COMPDYN 2011
III ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis, V. Plevris (eds.)
Corfu, Greece, 25–28 May 2011

SEISMIC ASSESSMENT OF 19TH CENTURY HERITAGE BUILDING THROUGH SIMULATION

Stylianos J. Pardalopoulos[1] **and Stavroula J. Pantazopoulou**[2]

[1] Demokritus University of Thrace
Dept. of Civil Engineering, 67100, Xanthi, Greece
spardalo@civil.duth.gr, pantaz@civil.duth.gr

Keywords: Assessment, Neoclassical Buildings, Deformed Shape, Modes.

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 affect the process 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. In the present paper, also investigated is the effect of horizontal diaphragm stiffness and connection to the walls; their contribution to dynamic response and appropriate methods of modelling are studied. Conclusions are calibrated based on comparative evaluation of the simulation results (obtained from detailed finite element modelling of a representative historic building located in the seismically active region of Thessaloniki) with the extent of actual damage patterns which have been observed due to strong earthquakes that have occurred during the buildings' lifetime. The paper also presents a relatively simple 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.*

1 INTRODUCTION

Rehabilitation and retrofitting of historical and heritage structures of the 19th and 20th Century is an issue of paramount importance in countries with built cultural heritage that also suffer from high seismicity, such as the countries of the eastern Mediterranean basin. In Greece a significant number of neoclassical buildings were constructed starting immediately after the war of independence and continuing over the last two centuries. These are concentrated mainly in significant urban areas and are considered today an inseparable part of the country's inventory of historical buildings. Over the several decades of their service life, many of those buildings have undergone severe shaking during the strong earthquakes that occasionally affect the urban areas of this highly active seismic region. Yet, even today they remain in good condition, being operational in many cases. Due to those buildings' historical importance 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 neoclassical 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 highlight the special modeling procedures that are needed in the process of simulation of 19th and 20th century neoclassical buildings. This special class of buildings, which is similar to many historical and heritage buildings across Europe, possess special characteristics due to their design and structural configuration, whose oversight during their modeling procedure could lead to serious misinterpretation of their seismic response. Also presented is a simple and rapid analysis procedure, based on the modal characteristics of this class of buildings, which can lead to as dependable estimates as the results of complicated and time-consuming dynamic analysis.

2 OVERVIEW OF THE SPECIAL CHARACTERISTICS OF THE NEOCLASSICAL BUILDINGS DUE TO THEIR DESIGN AND CONSTRUCTION

Seismic assessment of neoclassical buildings constructed in the 19th and 20th Century is a much more demanding procedure than seismic assessment of reinforced concrete structures in that the structural system in masonry buildings is less clearly defined, whereas a profound knowledge of the design principles and the construction practices of that era is required in modeling the individual structural components to dependably reproduce the mechanics of member behavior.

Knowledge of the designing principles of the 19th and 20th Century is essential for interpretation of the dynamic response of this class of buildings. Architects of the era, being influ-

enced by the shape and form of ancient Greek monuments, were designing neoclassical buildings that were characterized by symmetry in plan and in height, loaded with architectural features and elements inspired from archaic forms (for example colonnades, reliefs in gables, and carefully chosen aspect ratios). In the cases of single-unit buildings, the structure was designed in plan so as to have a simple geometrical shape, rectangular or prismatic, whereas multi-unit buildings were designed to have a system-plan organized in the form of an ***H*** 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 (Fig. 1). In every case, the rooms of the building were symmetrically arranged in both sides of a main corridor that was usually spanning from one end of the building to the other. Side views of the building were designed to have windows and doors symmetrically spaced with respect to a vertical axis that was intersecting the plan symmetry axis. Furthermore, the area occupied by openings (i.e. windows and doors) was gradually increased from the ground level to the top of the building (resulting to the corresponding decrease of the area of walls) to encourage a flexural-type behavior for the building in lateral vibration. Thus, buildings of that era are expected to develop favorable dynamic response during an earthquake, such as the absence of torsional effects in plan and a favorable drift distribution along the building height.

HELLENIC PARLIAMENT (1847) – ATHENS

Front View [1]

Plan View

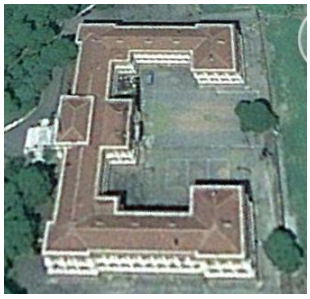
MINISTRY OF MACEDONIA AND THRACE (1891) – THESSALONIKI

Front View [1]

Plan View

PAPAFEIO ORPHANAGE (1903) – THESSALONIKI

Rear View



Plan View

Figure 1: Typical monumental neoclassical buildings built in Greece during the 19th and 20th century.

The construction practice in the 19th and 20th Century was also governed by empirical principles [2 - 6], the consideration of which is essential for proper modeling of the behavioral mechanisms that are developed in neoclassical buildings during an earthquake. Starting from the foundations, different techniques were utilized according to the compressive strength of the foundation soil, in the site where the building was to be constructed. Soft soils were reinforced by driving into the ground in arbitrary manner wooden piles up to 1.30m in length, having diameters that varied from 0.15m to 0.20m, and by filling the space between the piles with gravel. Until the end of the 1860s, grids made by wooden boards of cross sectional dimensions of 80mm x 320mm, closely spaced, were utilized in medium compressed soils in order to uniformly distribute the compressive stresses of the building's walls to a wider area. In stiff soils, wooden beam grids with cross sections of 170mm up to 200mm square were utilized instead of wooden boards. Those beams were spaced at a distance which ranged from 0.45m to 1.00m and the resulting in-between space was filled with compressed gravel. When the soil stability was secured, foundation walls were constructed. Those walls were made of stone and their width was equal or greater than the width of the walls of the first storey of the building. In buildings without a basement, the depth of the foundation walls was equal to 1/7 up to 1/6 of the total building height, whereas in buildings with a basement foundation walls extended to a depth of 1.00m up to 1.20m below the basement floor. During modeling it would be wise to assume that the wooden elements (piles, boards and beams) may have undergone some extent of disintegration after those many decades of contact with the soil or the environment and therefore it would be pertinent to model contacts (structure to soil, timber elements to structure, etc.) through use of proper modeling tools (springs, etc) particularly in representing the stiffness of the foundation of such buildings.

The vertical load-resisting elements of neoclassical buildings were walls comprising stones or bricks. Walls of this type had a significantly larger thickness as compared to the infill walls of reinforced concrete buildings (ranging from 0.30m to 1.20m), whereas this thickness was gradually reduced starting from the ground floor to the top of the building. Stone walls were usually three tier walls, having an exterior facade built of large, regularly-shaped rectangular stone blocks, whereas the in-between space was filled with smaller stones of arbitrary geometry, soil and crushed bricks or tiles. To secure the uniform distribution of gravity loads to all of the three vertical layers and to avoid lateral separation of the tiers, horizontal brick lacing layers were constructed for each meter of stone wall height or so and across the entire wall thickness. In the case of brick walls, solid bricks (i.e. without holes) were mostly used. In some cases of buildings with several storeys, the brick walls of the upper storeys were made of bricks with holes (lighter), in order to effect a reduction in the magnitude of the structure's self weight in the upper storeys. Finally, to secure that crossing walls would not separate during an earthquake, timber ties comprising two longitudinal timber beams were placed along the wall's length and anchored at the external face of the crossing walls, forming a passive confining arrangement for the infilled masonry blocks. After the introduction of iron in the construction practice (in the end of the 1860s) timber ties were replaced by iron blades. Keeping in mind the construction practice of the walls of the neoclassical buildings, it is essential to model the different types of strengths of the walls. Also note that analyses procedures referring to combinations of actions in ultimate limit state design of modern buildings which have relatively small weight compared to their total volume (i.e. 1.35g) are inappropriate for neoclassical buildings, as they impose unrealistically large loads and masses to the structure.

Finally, the floor systems of neoclassical buildings built before the 1870s often had timber beams as structural elements, which were spaced at a distance of 0.60m up to 1.00m, serving to transfer the loads from the floor to the surrounding walls. When the wall thickness in two

successive storeys remained constant, the timber beams of the floor between those two storeys were seated in pigeon-holes in the walls. In cases where the wall thickness in the storey above a floor level was reduced by setback of the interior façade of the wall, then the timber beams of the floor were nailed on beams parallel to the walls whose purpose was to uniformly distribute the floor loads along the length of the walls. After the 1860s the timber beams were replaced by iron elements. Two different versions of floors with iron beams were constructed. In the first case, secondary timber beams were placed between successive iron beams, to provide resistance to lateral torsional buckling in order to secure the constant spacing between parallel iron beams and also to allow timber floor planks to be supported by nailing. The second and most usual type of a floor with primary iron beams comprised brick arches spanning between parallel iron beams. Thus, consideration of the mechanisms developed in the structural system of this class of buildings, owing to the different types of floor configuration (friction between the timber beams and the walls, differential movement between the brick arches and the iron beams, etc.), is of paramount importance in the modeling procedure, as they can influence the analyses results to a significant extent.

3 EXAMPLE OF SIMULATING AND ANALYZING A NEOCLASSICAL BUILDING

To investigate the sensitivity of the analysis results obtained from simulation of neoclassical buildings with models comprising the various types of mechanisms that were discussed in the previous paragraph, a series of dynamic analyses have been performed to different types of models of a neoclassical building located in the centre of Thessaloniki. This building was constructed in 1893 in order to house the first Hellenic high school of Thessaloniki, according to the designs of Ernest Chiller, as those modified of the architects Kabanakis and Kokkinakis (Fig. 2); since its construction, the building had been continuously in operation as a school until the summer of 1978, when it suffered damage by a strong earthquake of 6.5 M_w on the Richter scale that struck the city of Thessaloniki. The epicenter of the ground motion was North-East of Thessaloniki, in the Volvi lake region.

3.1 Description of the Structural System

The neoclassical building considered in the analyses is a two-storey building with a basement and a roof, located in the centre of the city of Thessaloniki, Greece. The building has a 20.83m x 16.05m 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, including the roof, is 14.20m. The first storey comprises six rooms, symmetrically located in both sides of the storey's corridor. The second storey was originally constructed to have four halls, one big hall at each side of the corridor spanning from the front view of the building (north view) to the south and two smaller ones in the southern part of the plan. Over the years, the two big halls were divided, the eastern one with the addition of a 0.15m thick brick wall and the western one with the addition of a wooden panel. The basement comprises five rooms that were used as storage rooms and sanitary facilities, after the addition of miscellaneous infill brick walls. Floors were connected with a wooden stair in the southern part of the main corridor. The walls of the basement are made of stone, having a thickness equal to 0.75m in the perimeter of the building and 0.65m in the inner plan. Walls of the first and the second storey were built of solid brick. Perimetrical walls have 0.50m width, whereas internal walls 0.40m. Floors of the first and the second storey were made of double T iron beams having a 60mm x 180mm cross section, spaced at 0.70m 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 0.33m at the location of the iron beams and 0.25m 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.

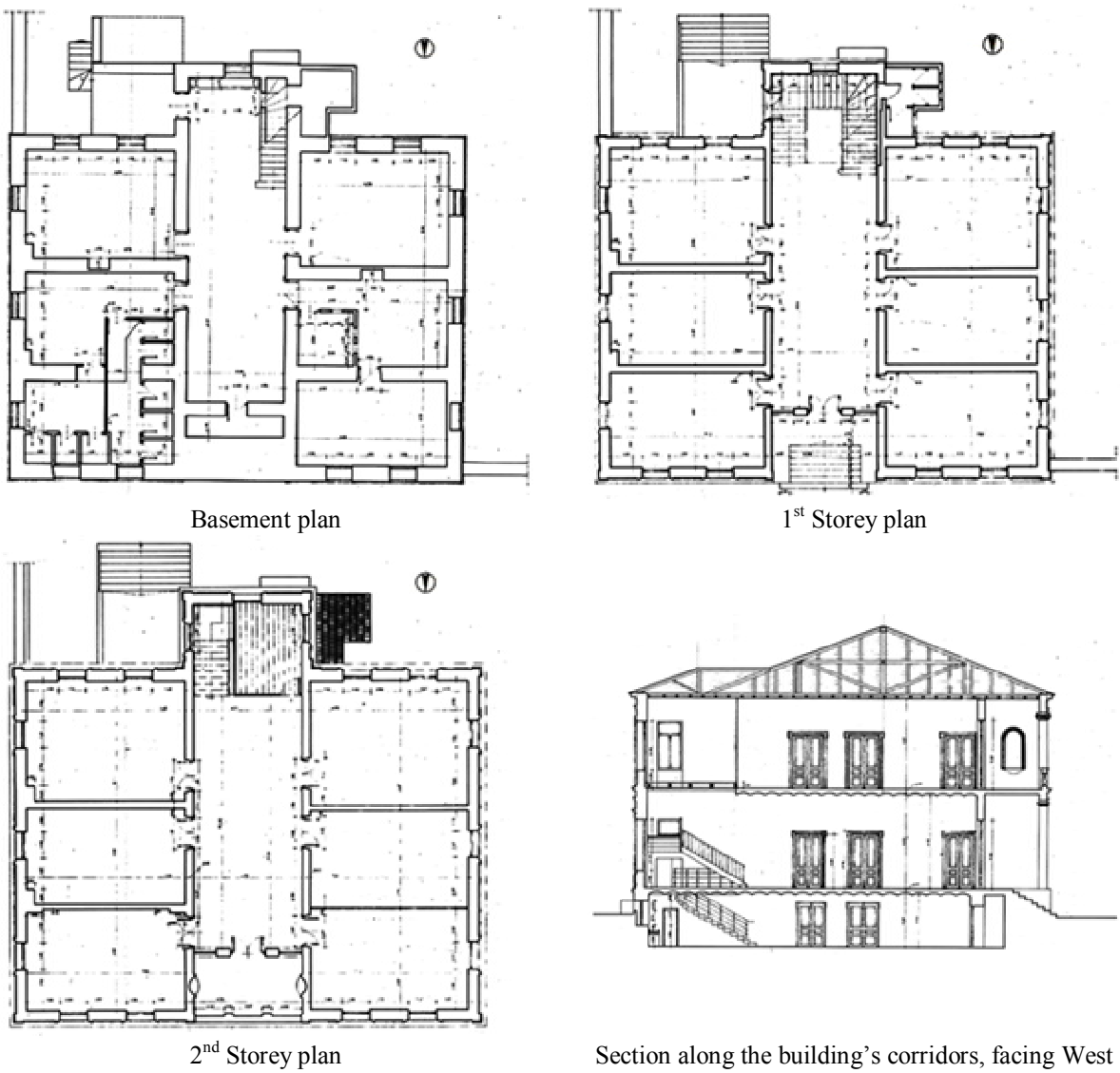


Figure 2: Plan and section views of the building used for the analyses.

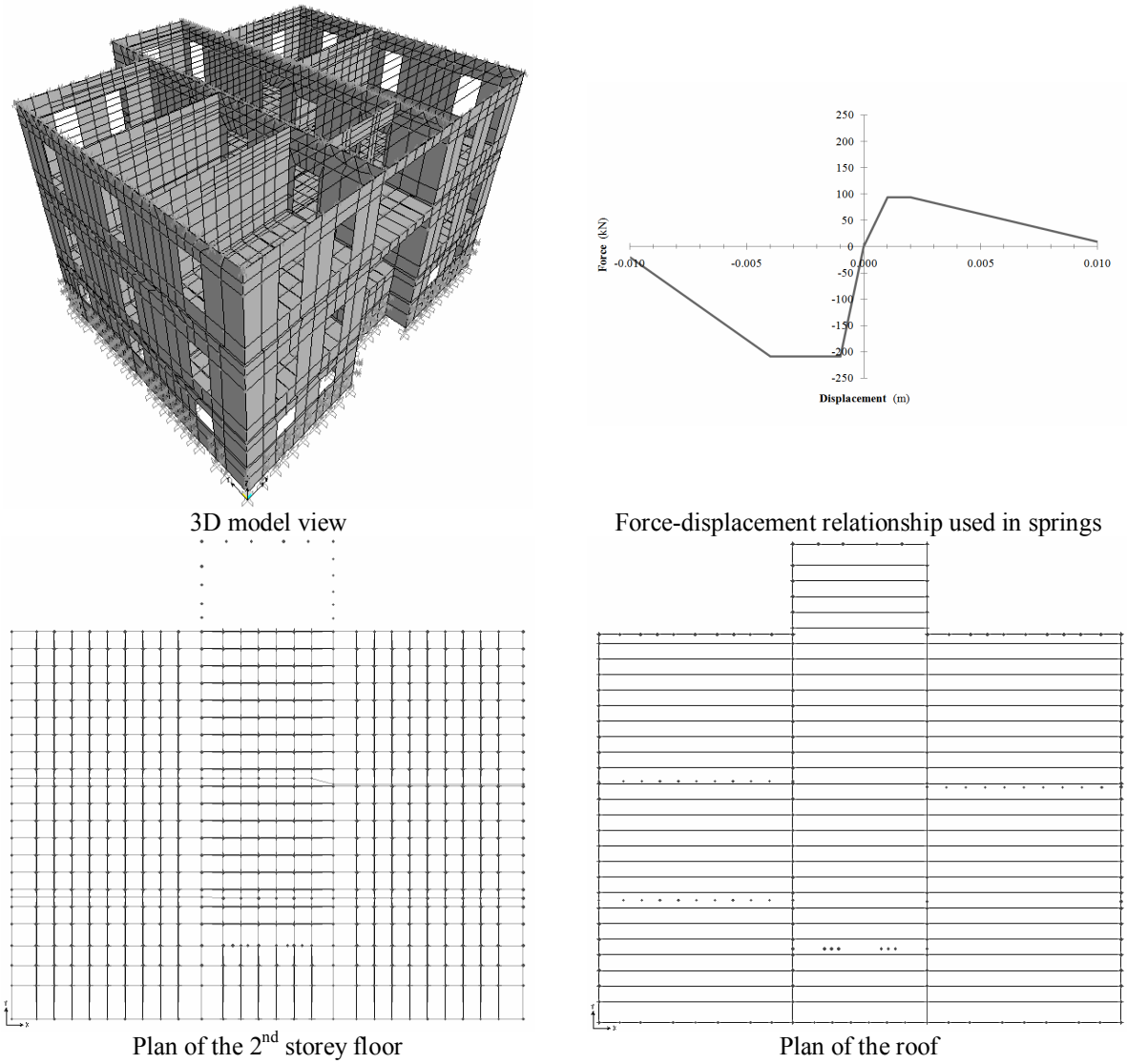
3.2 Description of he Structural Modes Used in the F.E. Study

To examine the degree of influence of the different mechanisms that are developed in the structural system of the building on its overall seismic response, parametric dynamic analyses were carried out using the recorded accelerations of the 1978 earthquake; results were compared with the observed damage patterns reported in post-earthquake reconnaissance of the building. Simulation and analyses were carried out using a finite element analysis program [7]. Building walls were modelled using four-node shell elements, capable of supporting forces and moments (6 d.o.f. per node). In the simulation possess, the wall thickness was reduced by 50mm from the measured dimensions in order to eliminate the plaster thickness on both sides of the walls. Thus, shell elements simulating the basement walls were assigned a thickness of

0.70m and 0.60m in the cases of exterior and interior walls respectively, whereas upper walls were modelled using a thickness of 0.45m and 0.35m for the exterior and interior brick walls of the first and the second floor. Along the basement walls, above and below the basement windows, two zones of shell elements accounting for brick lacing were used. Infill walls in the location of internal doors between the southern and the central storage rooms of the basement, the northern basement window of the west view of the building and between the northern and the central room at the east side of the corridor at the second storey were simulated using shell elements of 0.15m thickness, accounting for walls made of voided-bricks, since those additions were introduced in 1975. Timber panel divisions were omitted (in the northern and the central room at the west side of the corridor of the second floor). To investigate the degree of alteration of the building's seismic response due to the floors and the roof configuration, two model buildings were produced, *Model A* and *Model B*.

In *Model A*, the 1st and the 2nd storey floors were simulated utilizing linear elements, fixed at their ends to the shell elements of each room's wall, in order to account for the iron beams, whereas the brick arches were modelled as shell elements having a 0.16m thickness (i.e. the average thickness of the structural system of each of the two floors), accounting for voided-bricks, fixed to various points of the surrounding walls and the iron beams. Linear elements were used at the roof level, accounting for the timber beams having a 100mm square cross section over the perimeter walls and the two sides of the corridor, which were considered to be fixed at the nodes of the wall shell elements. Also included were linear elements parallel to the E-S direction of the plan, spaced at 0.60m and fixed at the nodes of the timber beams over the storey walls. These elements had the same cross section as the timber beams over the walls and modelled the horizontal members of the roof trusses.

In *Model B* (Fig. 3), gap elements were added between the nodes of the shell elements that represented the brick arches and the corresponding nodes of the iron beams, replacing the fixity condition for the same contact detail in *Model A*. Thus, gap elements were intended to model the separation between the brick arches and the iron beams when in tension whereas they were considered rigid in compression (perfect contact). Also added in *Model B* were springs between the nodes of the wall elements and the timber beams over the wall, which were meant to account for friction between the wall and the timber beams. Spring-response was described by a multi-linear force-displacement relationship (Fig. 3), that was calculated according with the development capacity over the beams' contact surfaces when accounting for friction with the building walls. In both models, the response of the shell and the linear elements was considered elastic. Modulus of elasticity of stone and bricks was considered 1000 times the value of the corresponding compressive strength, f_k; for stone f_k = 5.5 MPa, for solid bricks f_k = 4.0 MPa and for voided bricks f_k = 1.5 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 1GPa in the other two sectional directions. In all cases, the building load was taken equal to the self weight of the building and no variable load was considered, as during the earthquake (8pm of June 20) the school was not operational. Material density was 28.5 kN/m^3 for stone, 18 kN/m^3 for solid bricks and 14kN/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.



3D model view

Force-displacement relationship used in springs

Plan of the 2nd storey floor

Plan of the roof

Figure 3: Model B; Three-dimensional view of the model, plan views of the 2nd storey floor, the roof and force-displacement relationship used in the friction springs that connect the timber beams with the walls of the second storey.

Parametric modal and dynamic analyses were carried out on both models. To determine the free vibration mode shapes and the natural periods, *T*, of the building, both models were subjected to Ritz-vector modal analyses, to account for the spatial distribution of the dynamic loading. Also performed were dynamic time-history analyses according to the earthquake acceleration recordings of the 1978 Thessaloniki earthquake [8]. Response spectra of the accelerations used in the analyses with respect to the building's principal directions in plan, calculated when assuming 5% damping, are presented in Fig. 4. Masses considered in the Modal and Time-History analyses were automatically calculated by the program, by multiplying each element (shell or linear) volume by their material volume mass respectively.

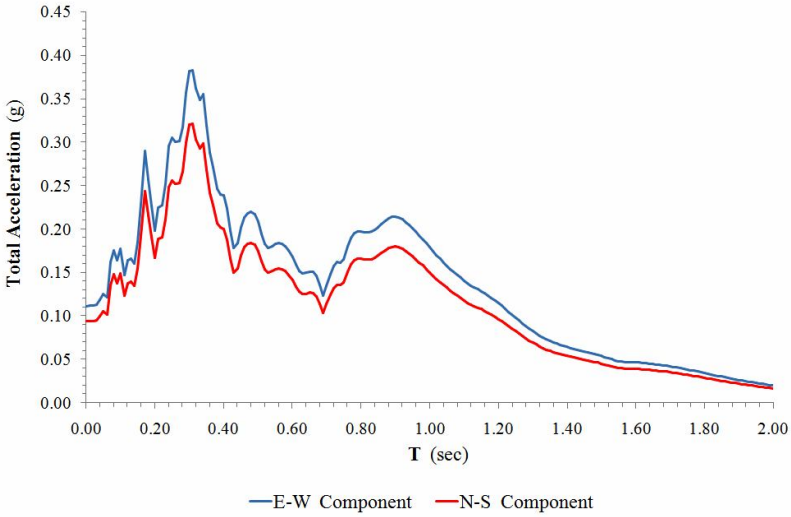


Figure 4: Elastic acceleration response spectra of the two components of the 1978 Thessaloniki earthquake that were used in the model analyses, calculated for 5% damping. The two components resulted from the same earthquake acceleration record, by projecting it to the two primary axes of the building's plan orientation.

From the results of the analyses of *Model A* and *Model B*, the following observations concerning modelling of neoclassical buildings for the purposes of seismic assessment are extracted. The first observation concerns the developed stresses in the shell elements of both models. Figure 5 presents the shear stress distribution across the walls and the floors of the two models as calculated from Time-History analyses in the time interval of the building's maximum seismic response in terms of roof displacement. As observed, the introduction of gap elements in the first and second storey floors as well as the use of friction springs in the connection of the timber beams with the second storey walls resulted in a significant reduction of the developed shear stresses of floors from *Model B* at about 50% in the first and 40% in the second floor as compared to the same shear stresses from *Model A*. Also altered were the stress distributions across the buildings floors, with *Model B* presenting a more uniform shear stress distribution than *Model A*. The same conclusions about the comparison between the magnitude of the developed stresses of *Model A* and *Model B* also apply in the developed shear stresses at the walls of the buildings, with the spread between those values being reduced to about 30%. Figure 6 illustrates the developed shear stresses on the different facades of the two models at the time of the building's maximum seismic response, together with the actual damage patterns of the building after the 1978 earthquake. From this comparison it is concluded that *Model B* correlates better with the actual damage patterns of the building than *Model A*.

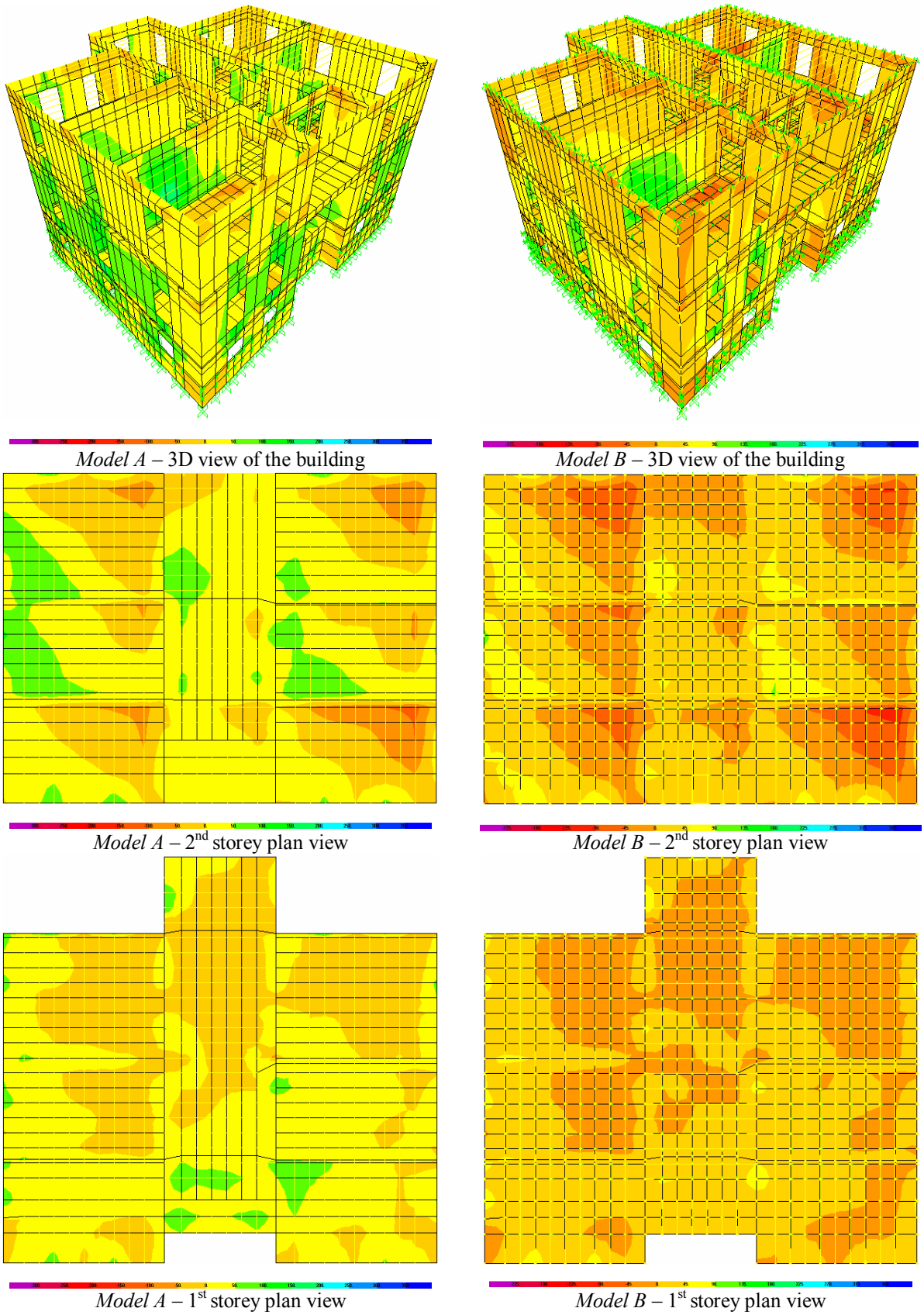
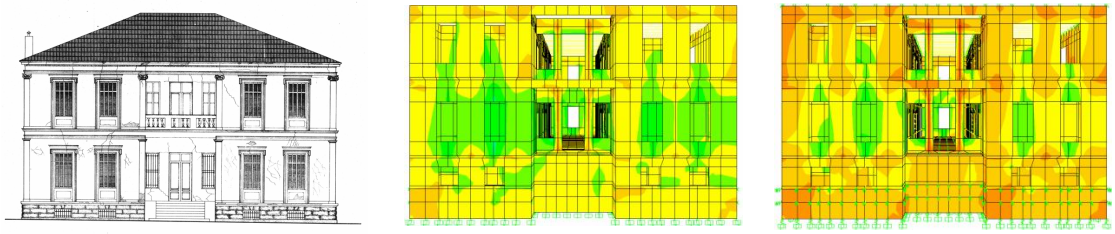


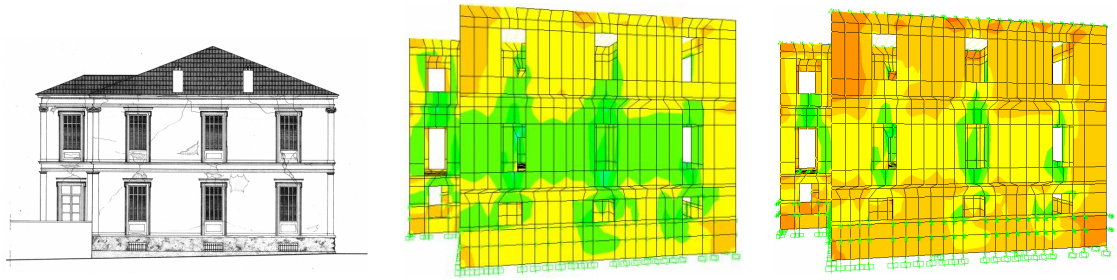
Figure 5: Shear stress distribution over the shell elements of Model A and Model B, as calculated at the time of the building's maximum seismic response from time-history analyses.

NORTH VIEW



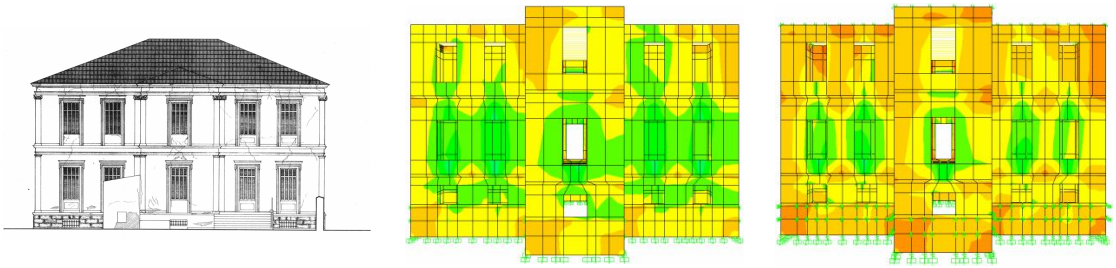
Damage pattern after the earthquake | Shear stresses in *Model A* at time of max. response | Shear stresses in *Model B* at time of max. response

EAST VIEW



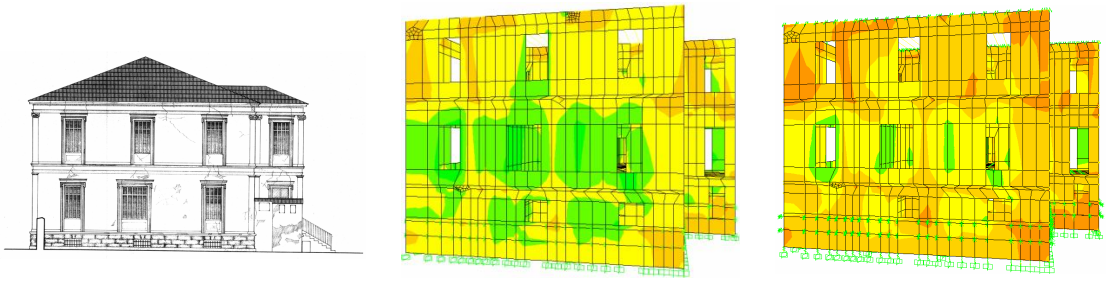
Damage pattern after the earthquake | Shear stresses in *Model A* at time of max. response | Shear stresses in *Model B* at time of max. response

SOUTH VIEW



Damage pattern after the earthquake | Shear stresses in *Model A* at time of max. response | Shear stresses in *Model B* at time of max. response

WEST VIEW

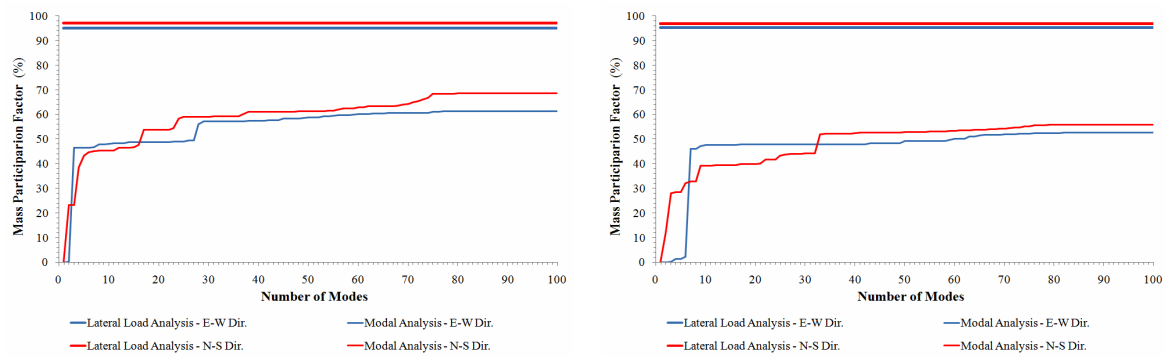


Damage pattern after the earthquake | Shear stresses in *Model A* at time of max. response | Shear stresses in *Model B* at time of max. response

Figure 6: Comparison between the damage patterns of the building after the 1978 Thessaloniki earthquake and the developed shear stresses in *Model A* and *Model B* at the time of the building's maximum seismic response in terms of roof displacement.

Modal analysis conducted on the buildings led to a large number of translational modes with closely spaced periods. This led to a group of very similar modes each having a small participation factor, leading to the requirement of including several modes in the calculation in order to mobilize the structural mass. Here an alternative approach was used to approximate the fundamental mode of vibration of the structure, by subjecting it to a notional gravitational field in the direction of lateral translation (i.e. along the longitudinal and transverse directions of the plan geometry). In light of the fact that gravitational forces are proportional to the mass of the structure, and the restoring forces in this situation 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 [9]. The mass associated with this shape obtained for *Model A* of the building was estimated at 95% of the total mass, a result that suggests a fundamental inconsistency between the modal analysis and the mode approximation method; this inconsistency, which is basically owing to the discretization approach used with the finite element model, is most likely responsible for the apparently false notion that the seismic response of the neo-classical building is governed by many mode shapes with relatively small mass participation factor.

This point is documented in Figure 7 which illustrates the spread of the mass participation factors for the first 100 modes of *Model A* and *Model B* in each of the buildings' principal directions, as calculated from the results of Ritz modal analyses. Also presented are the mass participation factors in each of the corresponding directions, derived from the deformed shape of the two models after lateral loading with their self weight (a horizontally oriented gravitational field). Note that the sum of the mass participation factors of the first 100 modes of *Model A* equals to 61.4% in E-W direction and 68.6% in N-S direction, whereas the corresponding mass participation from lateral loading with the building's weight is 95.0% and 97.1% respectively. Dispersion of the results is greater between the modal analysis mass participation factor, whereas mass participation obtained from lateral loading is even greater in *Model B*; 52.8% vs. 95.2% in E-W direction and 55.9% vs. 96.9% in N-S direction respectively.



Model A *Model B*

Figure 7: Comparison between the mass participation factor for the first 100 modes of *Model A* and *Model B* obtained from Modal Analyses, and Mass participation calculated from the deflected shape when models are subjected to a horizontal gravitational field along the E-W and N-S plan directions.

Based on resent research [10] which indicated that the displacement response distribution of a structure matches very well with the shape of the principal translational mode in the corresponding direction if the mass participation factor exceeds the value of 80%, a rapid ap-

proximate method for determining the maximum seismic response of neoclassical buildings is introduced. According to this procedure, the seismic response of a neoclassical building, in terms of developed stresses or deformed shape, can be determined in a two step procedure:
(a) First a simple static analysis is performed, where the model is loaded laterally with its own self weight acting in each of the two principal directions of the plan.
(b) In each direction of the building's plan, the corresponding quantity which is investigated (nodal displacements and/or developed elements stresses/forces) is multiplied by a factor which equals to the ratio of the spectral relative displacement normalized by the peak displacement throughout the building obtained from the gravitational analysis.
Figure 8 plots the comparison between the displacement profiles obtained from time history analysis and from the proposed spectrum-based procedure for the east corner of the North façade of the building. Note the excellent correlation between the two procedures.

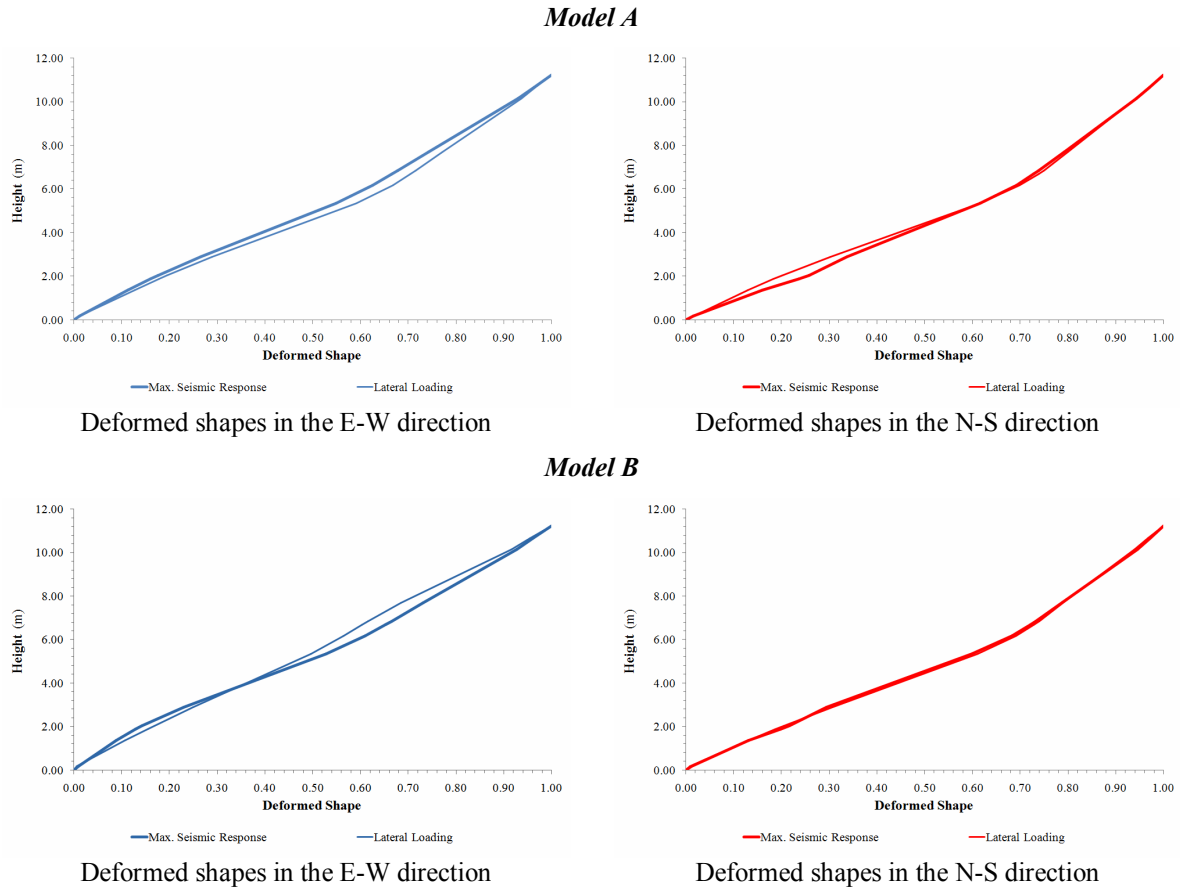


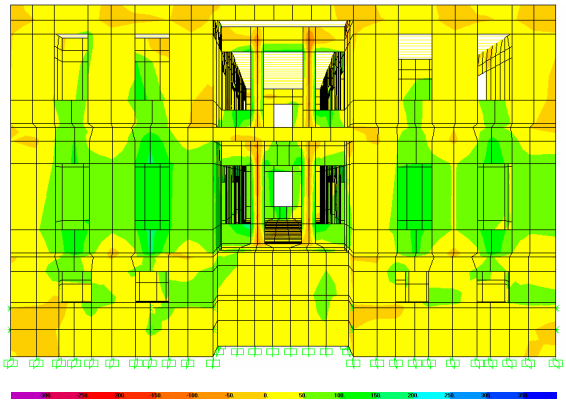
Figure 8: Deformed shapes of the east corner of the building's north face; Comparison between the shapes from the time-history analyses at the time of the building's maximum seismic response and the shapes from the lateral loading of the building with its self weight.

Figure 8 illustrates the displacement profiles of the east corner of the north face of *Model A* and *Model B*, for the cases of the maximum seismic response according to time-history analyses, and the deformed shape as calculated by the lateral loading of the models with their self weight. The correlation of these shapes is very good, especially in the N-S direction of the building.

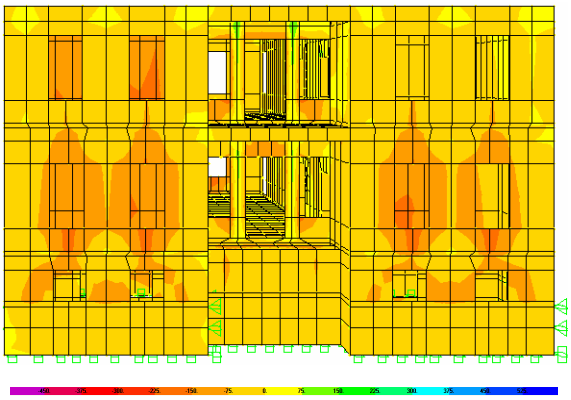
In Fig. 9, comparison of the shear stress distribution along the shell elements of the building's north view between the cases examined, i.e. the results from time history analysis (at the time

of peak response) against the results from the proposed spectrum-based approach is illustrated for *Model A* and *Model B*. As in the case of the comparison of the models' deformed shapes, the correlation between the shear stresses from the two analysis methods is very good.

Model A

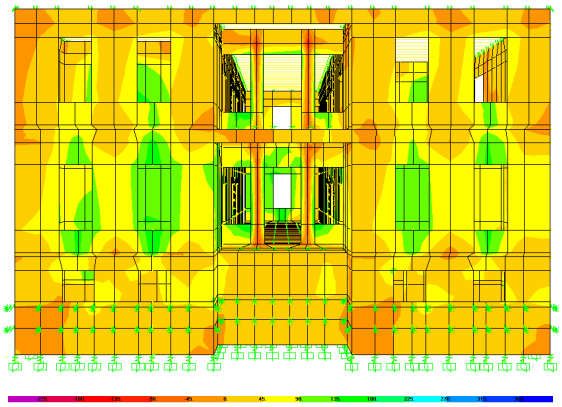


Shear stress distribution obtained from time-history analysis

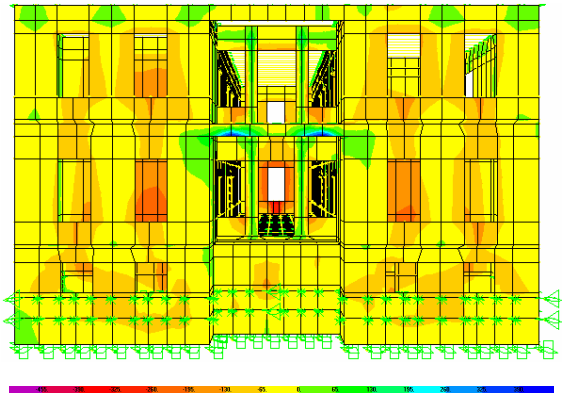


Shear stress distribution obtained from lateral loading static analysis

Model B



Shear stress distribution obtained from time-history analysis



Shear stress distribution obtained from lateral loading static analysis

Figure 9: Shear stress distribution at the north face of the building at the time of peak seismic response; Comparison between the results obtained from time-history analyses and static analyses with lateral loading proportional to the building's weight (a horizontal gravitational field).

4 CONCLUSIONS

Seismic assessment of neoclassical buildings of the 19th and 20th century through simulation is a very demanding task. Profound knowledge of the design and construction practices of 19th and 20th Century is essential so that the structural engineer can simulate accordingly the mechanisms that can be developed in this class of structures during an earthquake. By neglecting detailed consideration of the contact effects at the points of interaction between different materials can lead to serious overestimation of demands and a commensurate underestimation of the residual strength of the building, guiding towards severe rehabilitation methods that may alter or destroy the unique historical or architectural characteristics of the building. In the present study an example of modeling of an actual neoclassical building located in the centre of Thessaloniki, Greece, which suffered damages from the 1978 earthquake was presented, were two different types of modeling the horizontal diaphragm stiffness

and the diaphragm connection to the walls were tested. According to the analyses results, the difference in the magnitude of shear stresses developed in the two models was in the range of 45% for the floor elements of the building and about 30% for the walls.

Also presented in this paper is a simple and very reliable analyses procedure, in order to determine the maximum seismic response (displacements and/or forces/stresses) of neoclassical buildings. This procedure is based on the resent research according to which the deformed shape of a building at the time of its maximum response is approximated very well by the shape of its principal translation-twisting mode if the mass participation factor is greater than 80% [10]. According to this procedure, the maximum seismic response of a neoclassical building can easily be determined from the results of a static analysis, where lateral loading of the building model is performed with its self weight, multiplied accordingly so that the horizontal displacements of the target node at the top of the building be equal to the relative displacement of the elastic spectrum used in the assessment procedure.

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