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VULNERABILITY OF TORSIONALLY SENSITIVE HISTORICAL BUILDINGS UNDER SEISMIC LOADS

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Abstract. *The paper explores the effects of asymmetry in plan owing to the occurrence of slender wings in massive monumental and historical URM buildings that belong to the post-renaissance neoclassical school of architecture. Although it is common in buildings with flexible diaphragms to disregard the implications of torsion, in this particular case these effects are significant causing important differential displacements and therefore a high risk of damage in the wings. To study the problem, three-dimensional finite element models of a neoclassical building with asymmetric plan shape located in Thessaloniki, Greece, are analyzed to a strong ground motion that has been recorded in the region and their maximum seismic response is explored. This response is then compared with results obtained from application of the rapid seismic assessment procedure of unreinforced masonry buildings, which has been recently introduced by the authors and conclusions on the procedure's accuracy are derived. Emphasis is placed on the sensitivity of the torsional aspects of the response to the stiffness of horizontal diaphragms that is provided by the architectural forms of the era under investigation. Application of this rapid methodology enables easy inspection over the building's geometry to identify those characteristics that may predispose the tendency for a strong torsional component of response, but it may also help identify the efficacy of retrofitting through the addition of diaphragms as a strengthening approach that may mitigate the occurrence of damage.*

1 INTRODUCTION

Monumental unreinforced masonry (URM) buildings built during and after the Renaissance were constructed to house government or state activities (palaces, universities, hospitals, etc) in many European cities, symbolizing at the same time the economical, political or cultural maturity of the society to which they belonged. Having a lifetime ranging from one to more than five centuries, today this category of buildings are a living part of the European history, serving as a reminder of the era in which they have been constructed.

Figure 1: Examples of massive monumental structures across Europe [1]: (a) the palace of Versailles (1770), (b) the Schoenbrunn palace (1750), (c) the Hellenic parliament (1847).

During their long period of existence, buildings of this class of massive load-bearing masonry have suffered significant damages in their structural system, especially in countries of the eastern Mediterranean basin that have high seismicity. Due to those buildings' historical importance, preservation and restoration of their unique architectural and structural characteristics is among the main priorities of the states that own them, often regulated by international treaties for noninvasiveness and reversibility of the intervention (Charters adopted by the general assembly of ICOMOS, [2]). Assessment of the residual strength of those buildings is therefore a necessary task in order to identify those cases that are most vulnerable and have a disposition for torsional response components owing to their plan morphology. Although in most of these cases retrofitting solutions for strengthening are restricted due to international treaties, an assessment methodology can shed light to the most pertinent strategies that might mitigate a possible accidental loss of part of this heritage in the future.

Assessing the residual strength of monumental URM buildings is a much more demanding task as compared to assessment of residential buildings of the same era, not only because of their massive volume but mostly because this type o buildings possesses special dynamic characteristics due to their Architectural design that may cause significant damages during an earthquake. In order to achieve the desired harmony, which was inspired from highly regarded classical forms of Hellenic antiquity, monumental multi-unit buildings have a system-plan organized along a main axis of symmetry, whereas in many cases a transversal axis of symmetry in plan also exists. To achieve the desired symmetry in plan, Architects of that era organized the building volumes in the form of an ***H***, an ***L,*** a ***Π*** or a ***T***, with courts located in the recesses of these shapes so that the overall plan could be thought to form a concave prism, whereas at each building volume long corridors are usually spanning from one end of the building volume to the other (Fig. 2). The long and slender wings are always susceptible to out of plane action; but in many cases of uniaxial symmetry the plan arrangement of monumental buildings is responsible for the development of in-plan torsional phenomena during a seismic excitation in direction orthogonal to the axis of symmetry, and this can cause significant damages at locations of the building that are remote relative to the center of twist of the complex building plan. This is why systematic development of structural damages mainly at the locations where separate building volumes are joined, as well as at the walls of corridors with significant length between successive transverse walls, due to dynamic displacements in the out-of-plane direction is frequently reported in post earthquake reconnaissance reports of monumental URM buildings (Fig. 2).

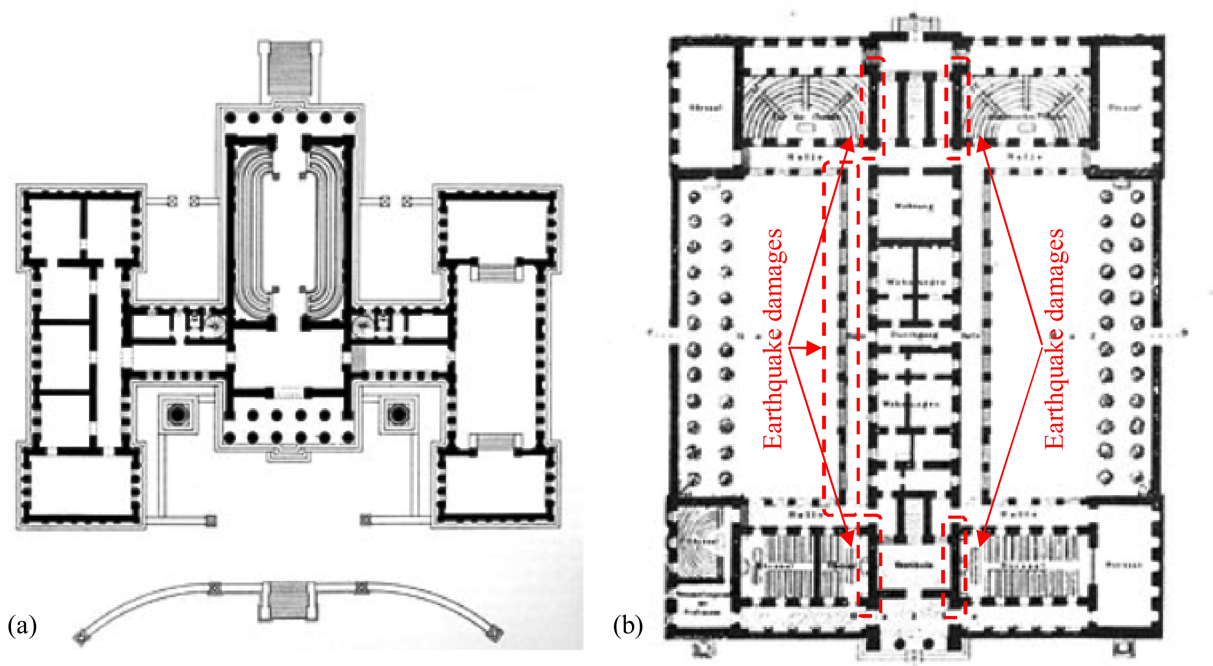


Figure 2: Plans of neoclassical monumental buildings (Athens [3]) and notation of locations of developed damages during the 1999 Athens earthquake: (a) the Academy of Athens (1887), (b) the University of Athens Central Building (1864).

In the present study the torsional response excited by the plan shape morphology of monumental and historical URM buildings is investigated. This is achieved via time-history dynamic analyses of three-dimensional finite element models of a neoclassical building with asymmetric plan shape located in Thessaloniki, Greece. Emphasis is given into investigating the effect of horizontal diaphragm stiffness provided by the architectural forms of that era to the overall torsional response of this class of buildings. This building is then assessed utilizing the latest version of the rapid seismic assessment procedure for URM building, which was recently introduced by the authors [4 - 6] enabling the evaluation of the accuracy of the rapid procedures when torsional behavior is prevalent.

2 RAPID SEISMIC ASSESSMENT PROCEDURE OF URM BUILDINGS

To facilitate seismic assessment of historical and heritage URM buildings a rapid procedure has been recently introduced by the authors [4 - 6], which produces results of equivalent accuracy to detailed time-history dynamic analysis based assessment procedures while requiring significantly less effort and shorter computational time. This procedure consists of the following two steps:

Step 1: Determination of envelope of the developed deformations along the examined building The first step of the introduced rapid assessment procedure is the determination of the envelope of the developed horizontal deformations along the structural system of the examined building during its seismic excitation. To do so, a three-dimensional finite element model of the examined building is subjected to a notional gravitational field that acts horizontally in each of the two principal plan directions of the building and then is analyzed statically. 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 at contact points. To capture both in-plane and out-of-plane actions, basic element unit in the discretization of the piers and spandrels is a typical thick shell with elastic properties. Each static analysis provides a deflected shape of the examined building analogous to the one corresponding at the instant of the building's maximum seismic response when subjected to a seismic excitation capable of mobilizing the biggest possible fraction of the building's total mass in the corresponding plan direction; note that the percentage of the total mass of a vibrating structure that can be activated during a seismic excitation depends of the correlation between the structure's dynamic characteristics and the earthquake's frequency content and height-wise mass distribution - in the case of URM historic buildings it ranges up to 50% [4]. The envelope of developed deformations along the structural system of the examined building corresponding to the spectral demand imposed at the building's site by the associated seismic code is then calculated by amplifying the deflected shape obtained from the static analysis in the i-th plan direction (i = X or Y) with the amplification factor f_i:

$$f_i = S_{Dd,i}(T_1) / U_{roof,i} \tag{1}$$

where, $S_{Dd}(T_1)$ is the spectral relative displacement demand corresponding to the building's fundamental period of vibration, T_1, and $U_{roof,i}$ is the average horizontal translation at the roof level of the building in the corresponding plan direction i, as calculated from static analysis. The same scaling (through f_i) maybe applied in the estimated member forces from the static analyses in order to obtain a rough estimate of peak member stresses/forces during the ground excitation. $S_{Dd}(T_1)$ can be evaluated according to EC8-1 [7], as per:

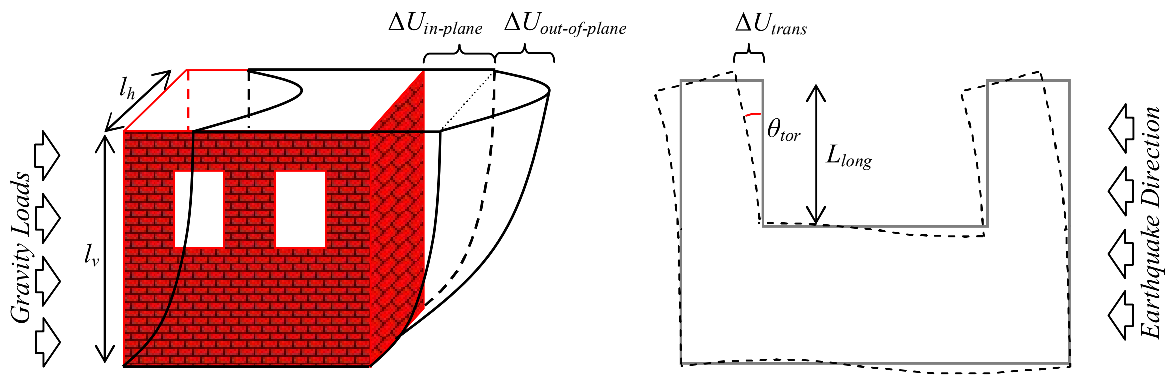
$$S_{Dd}(T_1) = S_d(T_1) \cdot \left[\frac{T_1}{2 \cdot \pi}\right]^2 \quad ; \quad S_d(T_1) = \gamma_I \cdot a_{gR} \cdot S \cdot 2.5 \tag{2}$$

where, γ_I is the importance factor of the building, a_{gR} is the reference peak ground acceleration on type A ground and S is the soil factor; $S_d(T_1)$ in Eq. 2 is the total spectral acceleration for a SDOF system with a period of T_1. The building's fundamental period of vibration, T_1, is approximated as (EC8-1 [7]):

$$T_1 = 0.050 \cdot H^{3/4} \tag{3}$$

where, H is the total building height, in m, measured from the level of foundation or the level of rigid basement.

Step 2: Determination of local seismic demand and application of acceptance critiria Bearing capacity in URM historical structures can be best identified by the amount of deformation occurring in the various components of the structure. Using deformation demand for the purpose of seismic assessment is more meaningful than force demand estimation as, based on equal displacement rule, elastic displacement demands are close to the inelastic ones, whereas analogous similarities do not exist between the forces calculated from inelastic and elastic analyses. In the framework of the introduced rapid seismic assessment procedure, local seismic demand is specified in terms of relative drift ratios referring to the in-plane relative deviation of the piers' and walls' ends from vertical, θ_{in}, and to URM facades deviating from the horizontal initial orientation out-of-plane, θ_{out}. The relative drift ratio in plane, θ_{in}, is defined as the horizontal relative displacement (i.e. the difference) that occurs between two points along the building height, (i.e. ends of a pier or a wall), divided by their distance. θ_{out} is defined as the outwards relative deflection between successive points in the building plan. Meaningful indices include the relative displacement between the midspan and corners of walls oriented normal to the direction of seismic action, depicting the out-of-plane action; also the relative displacement at the end points of wings relative to the main structure, indicating the tendency for torsional response, θ_{tor}. This may be examined at the floor levels and at the crest of the building. Definitions of θ_{in}, θ_{out} and θ_{tor} are presented in Figure 3. Thus, the fraction of the θ_{in} value which is owing to the torsional response is equal to $\theta_{tor} \cdot L/H$, where L is the length of wing from the point of support and H the height of the structure at the point considered.



$$\theta_{in} = \frac{\Delta U_{in-plane}}{l_v} \qquad \theta_{out} = \frac{2 \cdot \Delta U_{out-of-plane}}{l_h} \qquad \theta_{tor} = \frac{\Delta U_{trans}}{L_{long}}$$

Figure 3: Definitions of θ_{in}, θ_{out} and θ_{tor} relative drift ratios.

The values of θ_{in}, θ_{out} and θ_{tor} calculated above are then compared to the corresponding performance levels imposed by the seismic code applicable at the site. Cracking rotations (drift ratios) in masonry elements, θ_y, are in the order of 0.15% for in-plane and 0.20% for out-of-plane wall deformation (corresponding to the *Operational/Immediate Occupancy* limit state). The Hellenic code for assessment and structural interventions in masonry buildings (KADET 2014, [8]) classifies as *Significant but Repairable Damage* performance level values of relative drift ratios that vary between θ_y and $3/4 \cdot \theta_u$ (i.e. ductilities $\mu < 2.5$), whereas for higher values of θ_{in} and θ_{out} ranging up to the limit of $4/3 \cdot \theta_u$ damages occurring to the structural elements are classified as having attained the *Life-Safe/No collapse* performance level. The value of ultimate drift ratio, θ_u, according to the same code depends on the kind of deformation imposed on the examined wall. For in-plane deformations, θ_u equals:

$$\theta_u = 0.008 / L \tag{4}$$

whereas, for out-of-plane deformations:

$$\theta_u = \min \{\theta_{u,1}, \theta_{u,2}\} \;\; ; \;\; \theta_{u,1} = 0.003 \cdot H_o / t \, , \;\; \theta_{u,2} = \theta_{R,u} \cdot (1 - M_y / M_{Rd}) \tag{5}$$

In Equations 4 and 5, L is the length of the wall (measured in the horizontal direction), t is its width, $\theta_{R,u} = t/H_o$, $M_y = f_{wt} \cdot \left(B \cdot t^2\right)/6$, f_{wt} is the tensile strength of the wall, B is the dimension of the wall pier or spandrel that defines the width of the section that is bending, H_o is the distance from the point about which rotation of the wall occurs, $M_{Rd} = F_{Rd} \cdot H_o / 2$, $F_{Rd} = S_d(T_1) \cdot t \cdot \gamma \cdot A_{L,w}$ and $A_{L,w} = L \cdot H_w$, and H_w is the total wall height.

3 DEMONSTRATION OF THE SEISMIC RESPONSE OF MONUMENTAL URM BUILDINGS

To investigate the seismic response of torsionally sensitive monumental and historical buildings a series of time-history dynamic analyses have been performed to different types of finite element models of a monumental building located in Thessaloniki, Greece. The building, known as "Papafeion", was constructed in 1902 according to the designs of the architect Xenofon Paionidis and the sponsorship of Ioannis Papafis in order to function as an orphanage for the children of the Hellenic community of Thessaloniki, which at the time was under Ottoman rule. Since 1903 the building is being continuously in operation, functioning mainly as an orphanage or a hospital, with the most recent function to be a boarding school for orphan boys under the management of the Orthodox Church.

3.1 Description of the examined building

The Papafeio building is a three storey neoclassical building, consisting of three building wings which in plan forms the shape of the capital letter ***E*** (Fig. 4), codifying the initial letter of the words *Ελλάς*, *Ελευθερία* and *Ειρήνη*, the Greek words for *Greece*, *Freedom* and *Peace* which at the time were the object of national struggle. The entire building complex is enclosed within a rectangle of external dimensions of 81.50 by 55.44 m, whereas the building height measured from the floor of the first storey to the top of the third storey (the height of the timber roof is omitted) is 16.23 m (foundation height: 1.20 m, 1st storey: 3.60 m, 2nd storey: 5.65 m, 3rd storey 5.78 m) except of the central part of the central wing, where the corresponding height is 19.03 m due to an elevation of the roof of the building's ceremonial hall (Fig. 4(e)). The walls of the first storey are made of stone, having a thickness equal to 0.80 m at the perimeter of the building and 0.70 m at the inner plan. Walls of the second storey were built of solid bricks, whereas third storey walls are made of voided bricks. Perimeter walls are

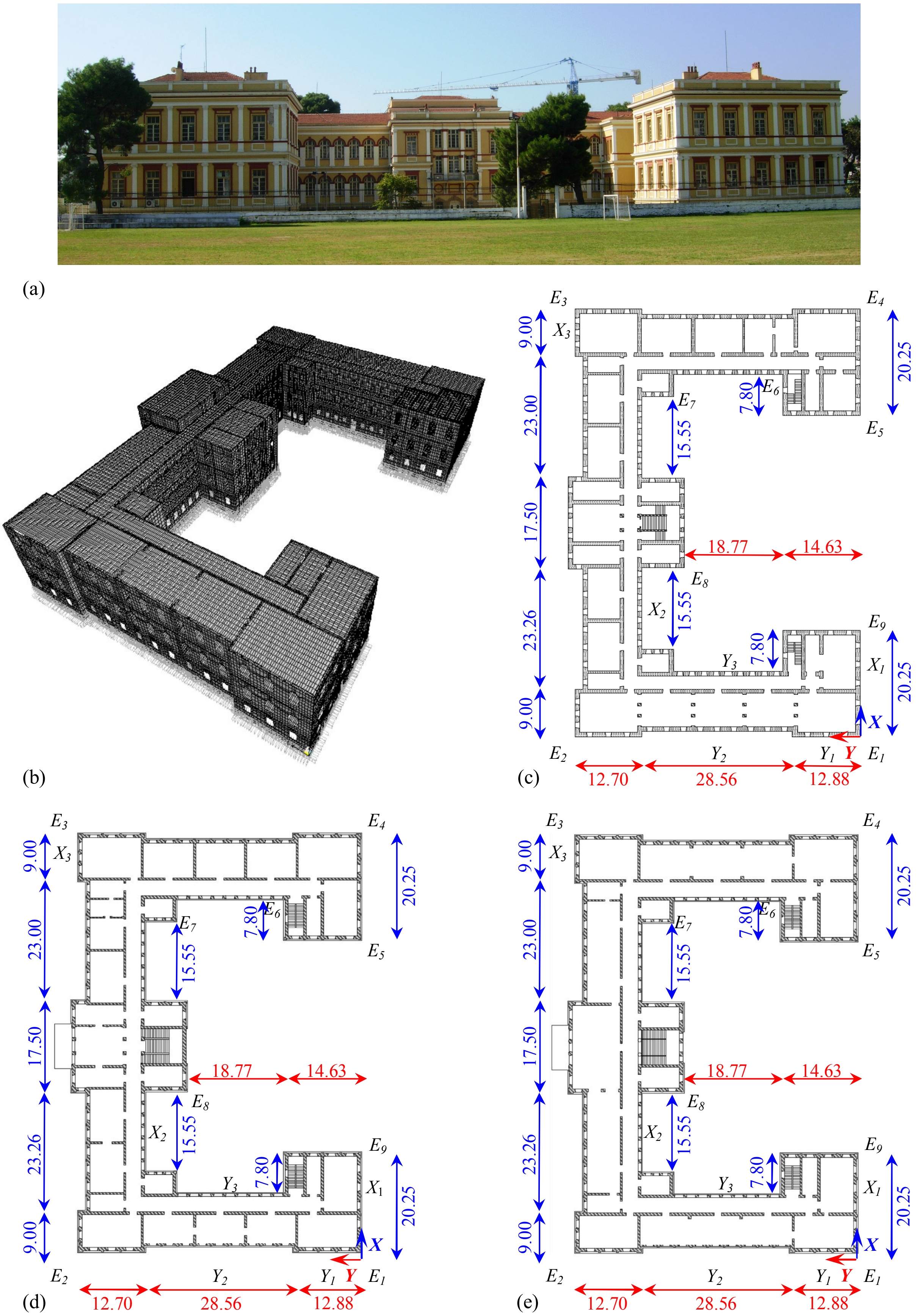


Figure 4: The Papafeio building: (a) North view, (b) 3D finite element model, (c - e) plan views of the 1st, the 2nd and the 3rd storey.

0.55 m in width, whereas internal walls are 0.40 m thick at both storeys. Floors of the first and the second storey are made of double T iron beams having a 100 x 180 mm and a 80 x 180 mm cross section at each storey, respectively, spacing along the small sides of the rooms and the corridors at 0.65 m at the first and 0.67 m at the second storey. The roof of the third storey is also made of double T iron beams, oriented similarly to the iron beams of the 1st and the 2nd storey floors, having a cross section of 80 x 200 mm and spacing at 0.85 m. At all floors and the roof brick–arches spanning in the transverse direction between successive iron beams are encased between the upper and lower flanges of the double T beams. The total thickness of the building's horizontal structural elements (including the finishing) at the location of the iron beams is 0.34 and 0.30 m at first and second storey floor and 0.20 m at third storey roof, whereas the thickness at highest point of the arches is about 0.10 m smaller. The last storey is covered by a roof made of timber, trusses spanning along the longitudinal dimensions of the building's wings, yet this roof is not part of the structural system of the building.

3.2 Modeling and analysis of the examined building

Investigation of the seismic response of the Papafeio building was achieved via simulation of the building as a three-dimensional finite model (Fig. 4(b)) by subjecting the model to time-history dynamic analyses [9]. In the building model walls were idealized using four-node shell elements, capable of supporting in plane and out of plane forces and moments (6 d.o.f. per node), whereas floors were simulated using linear elements for the iron beams and shell elements to represent the brick arches spanning between steel beams. A total number of more than 91000 area and 16000 frame elements were utilized, resulting in the formation of a finite element model of an input size of 180 MB. 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: (a) for stone f_k = 5.5 MPa, (b) for solid bricks f_k = 4.0 MPa and (c) for voided bricks f_k = 1.5 MPa. In the case of frame elements (iron beams), the modulus of elasticity was taken equal to 150 GPa. In all cases self weight of the building was calculated according to the material density; for stone 25 kN/m^3, for solid bricks 18 kN/m^3 and for voided bricks 14 kN/m^3. A roof weight equal to 1.5 kN/m^2 was assumed, uniformly distributed along the area elements of the third storey roof. Service loads were considered equal to 2.50 kN/m^2 for the roof and 3.50 kN/m^2 for the floors of the building. Masses considered in the dynamic analyses were automatically calculated by the program, by multiplying each element (shell or linear) volume by their respective density.

In order to investigate the influence of floor and roof stiffness within their plane to the overall seismic response of torsionally vulnerable URM buildings, two different versions of the produced building model were considered in the dynamic analyses. Note that according to the construction practices used from Renaissance until the 20th Century all over Europe, different types of floors and roofs were used in this category of buildings, providing different types of stiffness within their plane. In the first version of the considered building, the stiffness of the horizontal structural elements (floors and roof) was omitted, accounting for a building response which is controlled only by the response of the building's walls. The second version of the examined building model had full diaphragm action established at its floors and roof levels, accounting for the theoretical case where the horizontal structural elements of the same level at height in all building wings respond in plan as if they belong in the same undeformed horizontal plane. The actual seismic response of the building lies somewhere within the response of the two examined versions of the building model, which form the envelope of seismic response that can be developed from it.

Each of the two versions of the building model were subjected to time-history dynamic analyses for the case of the 1978 Thessaloniki earthquake record, selected from the ITSAK earthquake database [10]. From among the three components recorded for the specific 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*). This component was applied separately in each of the two principal directions in plan of the examined building in each case of dynamic analysis. The absolute acceleration and relative displacement response spectra of the acceleration record used in the dynamic analyses, S_a and S_d respectively, as those were calculated considering a viscous damping equal to ξ = 5%, are presented in Figure 5 in comparison to the corresponding spectra imposed by EC8-1 for seismic assessment of URM historical buildings at the building's site (γ_I = 1.3, S = 1.20).

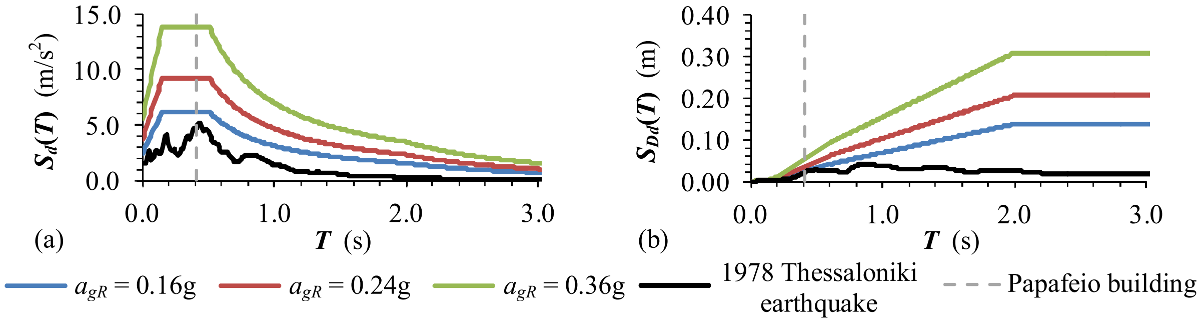


Figure 5: Response spectra of the earthquake record used in the analyses (calculated for 5% damping) and imposed by EC8-1 for assessment: (a) absolute acceleration spectra, (b) relative displacement spectra.

3.3 Examination of the results of time-history analyses

To understand the seismic response of torsionally sensitive, monumental, URM buildings, first the time-history of the horizontal displacements at the roof of the third storey of the Papafeio building is investigated. Figures 6 and 8 depict the time-history of the horizontal displacements parallel to both the X and Y plan directions, U_X and U_Y, at selected locations of the building's plan (for the location of each control point see Fig. 4(e)). U_i (i = X or Y) at those two figures derived from the dynamic analyses of the building model with no floor and roof stiffness, subjected to seismic excitation parallel to X and Y directions, respectively. Each plot depicts a time interval of 1.5s before and after the instant of maximization of the horizontal displacement of the control point in the same direction with the earthquake excitation. Note that time-histories of U_i are presented as a ratio of the maximum value of the time-history of the maximum horizontal displacement developed at the examined locations, estimated from, $U_{max}= \sqrt{(U_X^2 + U_Y^2)}$, so as to investigate the degree of influence of torsional phenomena owing to the building's plan shape. Similar graphs corresponding to the dynamic analyses of the building model with established diaphragm action at floors and roof levels are presented in Figures 7 and 9.

As depicted in Figs. 6 - 9, the degree of provided stiffness within the horizontal structural elements at this category of buildings plays a crucial role at the building's overall seismic response. In the case where floor stiffness is omitted (Figs. 6 and 8), each of the control points of the building displaces not only parallel to the earthquake excitation but also in the orthogonal direction by a significant amount, which may exceed up to 50% of the magnitude of its principal displacement. This applies at the edges of the building and at walls whose plane is parallel to the direction of the earthquake excitation, whereas walls oriented orthogonal to the ground excitation displace mainly in their out-of-plane direction (Figure 6, Facade Y_3). On the contrary, when full diaphragm action is achieved within building's floors and roof, all control

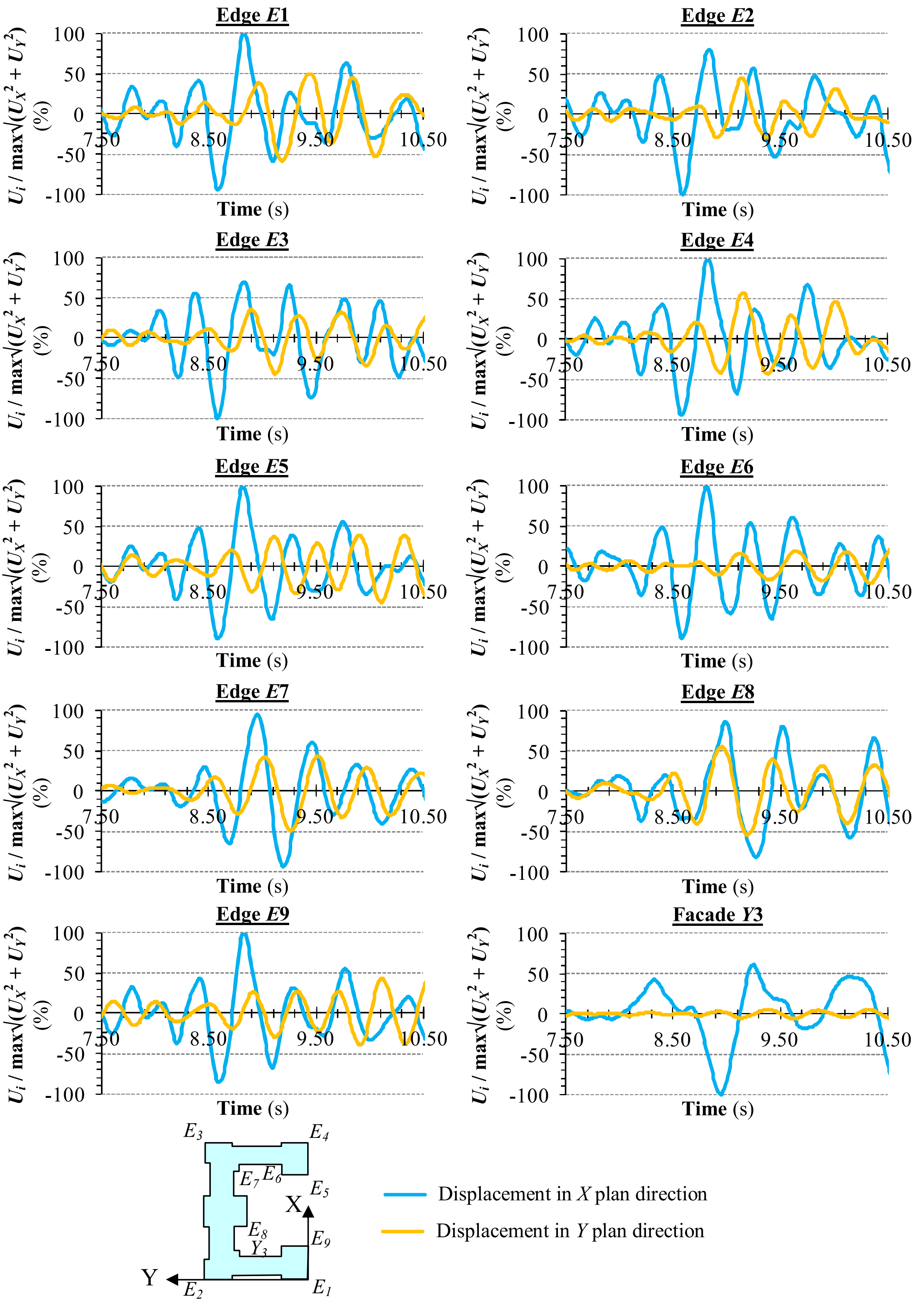


Figure 6: Time-histories of the developed horizontal displacements at roof level of the Papafeio building when zero stiffness is considered within its floors and roof levels - Earthquake action parallel to *X* plan direction.

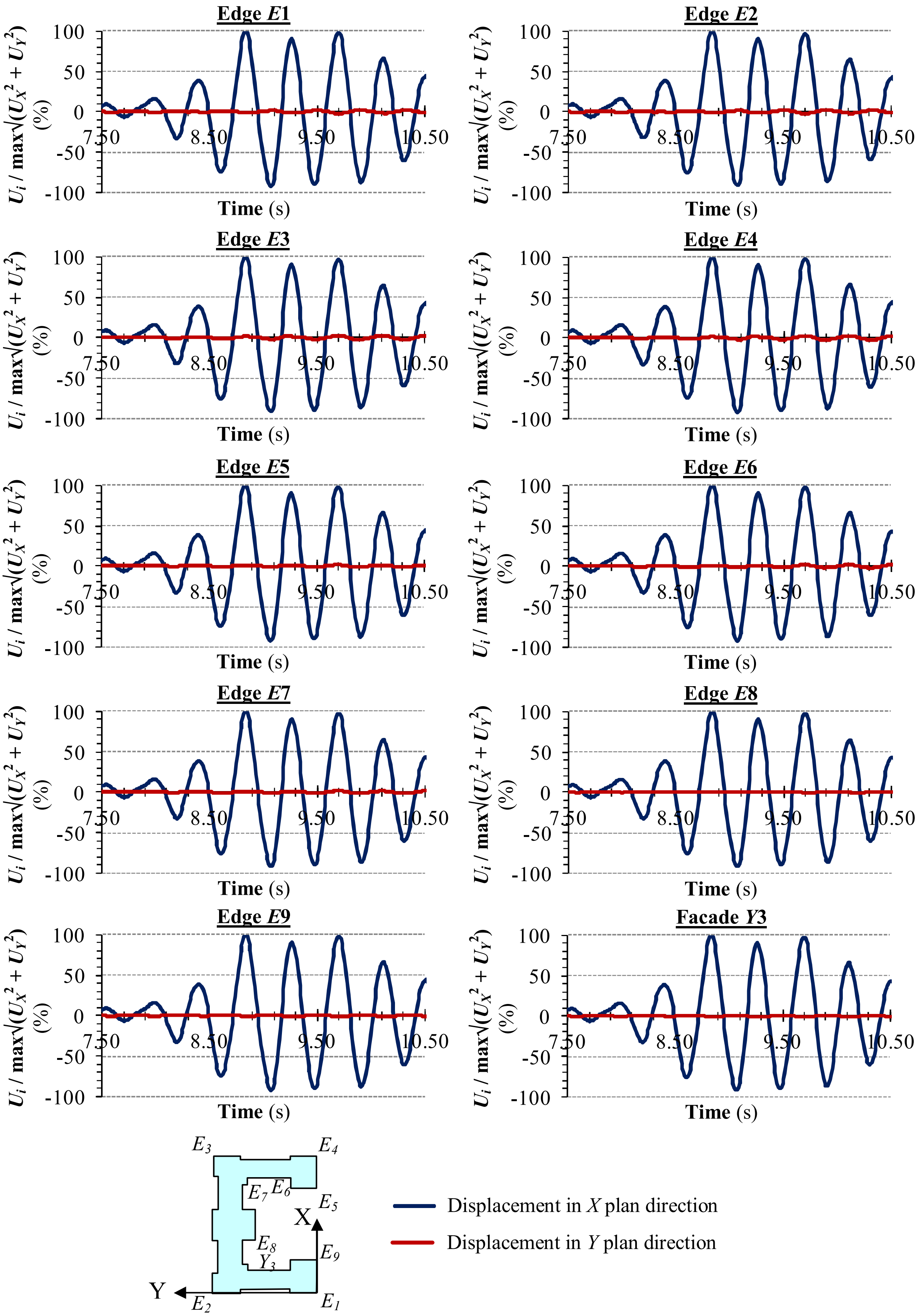


Figure 7: Time-histories of the developed horizontal displacements at roof level of the Papafeio building when full diaphragm action is considered within its floors and roof levels - Earthquake action in X plan direction.

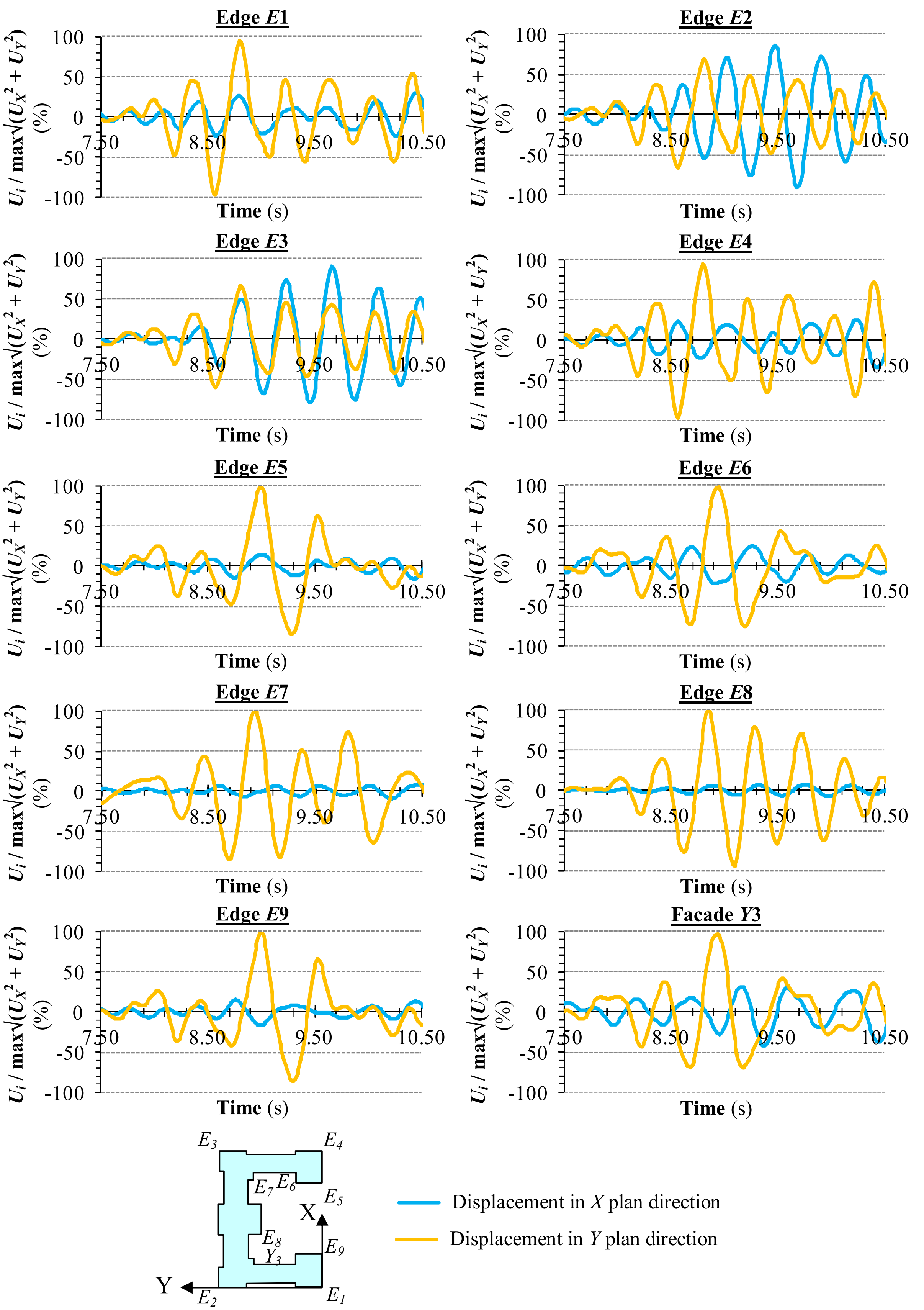


Figure 8: Time-histories of the developed horizontal displacements at roof level of the Papafeio building when diaphragm action is neglected in its floors and roof levels - Earthquake action parallel to *Y* plan direction.

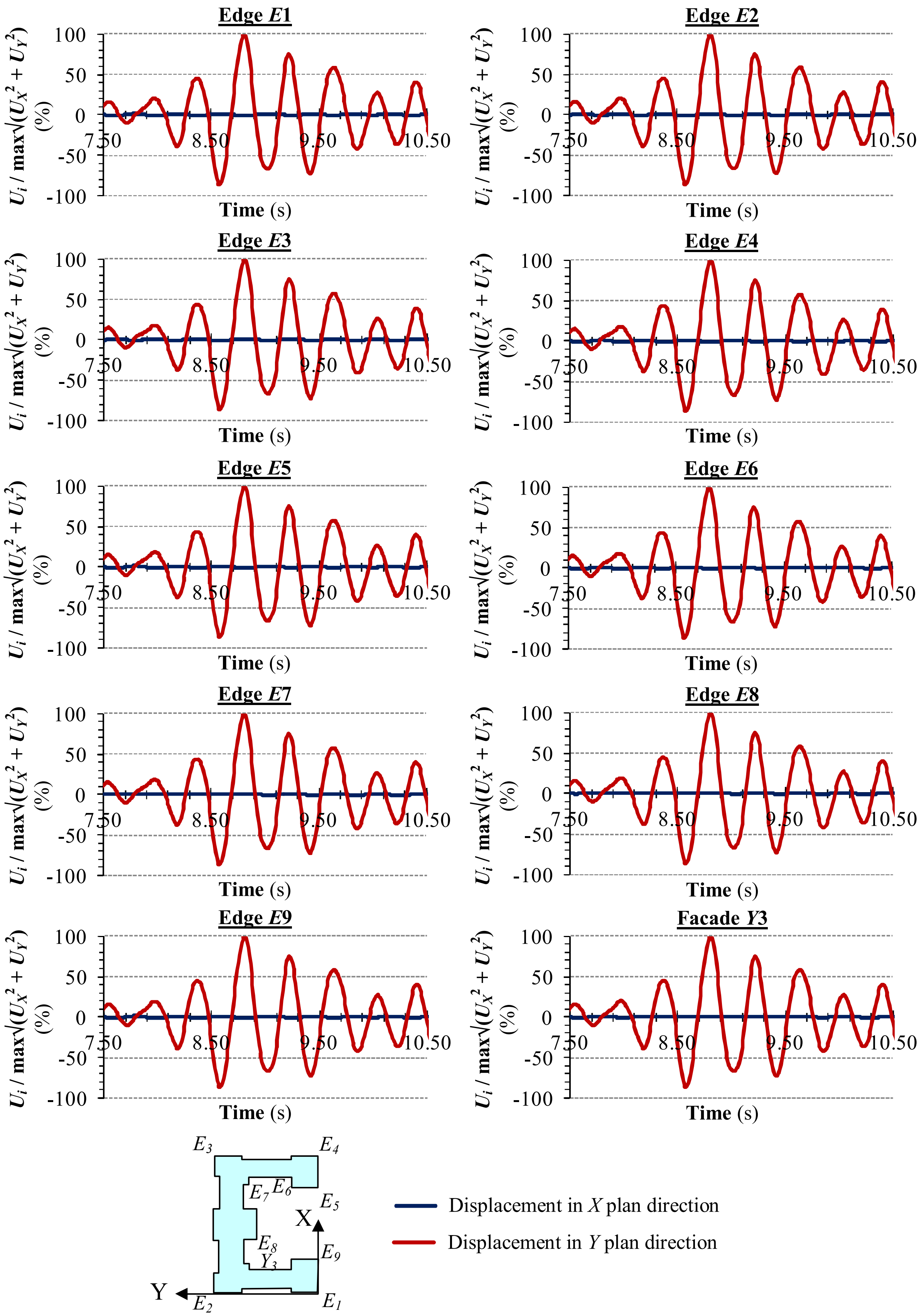


Figure 9: Time-histories of the developed horizontal displacements at roof level of the Papafeio building when full diaphragm action is considered within its floors and roof levels - Earthquake action in *Y* plan direction.

points displace mainly parallel to the direction of the earthquake action, regardless of orientation of the walls to whom they belong with respect to the earthquake's direction (Figs. 7 and 9). Also note that in the case of zero stiffness within the plane of the floors and roof the waveform of developed displacements varies among the different control points of the building, indicating a differential movement within locations of the same horizontal level of the building.

To further understand the influence of torsional effects developing at monumental URM buildings in association to the degree of achievable diaphragm action at their floors and roof to the structures' overall seismic response, Figures 10 and 11 depict the lateral displacements occurring height-wise at control locations at the instant of maximization of the horizontal roof displacement. The two figures present the deflected shapes of the same ten control locations examined at Figs. 6 - 9 at X and Y plan directions respectively, when the earthquake acts in the corresponding direction. Lines in light blue and orange in Fig. 10 and 11 respectively refer to the responses obtained from the building model with no-diaphragm action within its floors and roof, whereas lines in dark blue and red in the same figures correspond to the seismic response of the building model with full diaphragm action. Continuous lines plot the deformed shape of the control locations at the instant of their maximum roof displacement. Also plotted with dashed lines are the corresponding deformed shapes obtained after performing the proposed static analysis procedure for rapid determination of the building's maximum seismic response. These shapes have been amplified by the f_i factors (Eq. 1) so as the maximum horizontal displacement of the respective control location obtained from time-history dynamic analysis be equal with the horizontal displacement occurring from the proposed static analysis procedure at the same building height (this location in height corresponds to the roof level at all control locations except of the case of facade Y_3 when full diaphragm action is considered, which is at 91% of the total building height).

As illustrated in both figures, the horizontal deformations occurring at the building's control locations when the earthquake excitation acts in the same plan direction and full diaphragm action at the floors and roof level is considered are greater as compared to the corresponding displacements obtained in the absence of a diaphragm action. This difference is greater when the earthquake acts in X plan direction (perpendicular to the building's axis of symmetry) ranging up to the value of 3. Also depicted at Figs. 10 and 11 is the very good correlation between the deformed shapes of the control locations obtained by the two analysis procedures, whereas even when there is some discrepancy in the results, the interstorey drift calculated from the static analysis procedure results is greater than the corresponding one derived from the results of time-history dynamic analysis.

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. Yet, application of the introduced rapid analysis procedure requires significantly smaller computational time and means. Among the examined analyses cases, the volume of produced output files in the case of time–history dynamic analyses is in the range of 57 GB, requiring an execution time of about 18 h, whereas execution of the rapid analysis procedure in the same 3-D finite element models required less than two minutes for each plan direction of the examined building models and the volume of the produced output files were is the range of 1 GB (note that the input file of the Papafeio building alone has a volume of 0.18 GB). Thus, the proposed static analysis procedure is much simpler to process and handle than the time-history dynamic analysis procedure, especially in cases of massive structures, such as the monumental URM buildings; this renders the rapid approach a useful tool in the hands of practitioners in the field of seismic assessment.

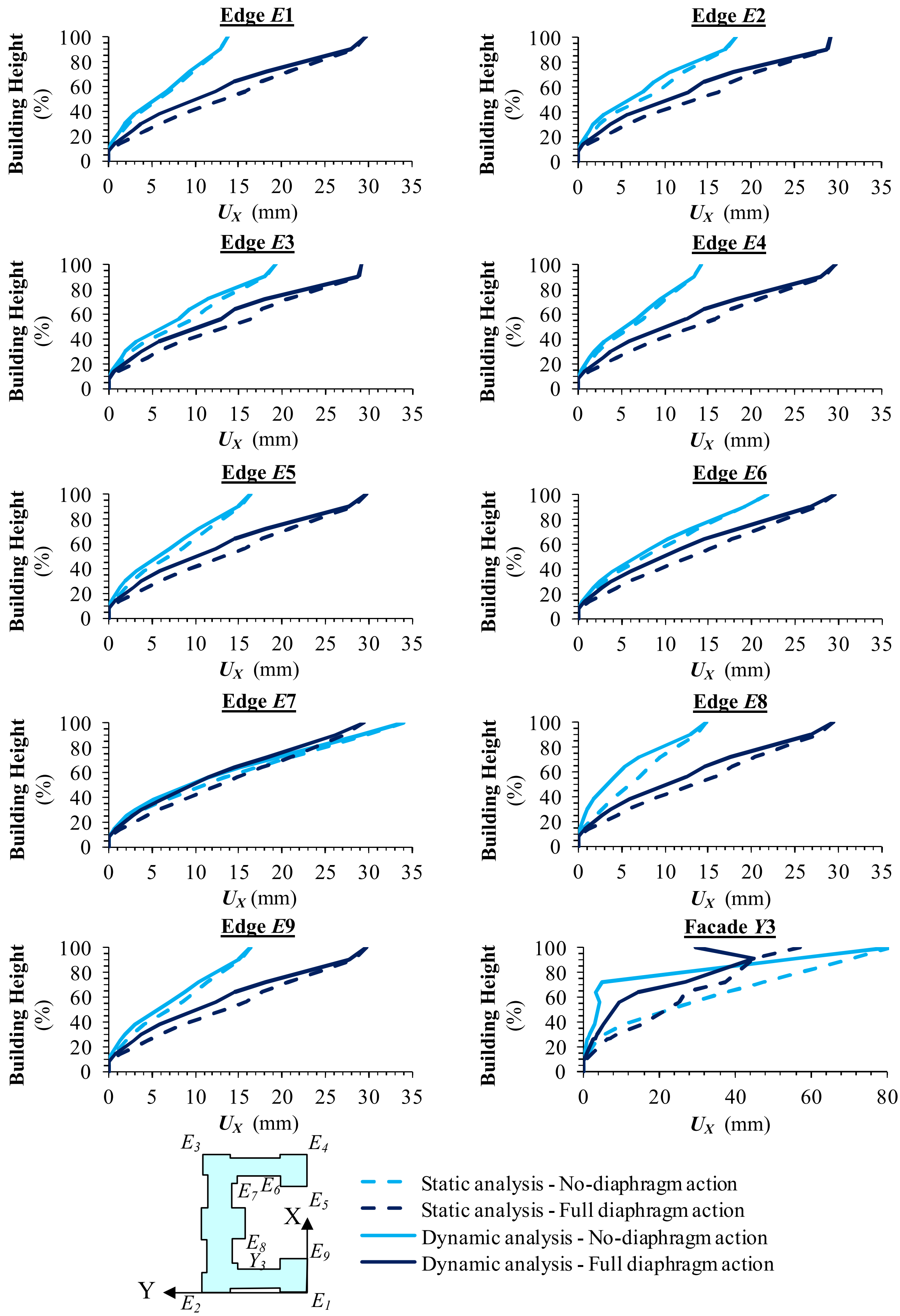


Figure 10: Lateral displacements at control locations of the Papafeio building (for identification of location see Fig. 4) in the X plan direction at the instant of the maximum roof displacement, when the building is subjected to earthquake excitation in the same direction.

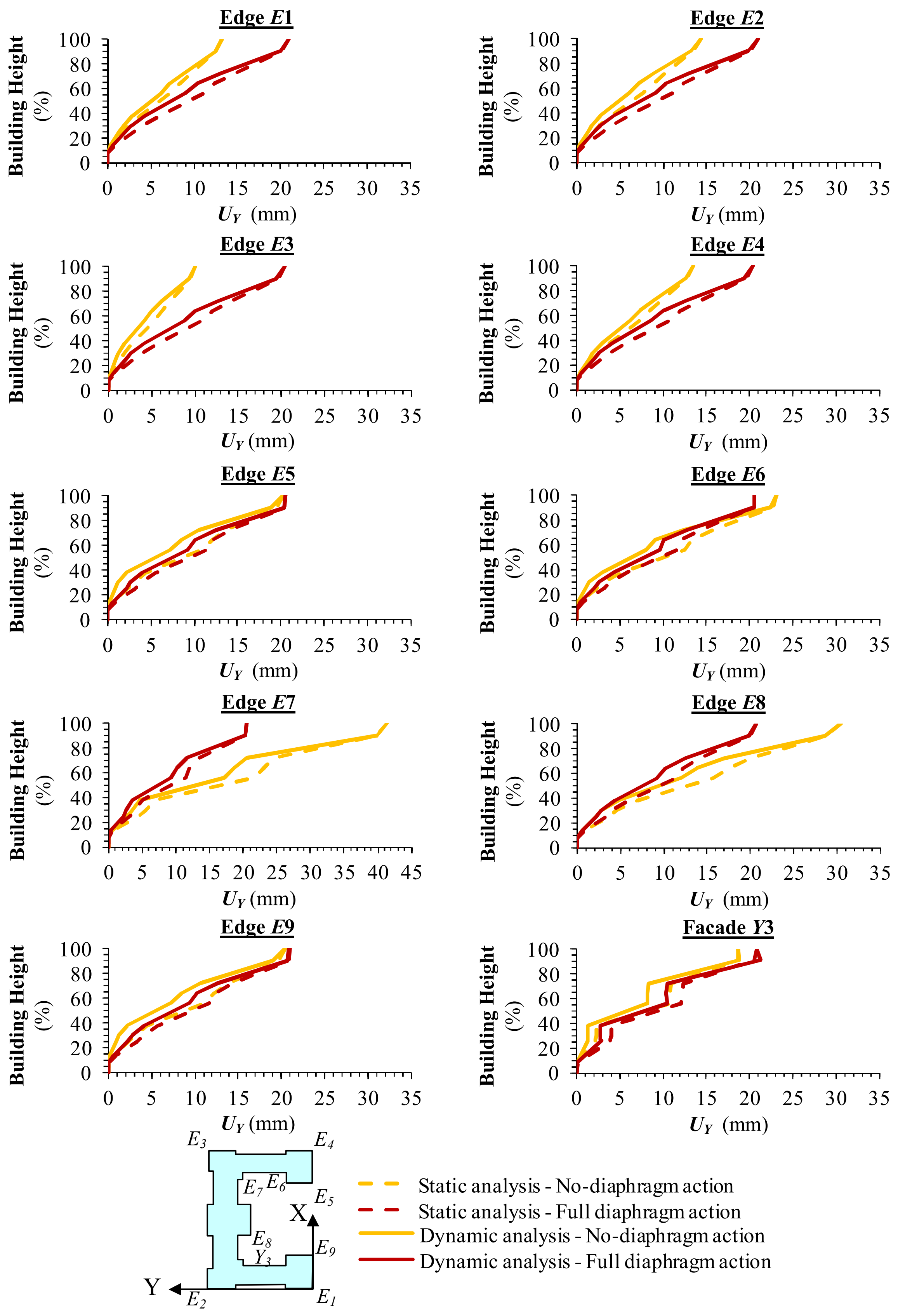


Figure 11: Lateral displacements at control locations of the Papafeio building (for identification of location see Fig. 4) in the *Y* plan direction at the instant of the maximum roof displacement, when the building is subjected to earthquake excitation in the same direction.

4 SEISMIC ASSESSMENT OF THE PAPAFEIO BUILDING

As demonstrated from the seismic response of the two versions of the Papafeio building model, the different degree of diaphragm action provided at floor levels of historic URM buildings due to the vast variety of construction practices used at horizontal structural elements in association with the influence of torsion due to the shape in plan of many of this type of buildings can lead to very different responses of their structural system. Therefore, seismic assessment of monumental and historical URM building is essential, especially in countries that suffer from high seismicity, so as to identify potential locations of damage at the case of the earthquake scenario applied in the greater region and to select the appropriate retrofit strategy that will secure the integrity of the structure as well as the preservation of its unique historic and architectural characteristics. In this framework, the rapid seismic assessment procedure introduced by the authors provides accurate results in a relatively straightforward manner. In this section an example application of the proposed procedure to the two versions of the Papafeio building model is presented for the case of the EC8-1 earthquake scenario applied at the building's site (a_{gR} = 0.16 g, Fig. 5) and the results are compared to the building's actual seismic response during the 1978 Thessaloniki earthquake (Fig. 5).

Application of the rapid seismic assessment procedure first requires the determination of the envelope of the structure's horizontal displacements caused by the earthquake scenario imposed at the building's site. Considering the height of the Papafeio building, measured from its foundations to the roof of the third storey as H = 16.23 m the building's fundamental period of vibration in both of its principal plan directions can be estimated according to Eq. 3 as T_1 = 0.40 s. According to EC8-1 (Eq. 2) the value of spectral absolute acceleration used for the building's seismic assessment is $S_d(T_1) = 1.3 \cdot 1.57 \cdot 1.20 \cdot 2.50 = 6.121$ m/s^2 (intersection of blue and gray lines, Fig. 5(a)), whereas the target roof displacement valid for both the building's principal plan directions is $S_{Dd}(T_1) = 6.121 \cdot (0.40 /(2 \cdot \pi))^2 = 24.81$ mm (Fig. 5(b)). The average horizontal displacement of the third storey roof in the X plan direction, as this was derived from the proposed static analysis procedure, is 21.88 and 19.92 mm when diaphragms were neglected and when full diaphragm action at floor levers was considered, respectively (i.e., practically the same). The corresponding values in the Y plan directions are 5.80 and 4.79 mm. Therefore, calculation of the developed displacements or stresses for conducting seismic assessment can be obtained by multiplying the corresponding parameter from the static analysis with the amplification factors (Eq. 1) f_x = 24.81 / 21.88 = 1.13 and f_y = 24.81 / 19.92 = 1.25 when floor stiffness is neglected and with f_x = 24.81 / 5.80 = 4.28 and f_y = 24.81 / 4.79 = 5.18 in the case of full diaphragm action at the building's floors. Consider for example the building's external wall Y_1 (Fig. 4(c-e)), whose length is L = 12.88 m. In the case of the building model with zero diaphragm action at floors and roof levels the horizontal displacements U_X at midspan (out-of-plane wall response) resulting from the proposed static analysis procedure in the X plan direction at roof levels of the 3rd, the 2nd and the 1st storey are 38.418, 22.876 and 4.626 mm, whereas the corresponding displacements at the northern end of the wall (Y = 0 m) are 6.245, 3.814 and 1.036 mm and at the wall's southern end (Y = 12.88 m) are 9.830, 6.031 and 2.955 mm. Therefore, the out-of-plane deflection of the Y_1 wall at the instant its maximum seismic response, θ_{out}, calculated according to Fig. 3 and the amplification factor f_x = 1.13, will be 0.58%, 0.34% and 0.05% at roof levels of the 3rd, the 2nd and the 1st storey, respectively. Note that when full diaphragm action is considered at floors and roof levels θ_{out} = 0, as all points of the same level move as if they belong in a perfectly undeformed plane. In the case of in-plane wall seismic response, the U_Y horizontal displacements at the roofs of the 3rd, the 2nd and the 1st storey and the floor of the 1st storey of wall Y_1 resulting from the proposed static analysis procedure in the Y plan direction are [4.172, 2.521, 0.708, 0.095] mm when no-diaphragm action is considered and [4.818, 2.838, 0.924, 0.116] mm in the case of full diaphragm action at floors and roof. With the corresponding values of f_y and the storey heights (see §3.1) the maximum value of in-plane relative drift ratios, θ_{in}, of the 3rd, the 2nd and the 1st storey of the wall Y_1 at the instant of their maximum seismic response will be [0.04, 0.04, 0.02] % and [0.18, 0.18, 0.12] % in the case of negligible and full diaphragm action, respectively.

The rapid seismic assessment procedure leads to the evaluation of performance of the Papafeio building for the EC8-1 earthquake scenario from the local seismic deformation capacities, as calculated from Eqs. 4 and 5. Table 1 presents the results of the seismic assessment procedure in terms of θ_{in} and θ_{out} at selected locations of the external walls denoted at Fig. 4(c-e) of the two versions of the building model (note that in the case of the building model with complete diaphragm action at its floors and roof θ_{out} = 0). The relative drift ratios along the height of the building edges that belong to the walls presented at Table 1 are equal to the corresponding values of θ_{in} (for example, the relative drift ratios along the height of the edge E_1 in the X plan direction equal to θ_{in} of the wall X_1, whereas those in the Y plan direction equal to θ_{in} of the wall Y_1). Furthermore, Table 2 presents the values of θ_{tor} which are expected at selected locations of the exterior of the building when the earthquake acts perpendicular to the examined locations, as the result of the torsional phenomena developing at the floors and the roof of the building due to its plan shape. Note that in the case of complete diaphragm action at floors and roof of the building the values of θ_{tor} are systematically lower than the corresponding ones when no diaphragm action is considered, by as much as 200 times. Exception to this rule is the response of the southern façade of the Papafeio building (the $E_2 – E_3$ façade) where in the case of complete diaphragm action the values of θ_{tor} at the floors and roof levels are greater by 8 to 73 times as compared to the corresponding values when no diaphragm action is considered.

		No Diaphragm Action		Complete Diaphragm Action	
Location	Storey	θ_{in} (%)	θ_{out} (%)	θ_{in} (%)	θ_{out} (%)
X_1	3rd	0.05	0.52	0.21	0
	2nd	0.05	0.32	0.21	0
	1st	0.03	0.06	0.15	0
X_2	3rd	0.08	1.10	0.21	0
	2nd	0.09	0.48	0.22	0
	1st	0.04	0.08	0.15	0
X_3	3rd	0.08	0.26	0.21	0
	2nd	0.08	0.16	0.22	0
	1st	0.04	0.03	0.15	0
Y_1	3rd	0.04	0.58	0.18	0
	2nd	0.04	0.34	0.18	0
	1st	0.02	0.05	0.12	0
Y_2	3rd	0.04	0.30	0.16	0
	2nd	0.04	0.25	0.18	0
	1st	0.02	0.01	0.09	0
Y_3	3rd	0.08	1.75	0.33	0
	2nd	0.09	0.81	0.37	0
	1st	0.03	0.11	0.14	0

Table 1: Values of θ_{in} and θ_{out} at selected locations of the external walls of the Papafeio building. Values in red correspond to development of moderate damages.

In the case of in-plane response of the building's walls, the value of θ_u derived from Eq. 4 according to the walls' geometric characteristics equals to 0.80%. Therefore, the onset for the *Repairable Damage* and *No-Collapse* performance levels equal to 0.60% and 1.07%, respectively. When out-of-plane response is considered, given the geometric characteristics of the walls at the selected locations, the onset for the *Repairable Damage* performance level ranges between 1.16% and 3.44%, whereas the corresponding value range for the *Life-Safe/No-Collapse* performance level is between 2.05% and 6.12%. Considering the values of θ_{in} and θ_{out} presented at Table 1, moderate damages are expected to occur at the external walls of the Papafeio building in the form of flexural cracking of the walls due to out of plane action. The results of the rapid seismic assessment are confirmed by the actual response of the building during the 1978 Thessaloniki earthquake, whose response spectra approximate those of EC8-1, calculated for a_{gR} = 0.16 g (Fig. 5).

Location	Storey	No Diaphragm Action θ_{tor} (%)	Complete Diaphragm Action θ_{tor} (%)
$E_1 - E_2 = E_3 - E_4$	3rd	0.006164	0.000169
	2nd	0.003268	0.000265
	1st	0.000460	0.000193
$E_2 - E_3$	3rd	0.000048	0.000411
	2nd	0.000042	0.000321
	1st	0.000003	0.000225
$E_6 - E_7$	3rd	0.035967	0.000178
	2nd	0.016499	0.000257
	1st	0.002991	0.000198
$E_1 - E_9 = E_4 - E_5$	3rd	0.030183	0.000427
	2nd	0.017470	0.000321
	1st	0.003281	0.000214

Table 2: Values of θ_{tor} at selected locations of the Papafeio building.

5 CONCLUSIONS

This paper investigates the seismic response of monumental buildings built during the post-renaissance era in many European cities. Buildings of this category often have a unidirectional plan symmetry, whereas in the other principal plan axis asymmetry occasionally triggers torsional phenomena during a seismic excitation, which can lead to important differential displacements along their structural system and therefore a high risk of damage localization in the remotest edges of the structure. Given the historical value of those buildings, which in many cases are inseparably connected with political, economical or cultural national milestones, seismic assessment of them is an essential step towards identifying potential locations of damage and towards selecting the appropriate retrofit strategy that will secure the integrity of those structures as well as the preservation of their unique historic and architectural characteristics.

In this framework, the rapid seismic procedure for unreinforced masonry buildings, which has recently been developed by the authors, has been applied to a monumental neoclassical building of Thessaloniki, Greece and results have been evaluated to assess the extent of anticipated damage. It was shown that the assessment procedure yielded results of equivalent accuracy to detailed time-history dynamic analysis based assessment procedures, as well as to the actual seismic response of the examined building. Yet, the introduced procedure required significantly shorter computational time and effort, and produced significantly smaller volume of output files, which makes it ideal for use in the case of massive structures such as the monumental buildings, where, even when powerful computational means are utilized seismic assessment can be an especially demanding task.

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