roof displacement, when the building is subjected to earthquake excitation in the same direction

comparison between the waveforms of the base shears and of the nodal horizontal displacements, all waveforms are normalized with respect their the absolute maximum value (i.e. in each graph the base shear is normalized with respect to the |max.| value of Q within the duration of the corresponding earthquake, whereas each U waveform is normalized with respect to its corresponding |max.| value). As observed in both figures, in most of the earthquake scenarios studied, all roof points

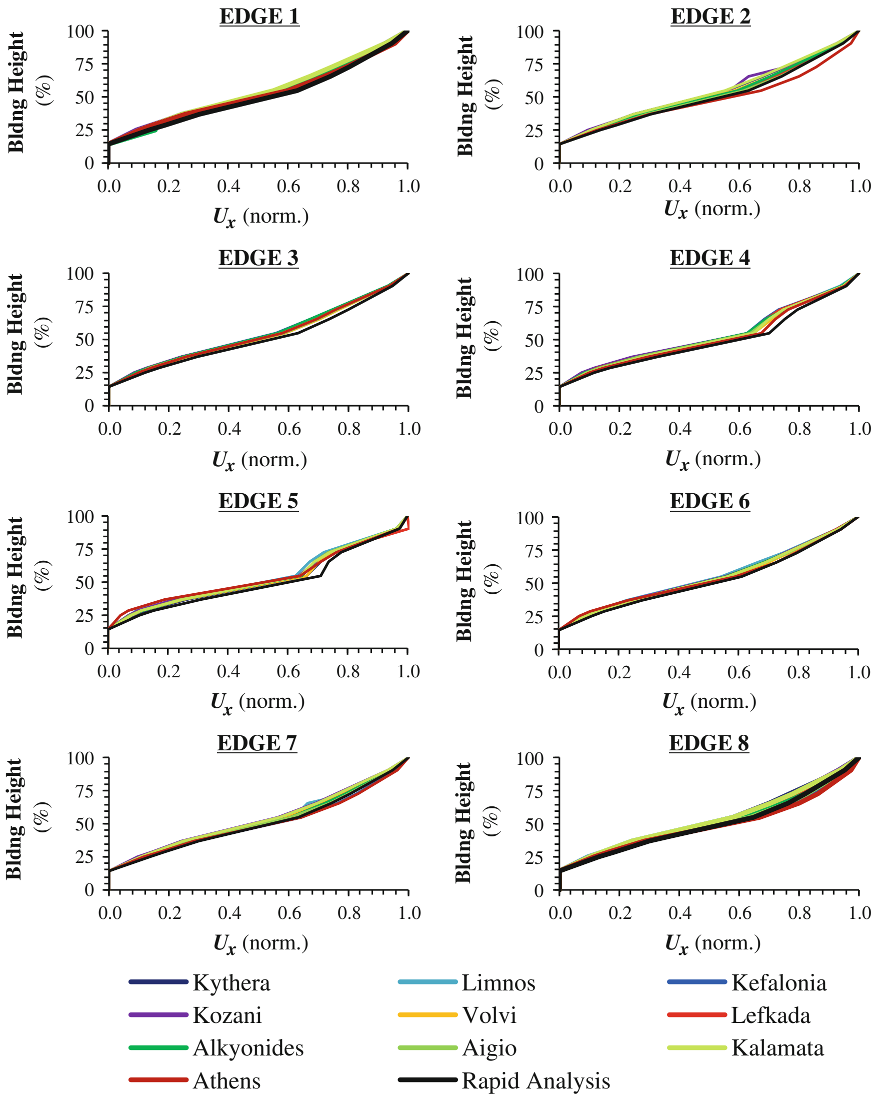

Fig. 9 Lateral displacement profiles of the eight edges of the Hellenic High School (for identification of location see Fig. 5a) in the North-South direction at the instant of maximum roof displacement, when the building is subjected to earthquake excitation in the same direction

vibrated completely in phase with the waveform of the corresponding base shear. These earthquake cases were also the ones with the best correlation between the actual displacement profiles of the buildings' edges and the corresponding deformed shapes that were determined by the rapid analyses procedure (Figs. 7–9). In some analyses cases (for example, the Kozani and Athens earthquakes in Fig. 10 and the Athens earthquake in Fig. 11) there is some phase difference between the

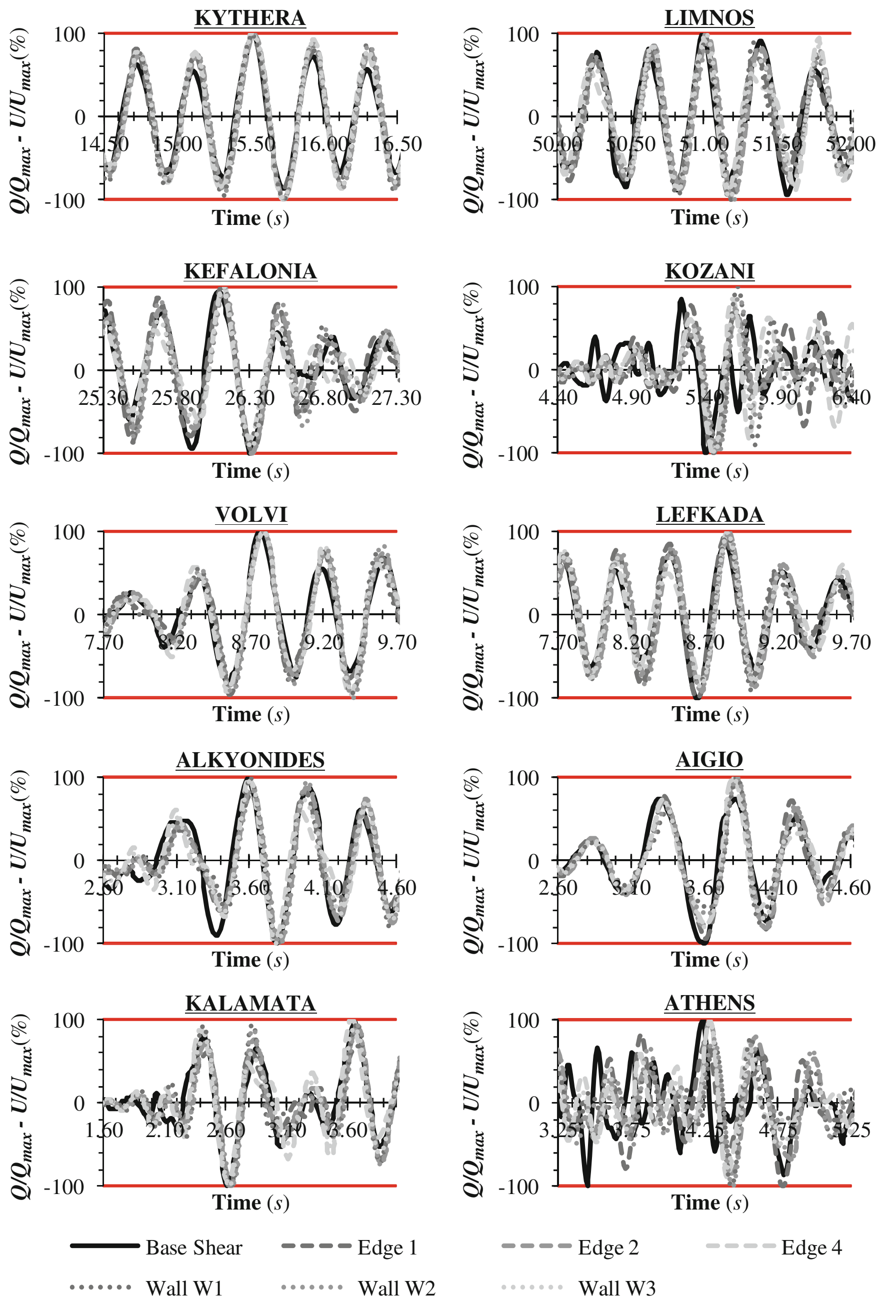

Fig. 10 Time-history of the developed base shear and horizontal roof displacements in East-West direction of the Hellenic High School for the ten earthquake excitations considered in the same direction (location of the selected points is illustrated in Fig. 5a)

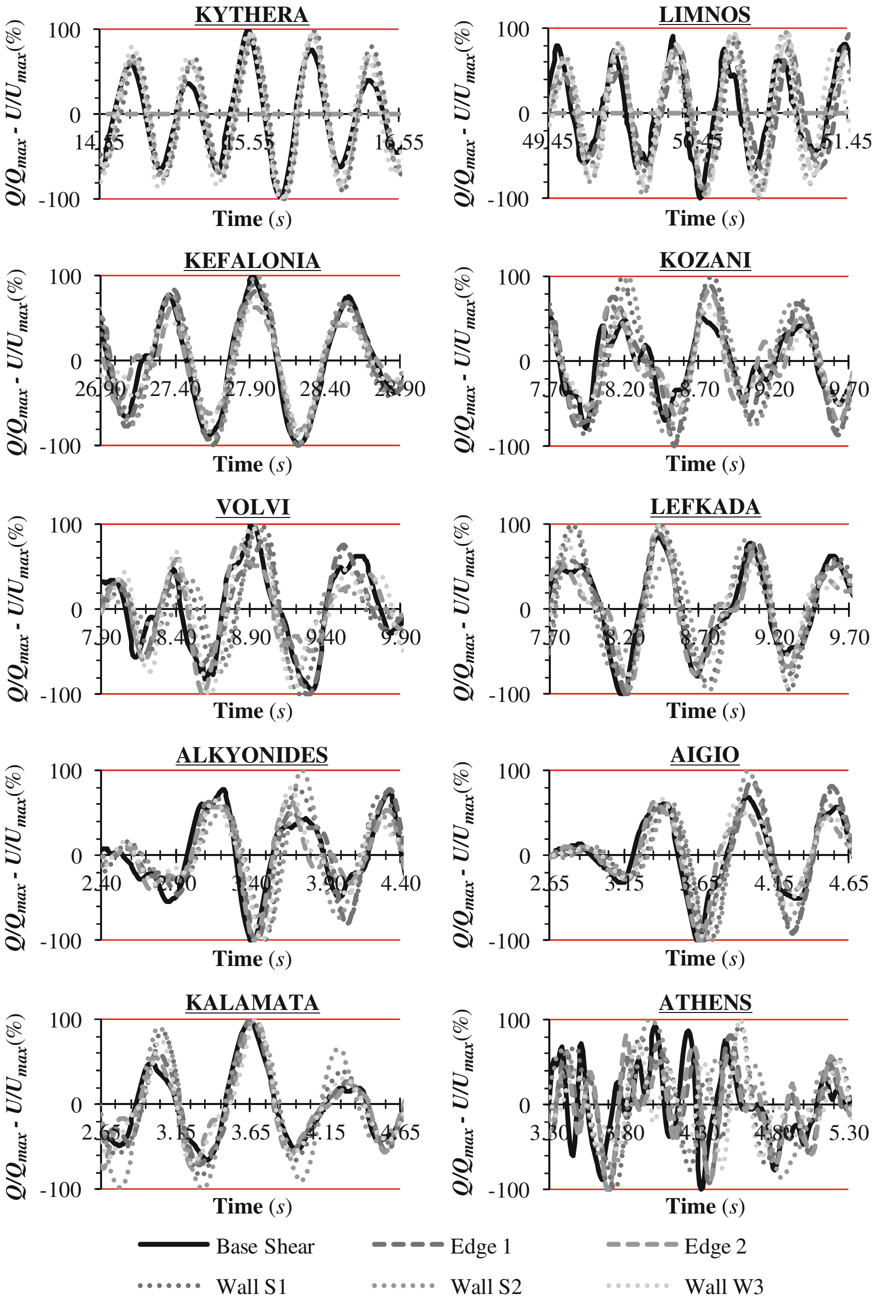

Fig. 11 Time-history of the developed base shear and horizontal roof displacements in North-South direction of the Hellenic Consulate for the ten earthquake excitations considered in the same direction (location of the selected points is illustrated in Fig. 5b)

various waveforms. This difference, which is responsible for the small deviation between the actual deflected shapes of the buildings and the deformed shapes obtained from the rapid analysis procedure (Figs. 7–9) is due to the participation of higher modes of vibration.

Yet, even in these cases where the combination of dynamic characteristics of the vibrating structure and the earthquake excitation can lead to participation of higher modes of vibration, the estimated response of the structure is actually more favourable as compared to the one determined by the proposed rapid procedure. This is due to the fact that in these cases the total mass of the building that is actively engaged in vibration at peak response (regardless of whether referring to the instant of peak base shear or the instant of peak lateral drift ratio) is smaller to the one corresponding to the building's lateral drift response, as this has been determined by the rapid seismic analysis—from a designers' perspective, the rapid analysis is far more conservative. Therefore, in the cases where the buildings' seismic response is affected by non-negligible higher mode participation, the magnitude of the developed deformations and forces or stresses along the buildings' components would be smaller roughly by the factor a_{HM}/a_{RA} as compared to the values calculated from rapid analysis, for the same target roof displacement. Here, a_{HM} is the mass participation factor calculated from the nodal horizontal displacements at the instant of peak dynamic response and a_{RA} is the mass participation factor associated with lateral translation derived from the rapid analysis procedure.

Figure 12 presents a comparison between the total mass participation factor, a_j, of the two examined buildings for the different analyses cases considered. In this figure, the horizontal, continuous, black line presents the mass participation factor in each of the E-W and N-S directions of the two examined building, as calculated from the finite element models' nodal masses, m_i, and the pattern (shape) of nodal lateral displacements, $U_{i,j}$, in the j-th plan direction (j = N-S or E-W) as per:

$$a_j = \frac{\left(\sum_{i=1}^{N} m_i \cdot U_{i,j}\right)^2}{\left(\sum_{i=1}^{N} m_i\right) \cdot \left(\sum_{i=1}^{N} m_i \cdot U_{i,j}^2\right)} \tag{3}$$

Also plotted in the same graph with horizontal, continuous, colour lines are the mass participation factors, a_j, as those were calculated from the time-history dynamic analyses nodal displacements $U_{i,\,j}$ that occurred at the instant of peak base shear in the corresponding plan direction, j. Finally, Fig. 12 plots with dashed black lines the total mass participation factor for each of the two plan directions of the buildings examined after conducting Eigen analyses with consideration of 1000 modes in each case. As depicted in all graphs of Fig. 12, the deformed shapes of the examined buildings that were determined from the rapid analysis procedure activate about 50 % of the buildings' total mass, in both of their plan directions, whereas in order to achieve the same through modal analysis an exorbitant number of modes need be considered. Note that in cases where the waveforms of base shear and nodal displacements have phase difference, the mass participation factor is reduced as compared to the cases where all waveforms are in phase, as well as from the

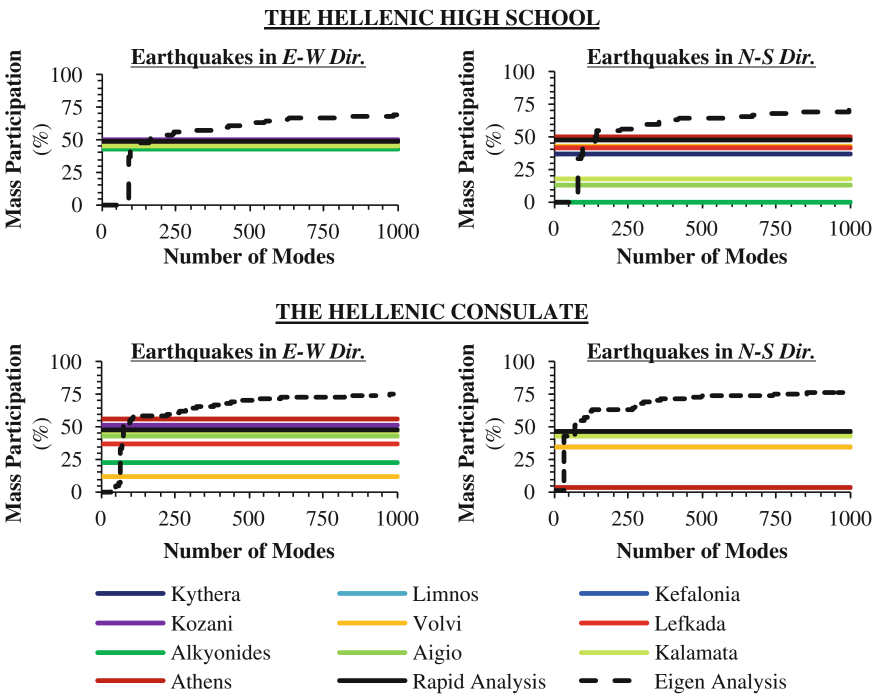

Fig. 12 Peak mass participation factor for the different analyses cases of the two case studies. The *horizontal colour lines* represent the values of a_j calculated from time-history dynamic analyses at the instant of maximum base shear, the *horizontal black lines* represent a_j calculated from rapid analyses and the *dashed black lines* illustrate the increase of a_j with respect to the number of Eigen modes considered

reference value of a_j obtained from rapid analyses. Also note that to obtain using modal analyses the same value of mass participation as that which is derived from the proposed rapid analyses, more than 150 modes would have to be considered for both the examined buildings, whereas the values of a_j do not increase significantly as compared to the values calculated from the results of rapid analyses even if as many as 1000 modes are considered (see Fig. 12).

5 Seismic Assessment Procedure for Historic and Heritage URM Buildings

To demonstrate how the proposed methodology for rapid estimation of seismic demand for URM buildings can be used in engineering practice an application example is presented based on the seismic response of the Hellenic High School during the 1978 Thessaloniki earthquake.

Table 2 Values of θ_{height}, calculated along the eight edges of the Hellenic High School (locations of the edges are illustrated in Fig. 13)

	E-W Direction—θ_{height} (%)		N-S Direction—θ_{height} (%)	
Edge	1st storey	2nd storey	1st Storey	2nd Storey
1	0.14	0.08	0.12	0.06
2	*0.16*	0.08	0.12	0.06
3	*0.15*	0.07	0.12	0.06
4	0.12	0.08	*0.18*	0.08
5	0.12	0.07	0.17	0.08
6	*0.16*	0.08	*0.16*	0.08
7	0.15	0.07	0.12	0.06
8	0.14	0.08	0.12	0.05

Values in *italic* correspond to *Operational/Immediate Occupancy* performance level. Values in *roman* correspond to elastic response with no damage

By using Eq. (2), where for the case of the examined building H is 13.32 m, the fundamental period of vibration in both of the building's plan directions is calculated as: $T_1 = 0.349$ s. Considering the relative displacement response spectrum of the used earthquake excitation component of the 1978 Volvi earthquake (yellow line in Fig. 6b) the target roof displacement of the examined building used in the rapid seismic assessment procedure is 0.0122 m, valid for the orthogonal directions in the plan. Note that the average values of peak displacement calculated from the corresponding values at the eight points of reference at the building crest, at the instant of peak base shear are 0.0163 and 0.0159 m in E-W and N-S, respectively.

The average horizontal roof displacements in the E-W and the N-S directions of the examined building derived from the rapid analysis procedure were 0.0038 and 0.0035 m, respectively. Therefore, calculation of the required seismic response parameter (developed displacement or stress) for conducting seismic assessment can be obtained by multiplying the corresponding parameter in the respective direction, obtained from the rapid analysis procedure, with the amplification factors $f_{E\text{-}W} = 0.0122/0.0038 = 3.21$ and $f_{N\text{-}S} = 0.0122/0.0035 = 3.48$, respectively (Eq. 1).

Table 2 presents the relative drift ratios in height, θ_{height}, that were calculated according to the proposed assessment procedure along the edges of the first and the second storey, in both E-W and N-S plan directions, whereas Table 3 presents the relative drift ratios in height and in plan, θ_{height} and θ_{plan} respectively, that were calculated at selected points of the external walls (for the location of all considered edges and points see Fig. 13). From the values of θ_{height} it is evident that during the 1978 Volvi earthquake the walls of the first storey of the building were subjected to larger deformations than the walls of the second storey (localization in first floor). Nevertheless, the developed deformations were not capable to cause severe damages to the building (max. value of θ_{height} equals to 0.18 %, which corresponds to the onset of significant cracking for masonry and is classified to the *Operational/ Immediate Occupancy* performance level). Similar conclusions may be stated for the in-plan rotation, θ_{plan}, in all storeys of the building. In this case, the values of

Table 3 Values of θ_{plan} at the middle of external walls of the Hellenic High School (locations of the points are illustrated in Fig. 13)

		θ_{plan} (%)		θ_{height} (%)	
Facade	Point	1st storey	2nd storey	1st storey	2nd storey
North	N_1	*0.16*	**0.32**	**0.23**	**0.21**
	N_2	*0.15*	**0.33**	**0.22**	**0.21**
South	S_1	*0.15*	**0.32**	**0.23**	*0.20*
	S_2	0.10	0.12	**0.21**	0.11
	S_3	*0.15*	**0.32**	**0.22**	*0.20*
East	E_1	0.07	0.12	*0.16*	0.11
	E_2	0.02	0.03	0.13	0.09
	E_3	0.11	*0.18*	*0.20*	0.12
	E_4	0.00	0.04	0.14	0.09
West	W_1	0.02	0.12	0.11	**0.22**
	W_2	0.13	*0.20*	**0.21**	0.12
	W_3	0.01	0.04	*0.15*	0.09

Values in *italic* correspond to *Operational/Immediate Occupancy* performance level; Values in *bold* correspond to *Repairable Damage* performance level. Values in *roman* correspond to elastic response with no damage

relative drift are slightly increased as compared to the corresponding values of θ_{height}. The maximum value of θ_{plan} equals to 0.33 %, which corresponds to a ductility level of 1.65 and can therefore be classified as *Repairable Damage* performance level.

In all cases, the results of the seismic assessment procedure were in compliance with the conclusions of the damage report regarding the building after the 1978 Volvi earthquake.

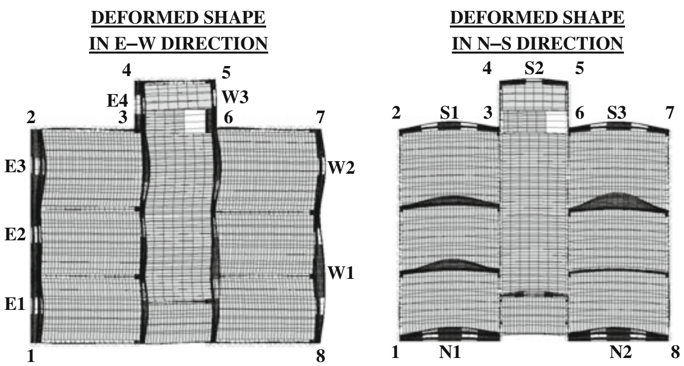

Fig. 13 Deformed shape of the Hellenic High School as this has derived from rapid analyses in East-West and North-South directions. Also presented are the points of θ_{heigh} and θ_{plan} calculations

6 Conclusions

Seismic assessment of load-bearing buildings of the 19th and 20th century is an emerging interest for the engineering community, as this class of buildings are a living part of the European modern history. Current assessment procedures, often regulated by international treaties for non-invasiveness and reversibility of the intervention combined with the safety requirements that are dictated by the buildings' modern day intended reuse, are based on a vast variety of analytical methods, ranging from the use of simplified mechanical models to the use of sophisticated finite element analyses programs combined with powerful computational means. Yet, the obtained results are often of limited reliability when these assessment procedures, which are a modification of design procedures used in frame structures with ductile behavior, are applied on URM structures with the known brittleness of the masonry material. Therefore, an urgent research need is facing the earthquake engineering community, regarding formulation of a simple framework for seismic assessment of URM structures that could also be used to guide seismic retrofit.

In this paper a rapid procedure for seismic assessment of URM historical buildings, which produces results of equivalent accuracy to detailed time-history dynamic analysis based assessment procedures while requiring significantly shorter computational time, is presented. According to the proposed method, a point of reference for seismic behavior assessment of the structure is the pattern of lateral displacements and internal forces obtained when the structure is loaded statically, in the horizontal direction, by a uniform gravitational field. This pattern is normalized with respect to the maximum lateral displacement at the roof level. Seismic demand is estimated in terms of displacement demand at the top of the building using a simple generalized single degree of freedom representation of complex distributed system, consistent with the established code procedures. The ESDOF displacement demand thus estimated is used to scale up the patterns of lateral displacements and internal forces obtained previously. Relative displacements between any two points in plan or in height of the structure, divided by their distance, define demand in terms of drift ratios. These drift ratios are subsequently compared with relative drift capacities, which, at the onset of significant cracking (apparent yielding) for masonry range between 0.15 and 0.20 % for both in-plane and out-of-plane deformation. From this comparison, estimation of ductility demanded throughout the structure is possible, thereby enabling assessment and localization of anticipated damage.

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