NATIONAL TECHNICAL UNIVERSITY OF ATHENS

School of Civil Engineering

MSc in Analysis and Design of Earthquake Resistant Structures

THESIS TOPIC

Survey of damages, assessment of seismic behavior and interventions to the listed building of Vasileos Georgiou 27, Thessalonikis

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<u>THESIS TOPIC:</u>

SURVEY OF DAMAGES, ASSESSMENT OF SEISMIC BEHAVIOUR AND INTERVENTIONS TO THE LISTED BUILDING OF VASILEOS GEORGIOU 27- FORMER 12TH PRIMARY SCHOOL THESSALONIKIS

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...to my mother

ABSTRACT

The current thesis involves the assessment of the seismic behaviour of a two storey unreinforced masonry listed building in Vasileos Georgiou and Mpizaniou Street in Thessaloniki. The vertical load bearing system of the building comprises by rubble stone masonry at the basement and brickwork masonry at the ground and upper – first floor level. The horizontal load bearing system of floors and roof structure are made by timber elements. The only exception is at the N-E part of the building where the floor is made by arched brickwork between steel beam members at all floor or ceiling levels. The building was modelled by using shell and frame finite elements with the use of SAP2000. The seismic analysis of the building was based on the Eurocode 8 elastic spectrum using a model with mechanical properties for walls based on experimental data of samples (stone and brick) tested in the laboratory. The results of the analysis revealed the mechanism of partial failure of the top floor walls mainly due to out of plane bending moment. Strengthening and intervention techniques such as the enhancement of diaphragm action at roof level is presented and implemented on the model while maintaining the architectural and aesthetical appearance of the historic building.

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CHAPTER 1

1. INTRODUCTION

1.1 General

The neo-classical villa, known as ‘Former 12th Primary School’, is situated at the junction point of Vasileos Georgiou and Mpizaniou Street at the South-East part of Thessaloniki.

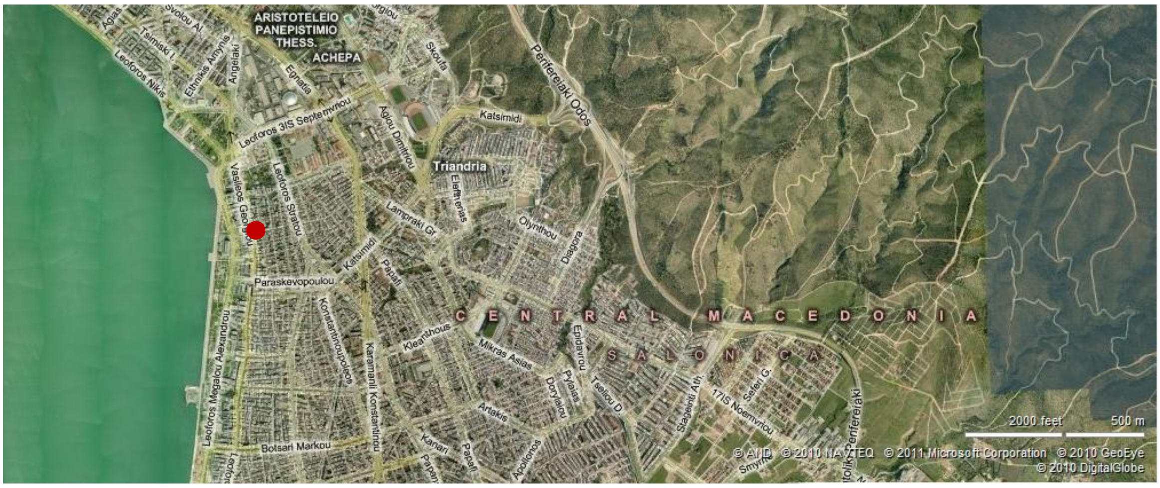


Figure 1.1 Panoramic view of SE part of Thess/niki

The location of the building together with the other two neighbouring buildings at Vas.Georgiou 25 and 29 (Figure 1.3, No.22 and 24) are a unique example of three in a row neo-classical buildings of the time before the area became occupied by modern high rise blocks of flats.

Figure 1.2 Panoramic view of the local area

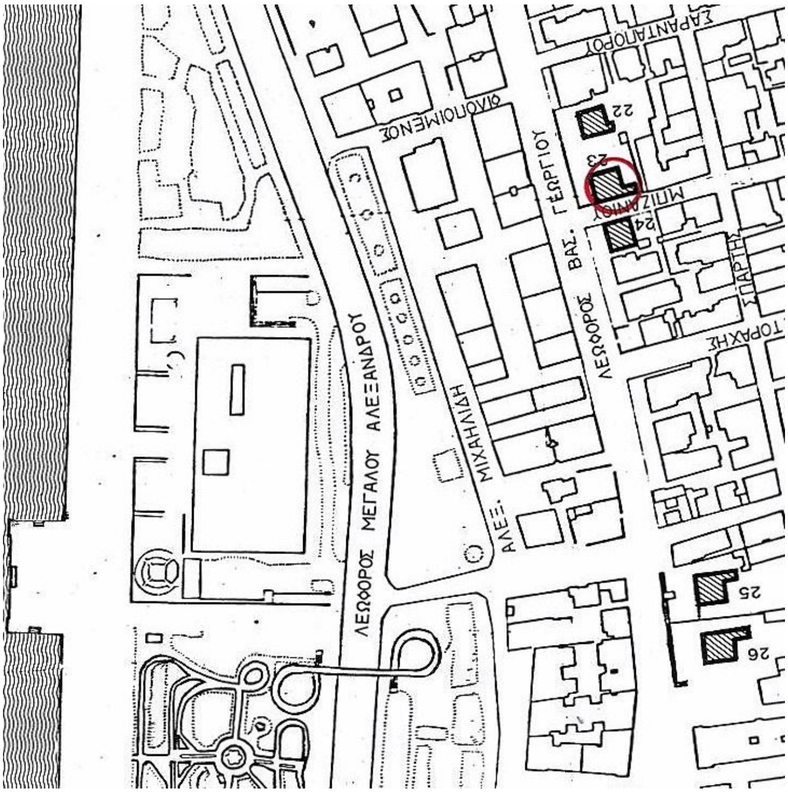


Figure 1.3 – Old map of the section area of "Towers"

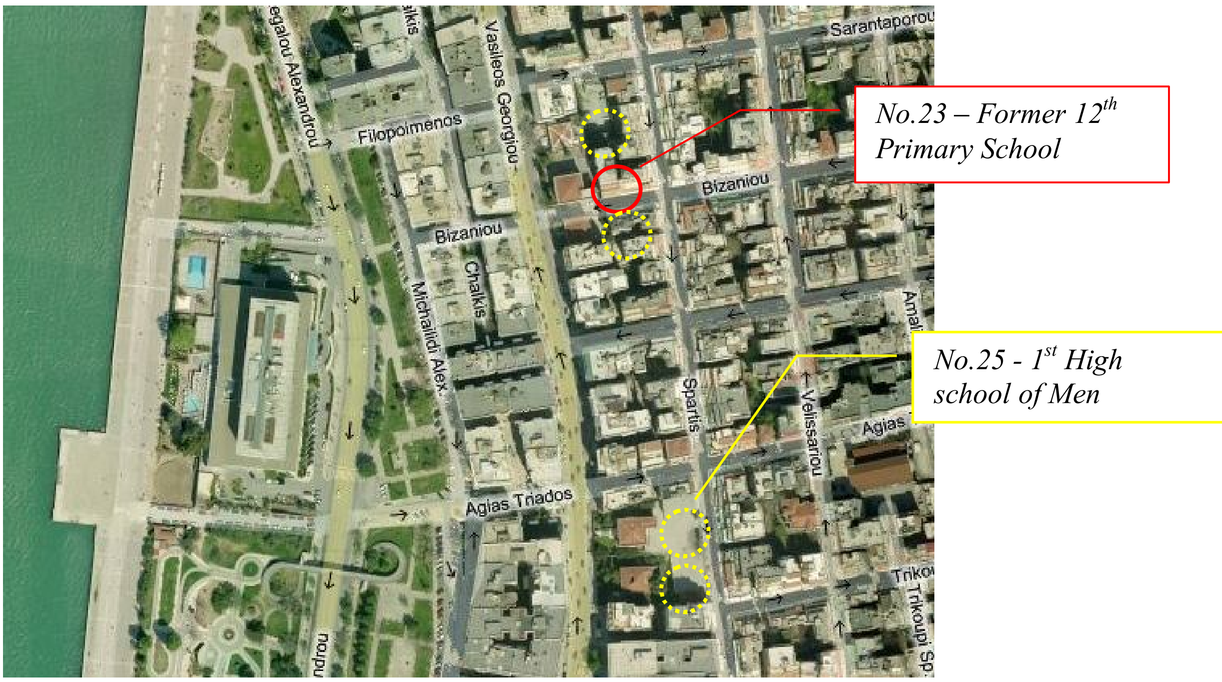


Figure 1.4 Panoramic view of the section area of "Towers"

The villa of Vasileos Georgiou 27 (former 12th Primary School) appears to have exactly the same architectural elements as the 1st Men's High School building at Vasilisis Olgas 3 (Fig.1.5,1.6 and No.25 in the map of Fig.1.3). Both of these building are within short distance from the coastline and only about 200m apart.

Figure 1.5 Former 12th Primary School – Main Front view from Vas. Georgiou St. (No.23 on map) (Archived section by YASBE).

Figure 1.6 1st Men's High School – View by Vas.Olgas 3 St. (No.25 on map)

1.2 Historical background

The exact age of the building is not known but it is estimated to have been built in the late 19th century, between 1891-1896, when it was used as a residence by Ivan or Givantse Aggelou Chatzimisef family until about 1915. During 1919-1952 the Greek authority occupied the building, and in 1952 the ownership passed into the ownership of the Greek Orthodox Church (O.D.E.P).

Since 1919 the building starts having different uses. For instance, it was used as offices to host the American Embassy, the 5th Men's High School, the National Youth Group (N.Y.G), the 6th and 7th Financial Office Ministry/Departments and since 1974 until May 2004 the 12th Primary School of Thes/niki. The current building was characterised/ classified as listed after the earthquake of 1978 after a decision made by the Ministry of Culture under ΥΠΠΕ/ΔΙΛΑΠ/Γ/3672/78892/10-01-1983 that was published at the F.E.K. 154/B/07-04-1983.

1.3 Brief Record of interventions

During the lifetime of the building, a number of alterations took place. Particularly, the most significant structural intervention internally has been the demolition of brick walls both at ground and upper floor level after the initial change of its use from a residency to office and teaching rooms. This type of alteration is estimated around 1920 where that particular type of steel beams were still used and manufactured until the mid war period (1923-1940). In addition, the localised closure of the timber floor at ground level to prevent access to the basement from the secondary timber staircase was another smaller intervention probably for safety reasons during its use as a school.

Externally, along the west elevation at the courtyard there is a new reinforced concrete frame staircase entrance to the ground floor level. The staircase is estimated to have been built around 1974-1979 in order to facilitate the school needs and provide supervision of the pupils within the courtyard area. Also, for school purposes, an external steel frame staircase was constructed at the rear north side of the building prior to 1988 not only to support the existing terrace but mainly to allow access for the pupils from the courtyard to the upper 1st floor level for safety reasons (Fig.1.7-1.9).

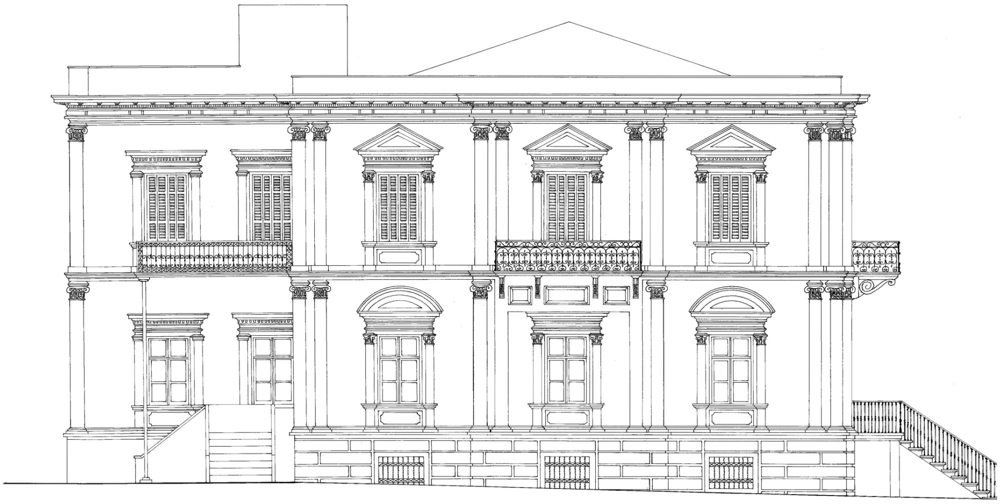


Figure 1.7 West elevation (YASBE archive of architect's drawings)

Figure 1.8 Panoramic west view (E.M.M photo archive ~ 1979)

Figure 1.9 Panoramic west view (YASBE photo archive ~ after 1988)

CHAPTER 2:

2. DESCRIPTION OF EXISTING STATE

2.1 SURVEYING

It should be noted that copies from the archived records of architectural drawings and photos (1988) were provided from the Earthquake Restitution Office of North Greece (called in greek - Y.A.S.B.E). Also, some additional surveying plan drawings and relevant photographic records, based on site investigation works carried out in 2004, were given by the Civil Engineers Mr. Ch.Ignataki and K.Stylianidi for an assessment of strengthening and rehabilitation of the school.

The survey and pathology of the building was concluded with additional and recent information of damages and interventions that took place since 2004.

2.2 Architectural elements

The building is two storey, with an upper-half basement, an upper ground and 1st floor level. The plan view of the main body of the building is square with a hipped roof enclosed within the parapet all along the perimeter of the main body. At roof level, the orthogonal part of the building at North side is covered by a flat roof/ terrace giving an overall L shape at the plan view of the building. The secondary staircase projects about 2.30m beyond the roof level providing access to the terrace level.

Figure 1.10 Panoramic west side view from inner courtyard (E.M.M, 1979)

The main front South view is along Vasileos Georgiou Street and consists of three symmetrical openings on each floor. At the central south part, there is a small balcony projection at 1st floor level with the main front entrance staircase to the upper ground floor.

Typically, next to the window or door openings there are decorative column /posts of Ionic and Corinthian style. Particularly, above all the openings of the upper -1st floor there are triangular gables and above the window openings at ground floor there are arched gables. A horizontal decorative strap separates and distinguishes each floor level with its architectural features. These architectural patterns are repeated on the rest of the side views.

In addition to the primary south entrance to the upper ground floor, there is also a secondary entrance from the north rear side of the building. Also, under the staircase of the main front south there is the main entrance door basement.

Figure 11 Main front South view along Vasileos Georgiou Street
(Photographic Archive of Y.A.S.B.E, 1988)

Based on survey study in 2004 by Mr. K. Stylianidi and Ch.Ignataki, the following information was obtained:

The overall height of the building, without its roof, is about 12m (with clear/ mixed heights of 2.7m at basement, 4.4m at upper ground and 4.9m at upper-1st floor level approximately). The upper/ half basement, due to external ground surface inclination of the courtyard projects/ extends (beyond ground surface level) from 1.20m at the N-E corner to 1.60m at the S-W corner.

The internal layout of the building follows the typical neo-classical arrangement having a central communal area with the rooms on each side. The original room layout should have been with three rooms on each side of the central communal area at both floor levels. This assumption is proven by the existence of the built in chimneys within the external/ perimeter side walls of each room (three on each side, along east and west elevation, as shown on Figure...). At the communal area of the ground floor, there is the main staircase providing access to the upper floor level. There is also a secondary staircase between the main body and the north-east section part (used as a kitchen area) of the building and links the floors with the top open terrace at roof level.

In order to be able to correlate the photographic record in Appendix I with the drawings in Appendix II, each floor of the building will be marked up with the following notation: basement (Y), ground (I) and upper floor (O).

2.3 Survey of the load bearing system

The vertical load bearing system of the building comprises rubble stone masonry at the basement (externally and internally) and brickwork masonry at the ground and upper – first floor level. The horizontal load bearing system of floors and roof structure are made timber elements. The only exception is at the N-E part of the building which was used as a kitchen and the floor is made by arched brickwork between steel beam members at all floors or ceiling levels.

2.4 Vertical load bearing system

Basement Walls:

The basement walls, as previously mentioned, are rubble stone masonry built (known as the 'greenstone' of Thessaloniki) with weak/poor strength of lime mortar. From relevant survey works by K. Stylianidi and Ch.Ignataki, it has been known that:

The effective thickness of the walls are approximately 60cm internally and 70cm externally with the exception of some perimeter walls that are wider of 90cm due to the support requirements of the balconies at south, east and west elevations. For the same reason the widening of these walls is repeated all along the wall height of upper levels too. Only the internal wall between Y8 and Y9 rooms, which used to be part of the secondary timber staircase, is made of header brickwork of 20cm thick.

For the foundation survey two local excavations were done internally at the basement. One at the S-W corner of the room no. Y2 and the other at the N-E corner of the room no. Y10 (refer to Drawing no. 01 and 06, Photos ...).
During the excavation works, it has been estimated/ assumed/ taken that the cut off level is the same all over and that there was no obvious signs of high water table.

Ground floor walls:

Typically, the internal walls are also header brickwork masonry of effective thickness 23cm except of the stretcher masonry walls between I1 – I11 and I8 – I9 rooms where the timber staircases are supported. The perimeter walls are typically double header brickwork masonry of effective thickness 46cm except of the local widening of 70cm due to the balcony supports. Based on visual inspection, there was no sign or evidence of horizontal tie beams at any height or elevation.

Due to the change in use of the building, as previously mentioned, the header brick walls between room I3 – I4 and I6 – I7 were demolished. As a result, the walls or timber floor beams above were supported by a timber beam and a pair of steel beams (as per Drawing No.02 and 05). At a later recent stage around 2004, an additional steel frame post arrangement of 100x100 SHS was placed below the original steel beam supports in order possibly to provide lateral support at the transverse walls due to the lack of the original wall (Photos….)

Although there are not any further signs of demolished walls at ground floor or basement, there is a pair of original steel beams at the ceiling level of room I1, which support a header brickwork masonry wall above at 1st floor level between rooms O1 – O1a. Based on previous local investigations of the ceiling by Stylianidi & Ignataki, the pair of steel beams are placed within concrete infill and appear to have surface corrosion at the bottom of their flanges.

The wall section below the windows at ground floor level appears to have a recess of 11cm except for the wall section of room I6 which has got a deeper recess of 33cm approximately. The piers above the openings appear to be built from two layers of 6 hole manufactured bricks in arched shape.

The north part of the wall in room I10 was rebuilt to fill in the recess of an existing fireplace (when the room was used as a kitchen) having left an obvious crack from the insufficient brick bond/ joint between the old and new infill (Photo...). In the same room I10, on the east wall side there is another recess of 22cm that was used as a cupboard with proper size window dimensions next to an existing window opening (Photo...). It is therefore evident that at that N-E corner of the building there is a significant reduction of the wall capacity/ strength.

<u>Upper-1st floor walls</u>:

Similarly, the internal walls are header brickwork walls of effective thickness 23cm except of the single stretcher brick wall between rooms O8 – O9 that supports partly the secondary staircase. The perimeter walls are wide header type masonry walls with effective thickness of 34 and 57cm where required for the cantilever balcony supports.

Due to the demolition of walls between rooms O3 – O4 and O5 – O6, timber beam and a pair of steel beams were placed to support the ceiling joist at roof level (Drawing No….). Once again, in order to provide some form of lateral supports to the transverse walls, a steel frame with 100x100 SHS posts was added below the original pair of steel beams.

Based on information obtained by Ignataki & Stylianidi research:, *The opening of the header brick wall between rooms O1α – O2 is 3.40m wide and 2.65m high. The remaining side wall piers are each one of 0.85m wide. A concrete beam of dimensions bxh=25x20cm was constructed as a lintel above the opening and the remaining header brick wall above was rebuilt.*

As previously mentioned at ground floor, the built in chimneys continue at upper floor creating a vertical gap along the construction of perimeter double header brick walls along the west and east elevation. Not only the room I10 at ground floor but also the room O10 on 1st floor has got the same problematic areas with fireplace wall recess and cupboard recess combined this time with increased dampness (Photos ...).

2.5 Horizontal load bearing system

<u>Basement floor ceiling/ Ground floor</u>:
Typically, all the timber floor beams are 85x190mm at 400mm centers approximately. On top of the timber beams there is only floor boarding and all ceilings are covered by a ceiling board.

At the section cut of the basement ceiling of room Y1at the door entrance of room Y6, the timber floor beams appear to be supported to a longitudinal timber beam that sits along the wall (Photos...). Based on some other local section cuts at ground floor level at rooms I4 and I6-I7, there is not a tie beam and the timber floor beams are supported directly to the basement wall (Photo ...).

The ceiling of room Y10 consists of arched brick floor supported between 'I' section steel beams of 140mm high at 450mm spacing (Dwg. 01). This type of steel beams is not classified on the typical category of INP beams (Normal Profile) given that the flange width is 50mm instead of 66mm for a standard INP140. So, on any drawings the notation used is "I140" to refer to this category.

<u>Ground floor ceiling/ 1st floor level</u>:
At ground floor ceiling, there is a similarity of materials and section sizes. At the ceiling level of room I1 there is a pair of steel beams I140 among the timber floor beams that support the header brick wall above from rooms O1 – O1α (Photo...). As previously mentioned, due to the demolition of walls between I3 – I4 and I6 – I7, the 1st floor timber joists are supported on a timber beam and a pair of steel beams I140. Further to this, a steel gable post/ frame has been added below the original intervention in order to provide support on the transverse walls (refer to Photos .... and Section detail Dwg.No.05).

The floor of the open terrace/ balcony at the rear of the building consists of arched bricks supported between steel beams I140 at 500mm spacing (Photos…) These steel beams are supported between the north perimeter wall of the building and couple of edge steel beams of I200 (with flange width 60mm instead of 90mm for INP200) that are partly supported at the perimeter walls and partly on a steel post of Ø120 that is founded at the courtyard ground level.

During the construction works of the external steel staircase for the safe access of pupils at 1st floor level, the terrace- balcony floor (plan view dimensions of 4.70m x 5.00m) has been propped at its mid span by a new steel beam of INP140 that is supported on three steel posts 80x80x6 SHS (Photo...). Also, the primary edge steel beams I200 have been strengthened by another steel beam (80x80x6 SHS) that is also supported on the above mentioned steel posts.

The load bearing system of the small balconies is made of 3 marble slabs (of 1.17x1.05m and 2.55x1.05m plan view dimensions and 10cm thick) that are supported on 4 cantilevered marbled beams of 12cm that are fixed at the head of ground floor perimeter walls.

1st floor ceiling / Roof:
Based on information obtained from Ignataki & Stylianidi survey for the roof:
The ceiling joists are 75x120mm at 35cm centers and are supported at the wall heads without any timber tie connection running along the wall head. Similarly with the ground floor, the support of the ceiling joist, due to the wall demolition of rooms O3 – O4 and O5 – O6, is on the timber beam and pair of steel beam (Refer to Dwg.no...and photos...).

The main load bearing system of the roof is based on two main diagonal timber trusses (Dwg.No.04, Photos...). The timber trusses do not have a complete truss support arrangement between. The tie beam of the lower part of the main truss (130x150mm) is supported partly at the ceiling joists (75x120mm at 350mm spacing) and partly on the wall heads of internal walls. The diagonal sloped members of the truss are propped through vertical tie members which are supported on internal brick walls. On top of the main truss, there are the longitudinal roof joists (65x90mm at 350mm spacing) that are supported at the bottom edge on the head of perimeter walls. Then, there is the plating which carries the French type ceramics as final roof cover.

It's been also noted that the perimeter walls of the secondary staircase do not cut the main diagonal truss at the N-E corner of the roof (Photo 97…??). The ceiling of the staircase and the top terrace floor of (room O10) have got flat roofs made of arched bricks between I140 steel beams.

2.6 PATHOLOGY OF LOAD BEARING STRUCTURE

2.6.1 General

An overall view of the building shows a fairly stable structure without any existing vertical wall movement or any separating cracks at the corners and junctions with perimeter walls. Also, so far, there are no signs of damages on the superstructure due to differential settlement of the foundations.

2.6.2 Horizontal load bearing system

At the timber floors, the diaphragm rigidity is almost negligible because the majority of timber floor beams are supported directly to the walls without any appropriate connection to a tie beam that runs parallel to the wall. Also, the lack of an appropriate tie between timber beams of adjacent floors contributes to the reduced diaphragm action. Therefore, the only existing diaphragm action relies on floor and ceiling boarding.

The arched floors between steel beams, at all ceiling levels of Y10, I10 and O10 rooms at the N-E part of the building, provide considerable diaphragm rigidity. However, the absence of any tie at the support of those steel beams (I140) at the wall head will also reduce the diaphragm action even on such floors. It should be noted that after a recent visual inspection, the surface of the bottom steel flanges are corroded (Photo ...).

Based on local section cuts of the floor boarding or ceiling, the timber beams or ceiling joists appear to be in fairly good condition without any timber rotting. Any obvious local balancing of floor is typical on this kind of timber floors. Although the main timber staircase from ground to first floor is in a good state, the secondary staircase is partly unstable and unsafe to be used not only between ground and upper floor but also for the access to the terrace at roof level.

The load bearing system of the roof is in a good condition and there is no question of its structural adequacy. Any local small damages are due to rain water pipeline drainage leakages. Having no access within the roof any information listed below is based on survey work data by Stylianidi & Ignataki. *The insufficient connections and junctions of the timber trusses and the absence of continuous bottom tie beams of the truss do not allow having a complete and efficient truss system support action. As a result, the diaphragm rigidity is insufficient being limited only at the contribution of plating. At the negligible diaphragm action of the roof contributes also the fact that there are no tie beams at the wall heads of upper floor to provide a sufficient connection of the load bearing roof members with perimeter or internal walls.*

At the marble made floor of small balconies there are not any significant cracks or damages to show any problem of the rigidity of the cantilever supports within the perimeter walls. For the 1st floor terrace at the rear of the building there is only an obvious corrosion at the main original steel edge beams (Photo....). Given that the arched floor within steel beams have been supported Στον μεγάλο εξώστη της βόρειας όψης παρατηρείται σοβαρή οξείδωση των σιδηροδοκών I200 κατά μήκος των during the intervention of the steel frame staircase, there is not any sign of movement under live loading conditions.

2.6.3 Vertical load bearing system

An important role in any damage of the building was played, on the one hand, by the demolition of internal walls and the insufficient strengthening of the load bearing system prior to the 1978 earthquake, and on the other hand by the limited/ partial maintenance and necessary protective measures 33 years after the earthquake.
Based on visual inspection, a qualitative interpretation of representative damages of cracks and interventions is shown in Appendix II, in a series of sectional drawings.

In general, it should be noted the following:

- A series of walls have been demolished completely such as between I3 – I4 and I6 – I7 rooms at ground floor and also at 1st floor between O3 – O4 and 05 – O6 rooms. The only partial demolition of a wall is between rooms O1a – O2 at 1st floor level (Section 1b). It is quite likely that the demolition of the wall between I6 – I7 rooms may have caused the cracks at the wall above between O6 – O7 due to the deformation of the steel beams I140 that was used to support the wall above (Section 2b).

- The presence of vertical cracks at the junction point of wall O2-O3 with the perimeter wall of west elevation (Photo... and Section dwg.2b), and also at the meeting/ intersection point of wall O9-O10 with the perimeter wall of east elevation are both of special interest (Section dwg.5a). Particularly, on the second case, the crack width is about 5-6mm showing an insufficient connection with the perimeter wall interface.
- There were not any visible cracks in the rubble stone masonry walls at basement. The only damages were locally at rooms Y10 and Y9 (Photo...) with the disintegration of plaster and mortar due to weathering conditions.
- Any walls that do not continue all the way through the floors above, they offer a minor influence/ contribution taking any lateral loading. On this category are the walls O6-O7, O1a-O1 at 1st floor and I1-I11, I5-I6 at ground floor. It should be noted, with exception of the wall at O1a-O2 that is partially removed, all the rest of the walls that have been demolished and are not continuous at the floors above are walls with an east-west orientation. This may result in a significant increase of the moment or shear resistance requirements on remaining walls of this direction under lateral/ seismic loading.
- At present time, all apparent cracks in brickwork masonry walls are mainly at 1st floor level. The majority are oblique flexural and shear cracks located above openings and starting from the corner points of doors or windows towards the ceiling. It can be said that this kind of cracking is common on structures having fairly strong piers and weak lintels given the lack of horizontal ties above the openings. Hence, even under the influence of a mild seismic loading cracking will develop at the weaker areas above the openings.

Chapter 3:

MECHANICAL PROPERTIES OF WALLS

From laboratory tests of samples of stone and brick the following results were obtained, respectively:

3.1 Rubble stone of basement walls

α/α Sample	Stone dimensions *l x b x (h)* *(mm)*	Weight (kgr)	Unit Weight γ_s		Compr. Load P_u (kN)	Loaded area *l x b* (mm^2)	Compression strength f_s (MPa)
			(kgr/mm^3)	(kN/m^3)			
1	54x40x(61)	0.372	2.8233E-06	27.70	291.1	2160	134.77
2	50x61x(39)	0.354	2.976E-06	29.19	414	3050	135.74

Table 3.1 Results of rubble stone compression tests

Based on the above results, for the rubble stone we have:

Unit weight: γ_s **= 28.45 kN/m^3**

Compression strength: f_s **= 135.25 MPa**

From relationship $f_{wc} = \frac{2}{3}\sqrt{f_{sc}} - \alpha + \beta\, f_{mc}$, the compressive strength of rubble stone is derived f_{wc} **= 5.753 MPa.**

3.2 Brickwork walls of ground and upper floor

α/α Δοκιμίου	Διαστ. πλίνθων l x b x (h) (mm)	Βάρος (kgr)	Ειδικό Βάρος (kgr/mm^3)	Ειδικό Βάρος (kN/m^3)	Φορτίο Θραύσης P_u (kN)	Φορτιζόμενη επιφάνεια l x b (mm^2)	Θλιπτική Αντοχή f_b (MPa)
1	210x100x(74)	2.197	1.414E-06	13.87	144.8	21000	6.90
2 (υπόγ.)	64x50x(45)	0.245	1.701E-06	16.69	48.5	3200	15.16
3(υπόγ.)	40x40x(43)	0.128	1.86E-06	18.25	47	1600	29.38
4 (όρ.)	52x45x(50)	0.187	1.598E-06	15.68	10.3	2340	4.40

Table3.2 Results of brick compression tests

Based on the most conservative option, the brick compression strength is taken:
f_b **= 5.65 MPa**.

From relationship $f_{ck} = K \cdot f_b^{0.7} \cdot f_m^{0.3}$, the compressive strength of brickwork is derived

$\mathbf{f_{ck} = 1.54}$ **MPa**

3.3 Foundation ground

Based on existing ground site investigation report of an adjacent site and given the seismic zone II and ground foundation category B of current Aseismic regulation, the allowable earth ground stress is taken $\sigma_{allow} = 150KN/m^2$.

CHAPTER 4:

MODELLING – ANALYSIS

The building walls were modelled and analyzed using finite element analysis (SAP2000). Linear elastic behaviour was assumed using three and four node shell elements, a process that is usually adopted for the analysis of similar structural systems. Beam elements were used to model the timber and steel members.

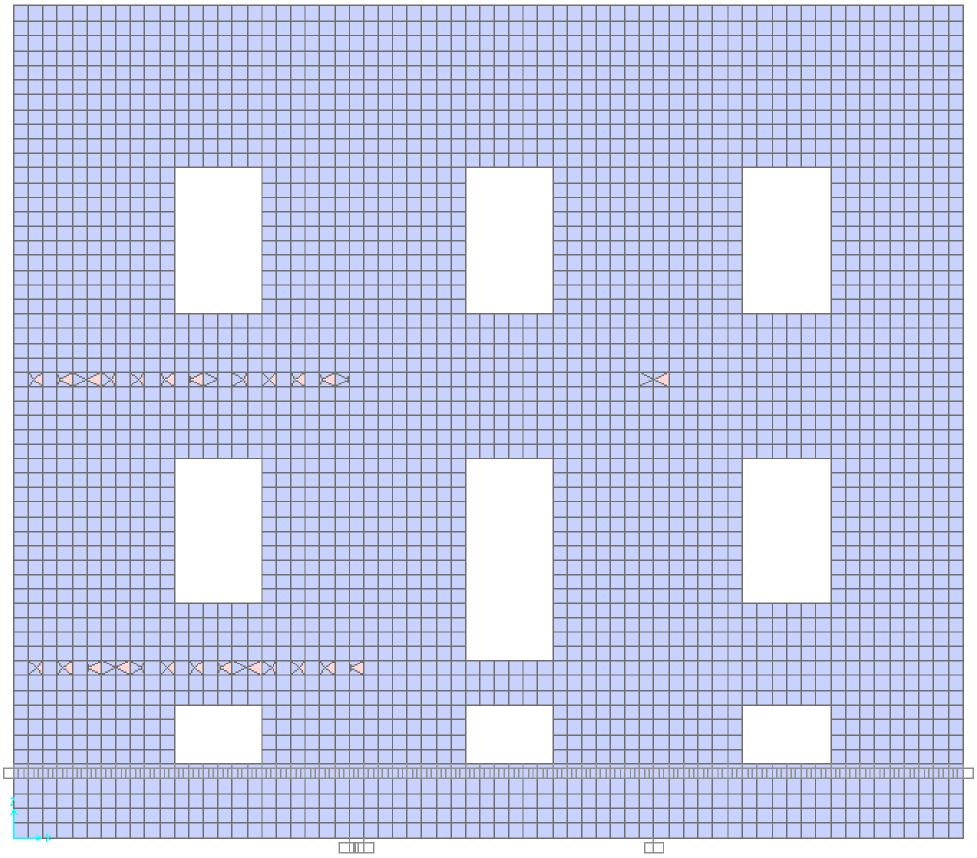


Figure 4.1 Typical perimeter wall meshing from the model

Along X direction the 5th mode is the first most significant with T= 0.27sec and modal mass (20%)

Along Y direction the 6th mode is the first most significant with T= 0.26sec and modal mass (25%)

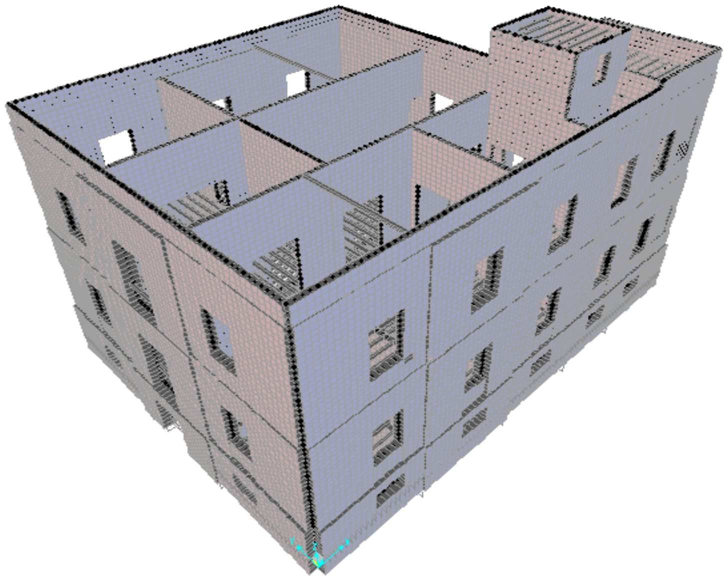


Figure 4.2 3D model

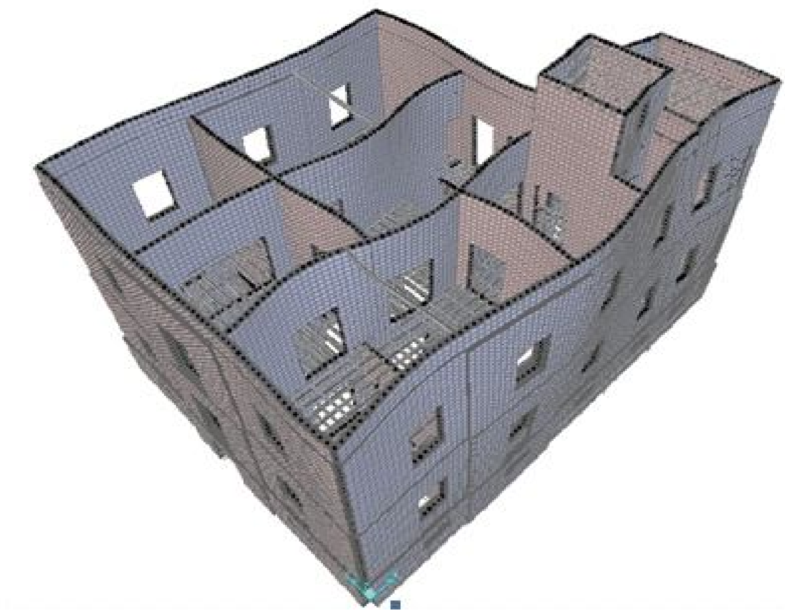


Figure 4.3 6th mode shape along Y direction

CHAPTER 5:

DESIGN CHECKS – CORRELATION OF ANALYTICAL RESULTS WITH OBSERVED DAMAGES

5.1 DESIGN CHECKS

The main checks that have been carried out were for shear capacity and for out of plane bending moment under dynamic loading.

1. Calculation of the resisting shear force and comparison with the acting shear

Satisfying the relationship $V_R = f_v\ l_c\ t \geq V_s$

$f_{vk} = f_{vko} + 0.4\ \sigma_d$

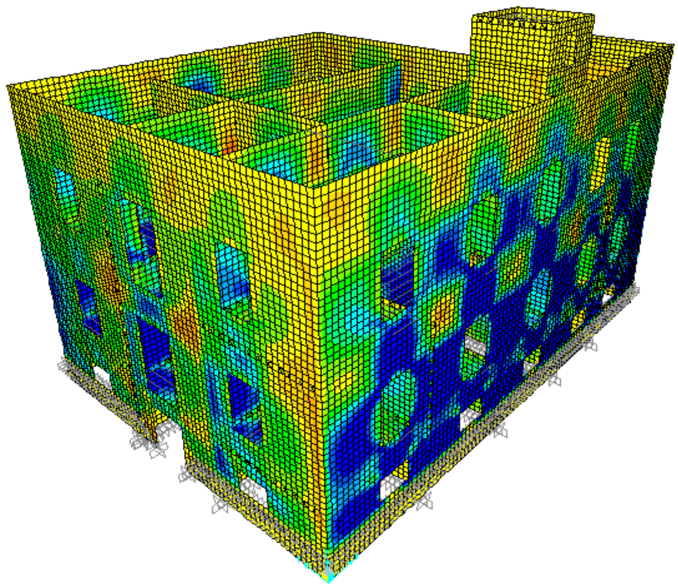


Figure 5.1 3D model for shear

2. Calculation of the out-of-plane bending moment for:

a). Bending parallel to horizontal joints

$M_R = ½ \sigma_d \cdot t^2 \cdot l \cdot (1- \sigma_d / f_c)$

$M_R > M_S$

b). Bending perpendicular to horizontal joints

$\sigma_t = M_s / W = 6 M_s / lt^2 < f_t$

(Assumed f_t = 100kPa)

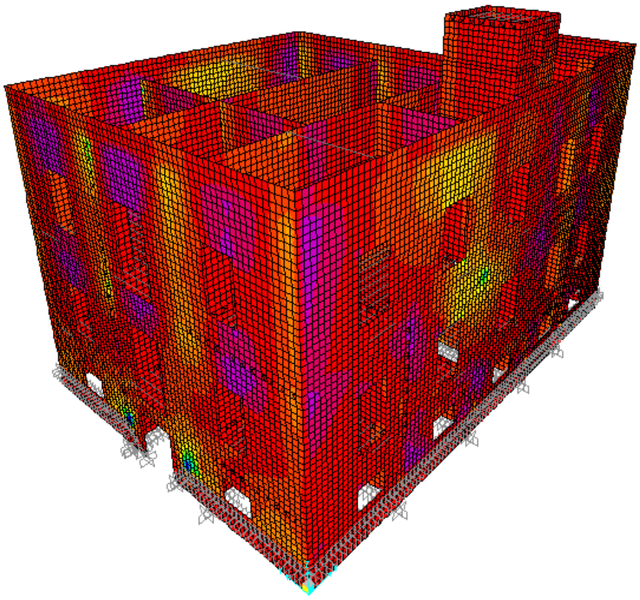
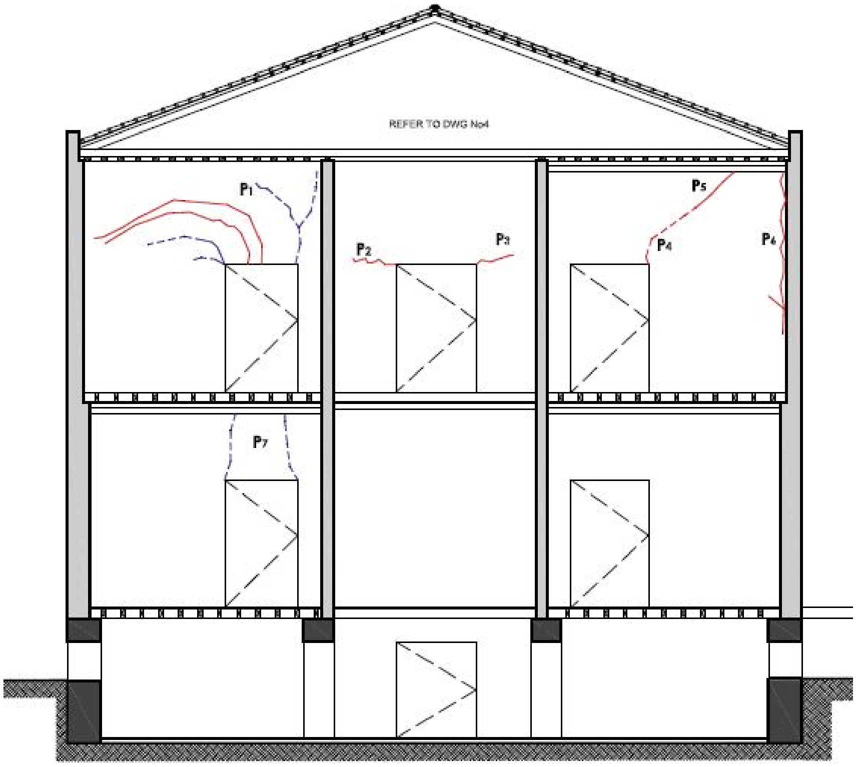
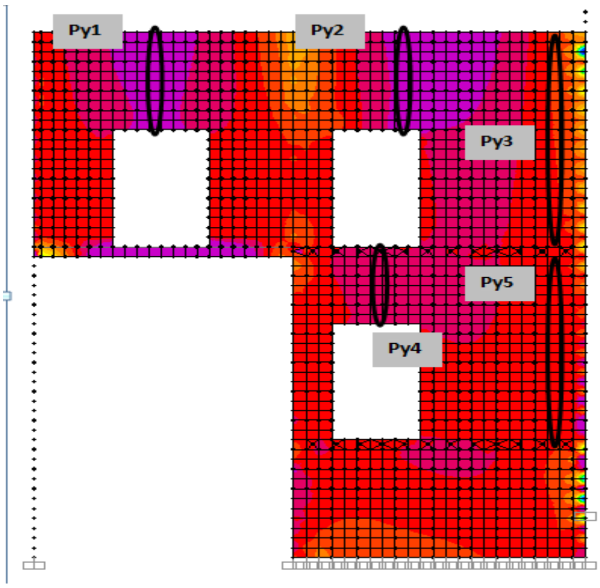


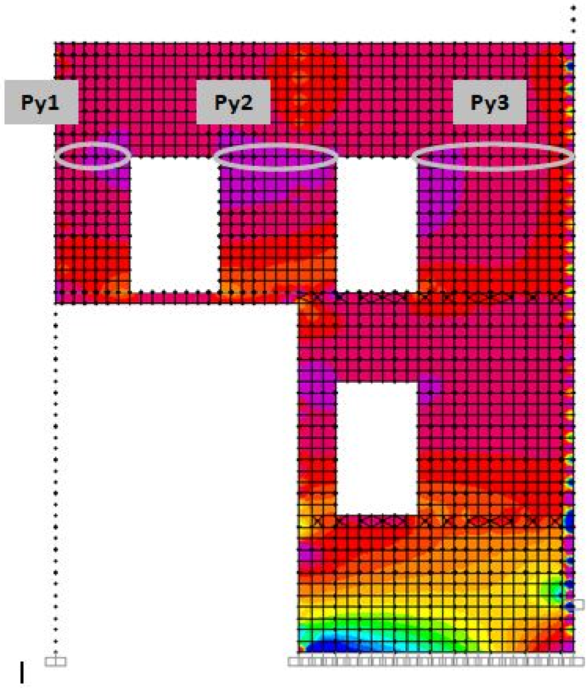
Figure 5.2 3D model for the out of plane bending moment

5.2 CORRELATION OF ANALYTICAL RESULTS WITH OBSERVED DAMAGES

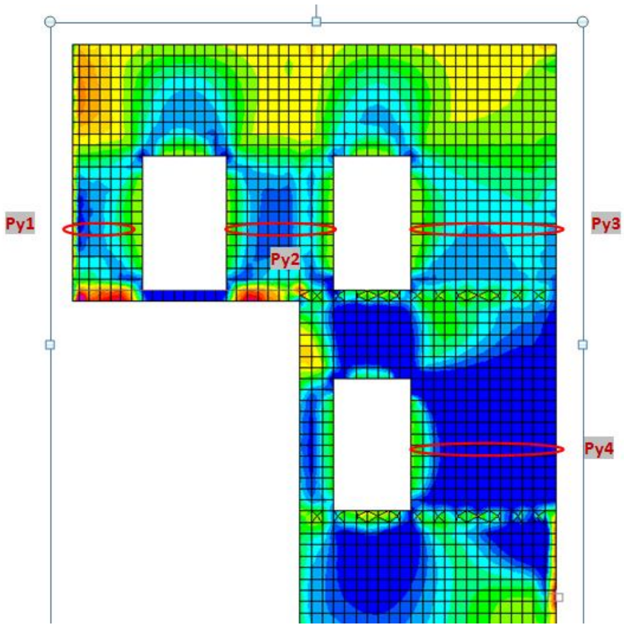
The sections and images below represent the relation of the damages with the shear and out of plane bending moments respectively.



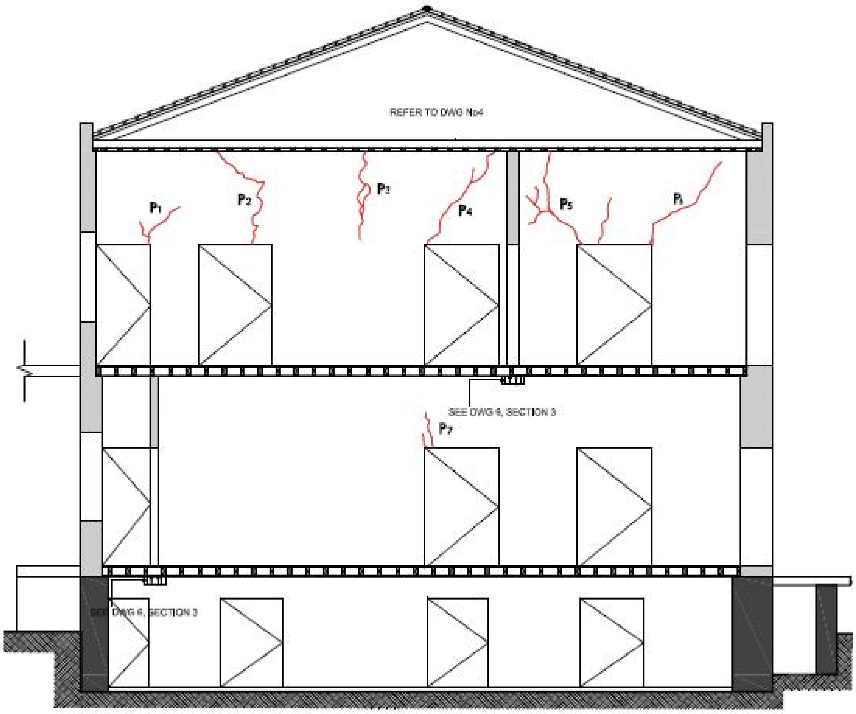


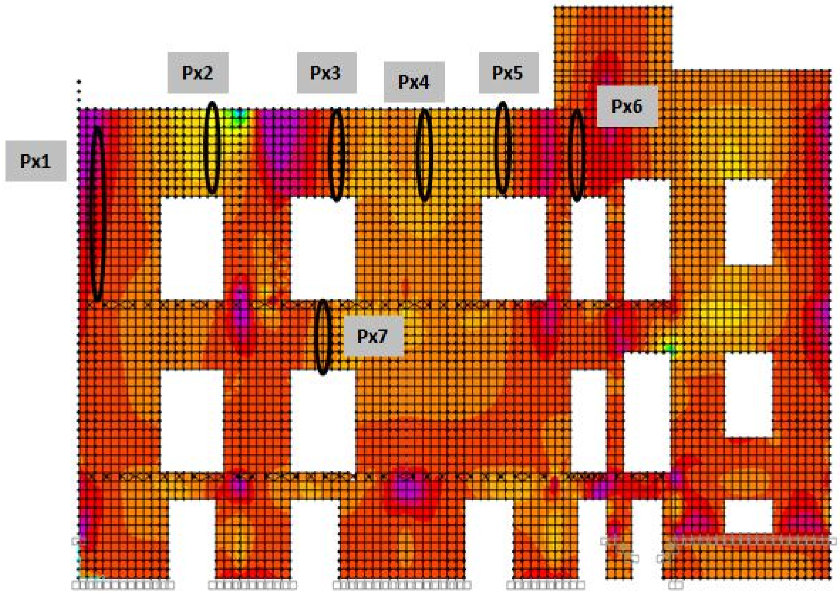


Py1
Py2
Py3



Py1
Py2
Py3
Py4





A sample of the design check for the above is shown below on the tables.

Table 1

Load combination: G+0.3Q+Ex+0.3Ey			Wall dimensions		Check of Bending Moment (M_{22}) parallel to horizontal joints (walls vertically to hor.load)							
Piers	Section cut level	Plane Orientation	l_w-length (m)	b_{w-} thick. (m)	N_u (= Fz) (kN)	$M_{d1} = M_{22}$=(M_1) (kNm)	M_s (kNm)	σ_d (kN/m^2)	$f_d = f_c$ (kN/m^2)	(1-σ/f_c)	M_R (kNm)	RESULT
Py1	Hor. cut @ 1st. Floor	YZ pl.@Grid B1	1.38	0.23	-411.54	11.1	11.10	1296.60	1540	0.16	7.48	FAILURE
Py2	Hor. cut @ 1st. Floor	YZ pl.@Grid B1	2.1	0.23	-3.17	22.7	22.70	6.56	1540	1.00	0.36	FAILURE
Py3	Hor. cut @ 1st. Floor	YZ pl.@Grid B1	2.88	0.23	-187.3	-8.22	8.22	282.76	1540	0.82	17.58	OK
Py4	Hor. cut @ 1st. Floor	YZ pl.@Grid C	2.87	0.23	-236.3	2.8	2.80	357.98	1540	0.77	20.86	OK
Py5	Hor. cut @ 1st. Floor	YZ pl.@Grid D	2.9	0.23	-138.98	21.3	21.30	208.37	1540	0.86	13.82	FAILURE
Py6a	Hor. cut @ 1st. Floor	YZ pl.@Grid E	1.6	0.46	-13.3	-26.14	26.14	18.07	1540	0.99	3.02	FAILURE
Py6b	Hor. cut @ 1st. Floor	YZ pl.@Grid E	1.7	0.23	-347.13	7.5	7.50	887.80	1540	0.42	16.91	OK

Table 2

Load combination: G+0.3Q+Ex+0.3Ey			Wall dimensions		Check of Bend. Moment (M_{11}) perpend. to horizontal joints					
Piers	Section cut level	Plane Orientation	l_w-length (m)	b_w-thick. (m)	N_u (= F_1) (kN)	$M_u = M_{11} = (M_z)$ (kNm)	Z -Sect. Mod. (m^3)	σ_t (kPa)	f_t (kPa)	RESULT
Py1	Vert.Cut @1st	Yzpl. @ Grid B1	2.23	0.23	-8	53.4	0.02	2716.0	100	Failure
Py2	Vert.Cut @1st	Yzpl. @ Grid B1	2.23	0.23	-46.2	-63.5	0.02	3229.7	100	Failure
Py3	Vert.Cut @1st	Yzpl. @ Grid B1	4.9	0.23	-190.34	-83.55	0.04	1934.0	100	Failure
Py4	Vert.Cut @Gr.	Yzpl. @ Grid B1	1.76	0.23	-11.7	-18.74	0.02	1207.7	100	Failure
Py5	Vert.Cut @Gr.	Yzpl. @ Grid B1	4.4	0.23	-58.7	-28	0.04	721.8	100	Failure
Py6	Vert.Cut @1st	Yzpl. @ Grid C	5.12	0.23	-203	-102.2	0.05	2264.0	100	Failure
Py7	Vert.Cut @1st	Yzpl. @ Grid C	2.23	0.23	-18.93	-71.65	0.02	3644.2	100	Failure
Py8	Vert.Cut @1st	Yzpl. @ Grid D	4.9	0.23	-146.2	52.58	0.04	1217.1	100	Failure
Py9	Vert.Cut @1st	Yzpl. @ Grid D	2.23	0.23	-10.42	-37.86	0.02	1925.6	100	Failure
Py10	Vert.Cut @1st	Yzpl. @ Grid E	4.9	0.23	-388.36	193	0.04	4467.4	100	Failure
Py11	Vert.Cut @1st	Yzpl. @ Grid E	2.23	0.23	-38.6	45.9	0.02	2334.6	100	Failure

Table 3

Load combination: G+0.3Q+Ey+0.3Ex			Wall dimensions		**Check of Bending Moment (M_{11}) perpendicular to horizontal joints (for all walls)**					
Piers	Section cut level	Plane Orientation	l_w- length (m)	b_{w}- thick. (m)	N_u (= F_1) (kN)	M_u =M_{11}= (M_z) (kNm)	Z -Sect. Mod. (m^3)	σ_t (kPa)	f_t (kPa)	RESULT
Px1	Vert.cut @ 1st	XZ pl.@Grid 2	4.9	0.23	-203.4	205.8	0.04	4763.7	100	Failure
Px2	Vert.cut @ 1st	XZ pl.@Grid 2	2.23	0.23	-85.8	-110.9	0.02	5640.6	100	Failure
Px3	Vert.cut @ 1st	XZ pl.@Grid 2	2.23	0.23	-84.7	-55.2	0.02	2807.6	100	Failure
Px4	Vert.cut @ 1st	XZ pl.@Grid 2	2.23	0.23	-34.7	47.9	0.02	2436.3	100	Failure
Px5	Vert.cut @ 1st	XZ pl.@Grid 2	2.23	0.23	-234.3	46.5	0.02	2365.1	100	Failure
Px6	Vert.cut @ 1st	XZ pl.@Grid 2	2.23	0.23	-96.4	-31.7	0.02	1612.3	100	Failure
Px7	Vert.cut @ 1st	XZ pl.@Grid 2	1.76	0.23	-124.4	-20.8	0.02	1340.4	100	Failure
Px8	Vert.cut @ 1st	XZ pl.@Grid 3	4.9	0.23	-209.8	-110.4	0.04	2555.5	100	Failure
Px9	Vert.cut @ 1st	XZ pl.@Grid 3	2.23	0.23	-212.3	174.3	0.02	8865.2	100	Failure
Px10	Vert.cut @ 1st	XZ pl.@Grid 3	2.23	0.23	-224.3	-81.2	0.02	4130.0	100	Failure
Px11	Vert.cut @ 1st	XZ pl.@Grid 3	2.23	0.23	-3.8	-127.3	0.02	6474.7	100	Failure

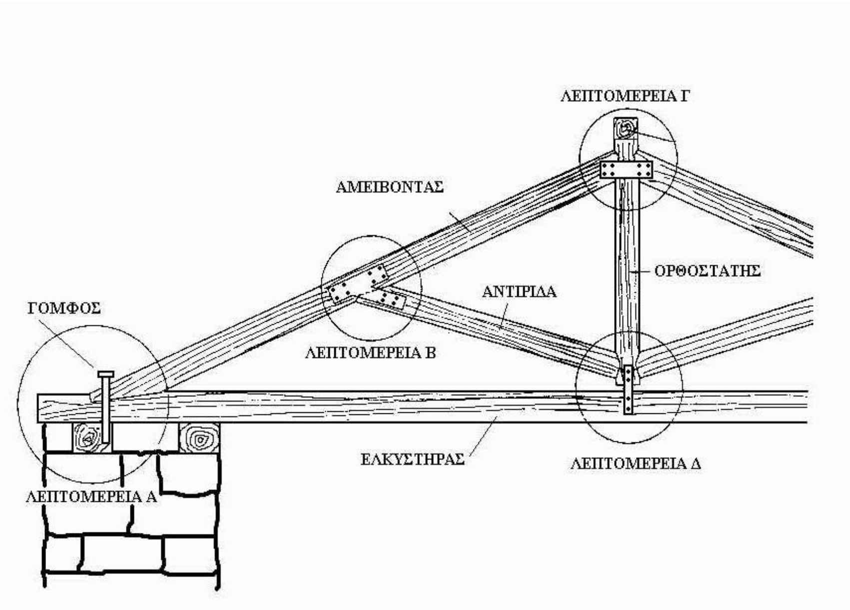
CHAPTER 6:

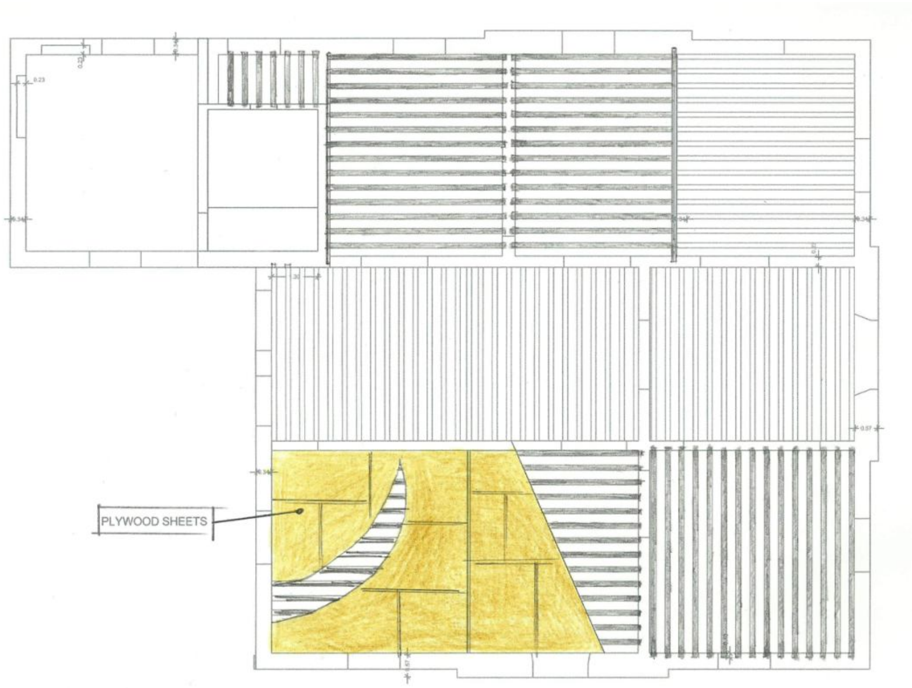
PROPOSED INTERVENTIONS

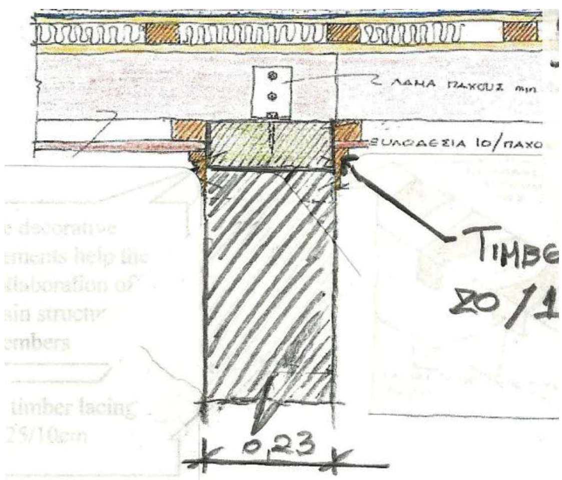
Given the present structural condition and the architectural or aesthetical requirements of a listed building, the interventions adopted were:

1. Enhancement of the diaphragm action at the roof level using light plywood sheets and
2. Placement of timber tie beams at the head of the internal and perimeter walls on upper floor.

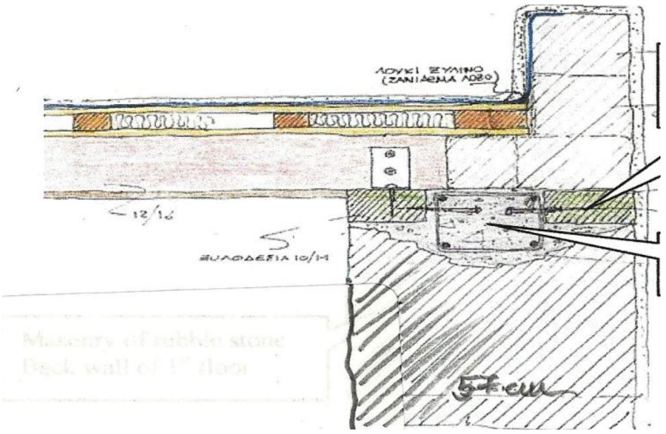
Some typical construction detailed sketches are shown below to illustrate the physical meaning of the application of the diaphragm action into the model. Comparison results will be shown on the full version of the thesis.



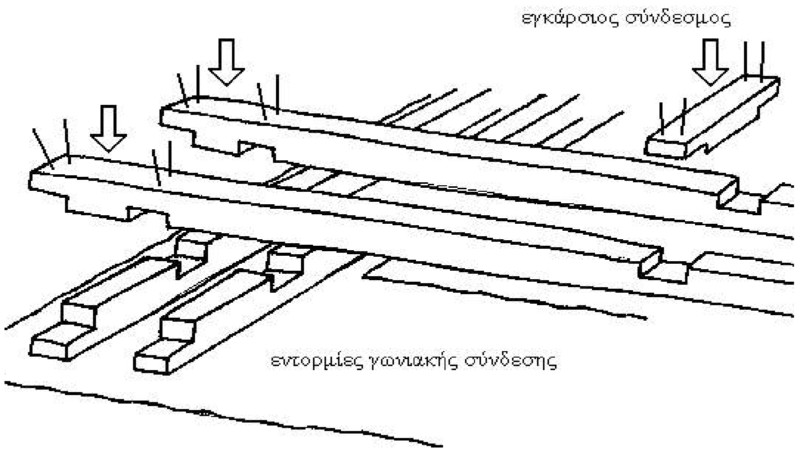




Detail 1 - Connection of timber roof diaphragm to internal walls



Detail 2 - Connection of the timber roof diaphragm to perimeter walls



Timber tie beam along perimeter

CHAPTER 7:

CONCLUSIONS – FURTHER WORK

The results derived from the analysis model were quite reliable and comparable with the observed damages. Further work will be done to examine some other parameters with different mechanical properties of walls. In addition, providing that the current thesis is a summary, another full detailed version with all results will be re-issued for the record.

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Last but not least, I am grateful to my parents for their continuous support during the postgraduate studies.

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[7] Greek Earthquake Resistant Regulation (E.A.K. 2003)

[8] M. Dimosthenous, “Methods and materials of restoration and strengthening of listed buildings of load bearing masonry walls”, T.E.E/T.K.M Edition, Feb. 2009.

APPENDICE I

PHOTOGRAPHIC RECORD:

1. M. XOFI (2011)
2. K. STYLIANIDI & CH. IGNATAKI (2004)

APPENDICE II

DRAWINGS

(Plans and Sections)