The analysis of a five-storey brick Masonry building "type" 77/5

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ABSTRACT

This paper presents the main results of numerical analyses carried out to evaluate the seismic response of an existing brick masonry building, *type 77/5*, built in 1977.(*fig. 1*)
This type is a representative of many other brick masonry buildings built in 1975-1990, in Albania.
The main reason why this building was chosen for analysis is: Its *floor plan derives from regular geometry, based on recommendations of the Eurocode 8* [6]. [**4.2.3.2 Criteria for regularity in the plan,** pg. 48, Part 1]

This study firstly intends to analyze and after to propose a way how to rehabilitate this type of building, if this result necessary after the analysis.
The study comprehended analysis based on 2 steps:

- a linear analysis, with help of finite elements model

- a nonlinear analysis, carried out with a simplified modeling procedure.

These numerical analyses refer to the strengthened building. The results showed that both procedures were useful to investigate the structural problems. The finite elements model furnished a good prediction of the masonry stresses under vertical loads and the modal response of the structure. [13]

The non-linear analyzes, with simplified method is performed based on the AM quake program. For this analysis a value of ag = 0.27 m / s2 was accepted.

The results of the nonlinear analysis are not the subject of this publication.

Key words: brick (9), building (19), masonry (11) wall (10), slab (4) concrete (4) seismic (7)

INTRODUCTION

Fig. 1 - The **3D** view of the building **77/5**

During the January 1988 earthquake, I used to live in a building type 77/5(Fig. 1). Although the damages that the building experienced were small, their locations were rather interesting. Based on the studies done thus far in seismic behavior of masonry buildings, the damaged areas were the same as those predicted for such buildings. That is the main reason why i choose to investigate this topic even further, utilizing the modern-day software advancements.

While this 5 storey building makes a considerable percentage of the residential buildings, all over Albania, the basic question is: "Are safe these buildings under seismic actions, to be housed from so many families?"

The structural stability of existing masonry buildings is a topic of great interest, notably in the light of evolution of technical regulations, i.e. the continuous improvements that have been made in the theory of masonry buildings.
This raises on the one hand, the choice of conservative techniques for the reinforcement of these inhabited buildings, and on the other hand the development of adequate numerical procedures for their seismic verification.
Different models for the assessment of masonry structures exist in the literature: they are one-dimensional (frame or macro-element) and two-dimensional (finite elements).

Among these, those based on finite element modeling and those that use simplified macro-element models are of particular importance.

- The finite element method offers numerous possibilities for modeling all structural cases, however, nonlinear analyzes are particularly cumbersome from the point of view of computation and the results reading.

The poor tensile strength of the masonry does not allow the direct use of elastic models linear for the prediction of the response and the damage of a building, subject to seismic actions. [12].

From this perspective, the use of a finite element model in the linear field, ETABS model, developed *by Computer and Structure Inc.*, in our case, appears interesting, for study of the stress state, under the action of static loads and the modal behavior of the building.

- The one-dimensional ones are based upon a simple approach which includes models that schematize the structure as an equivalent frame. The first frame model was proposed by Tomazevic (1978) and it is the well known POR method [9], where the masonry walls are schematized by a set of piers connected by a rigid spandrel.

Many of the macro-element methods developed as evolutions of the POR method, are based on an incremental iterative procedure (non-linear static analysis) in which, the seismic load, is evaluated for the **collapse of the building**. Among them are 2 programs that we used in our analyzes, such **AM-Quake** and **Atena**. They perform non-linear dynamic analysis, with step by step technique, for masonry buildings with rigid floors in their own plane and congruent with the walls.

MATERIALS AND METHODS

The main purpose of this study is to analyze the effect of seismic action on the sustainability of buildings, type 77/5 [11]

The main building material is **brick masonry**.

The bricks are clay bricks, **Class = 7.5 N/mm²**

Cement mortar is **Class = 2.5 N/mm²**

The ground and the first floor walls of the building are 38 cm thick. The second till fourth floor walls are 25 cm thick.

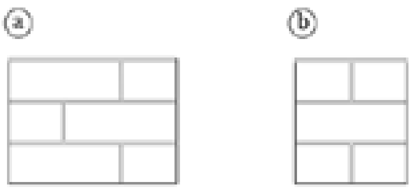

Fig. 2 - Masonry sections: **a**- wall 38 cm ; **b** – wall 25 cm

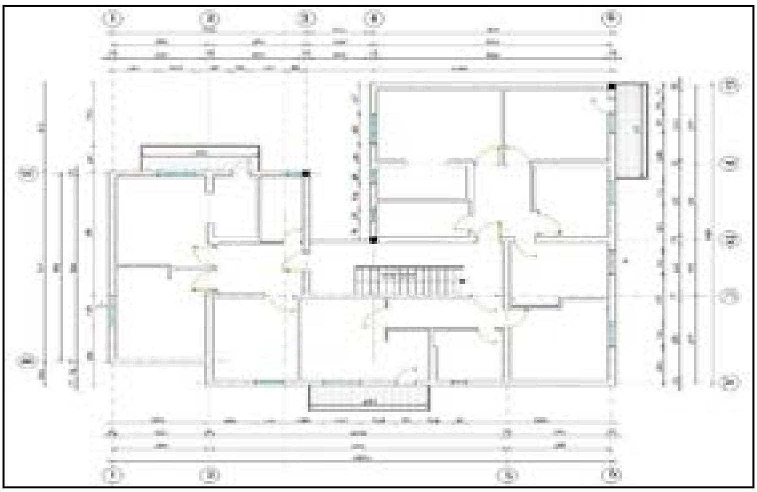

Fig. 3 Ground floor plan [11]

Based on above materials, bricks and mortar, Table 1 give this *Resistance of masonry:*

$$f_k = 1.1\ N/mm^2$$

Tab.1 Resistance in pressure, $\mathbf{f_k}$ of masonry [5]

Nr	*Brick class*	*Mortar class (N/mm2)*						
	N/mm2	***10***	***7.5***	***5.0***	***2.5***	***1.5***	***0.4***	***0.0***
1	***15***	***2.2***	***2.0***	***1.8***	***1.5***	***1.35***	***1.2***	***0.8***
2	***10***	***1.8***	***1.7***	***1.5***	***1.3***	***1.1***	***0.9***	***0.6***
3	***7.5***	***1.5***	***1.4***	***1.3***	***1.1***	***0.9***	***0.7***	***0.5***

The intermediate floor slabs are type Zoellner, cross section shows in Fig.7. They are composed of bricks height 15cm, filled every 20 cm with concrete, width 8cm.

The concrete grade is accepted relatively low, C15/20, due to the bad quality of raw materials at that time.

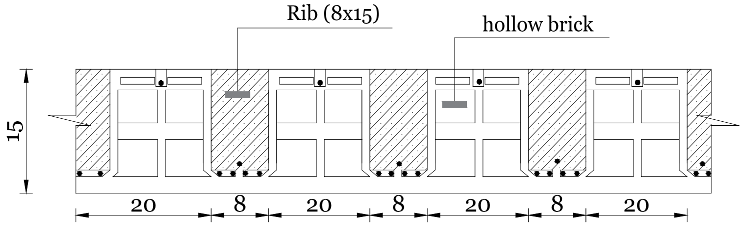

Fig. 4 Slabs cross section

In Etabs these slabs are converted in secondary beams, they transmit the load in one direction, in X or Y respectively.

The foundations of these buildings are stone walls, with cement mortar, a significant weakness, especially if we consider the preparation conditions and the quality of participant materials, especially the cleanliness of sand and gravel, at that time. I base this on my several years of experience in construction site. The foundations effect on building safety belongs to another analysis.

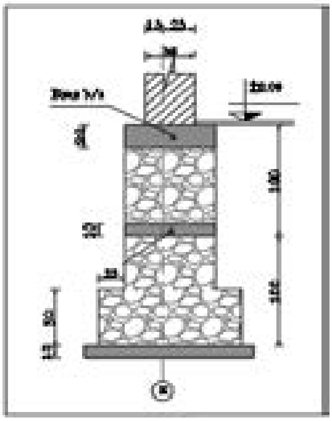

Fig. 5 Typical foundation section

The masonry elasticity modulus **E,** for serviceability conditions, in **EC6** is recommended **E= 1000 f_k** N/mm2, while for the calculation on the last limit state (mainly in **nonlinear analysis**) is recommended to use the value **600 f_k N/mm2**. [6]

From various comparisons with experimental values, (Tomazevic 1999) results that:

- "**Recommendations in Eurocode lead to overestimation of the modulus of elasticity**".

Author Thomas Zimmermann* (Zimmermann, et al., 2012) recommends the following equation as closer to experimental values [5]

$\mathbf{E = 300\ f_k}$ [N/mm2] = 300 *1.1 = **330 N/mm2** **(**in nonlinear analysis)

RESULTS AND DISCCUSSION

It should be noted that **KTP.N.2-89** (Technical Design Conditions, publication of the Seismological Center, Tirana) recommends some essential limitations for the floor plans of the buildings. Thus, in fig 1, the dimension "**c**" must respect the condition: $c < 0.25$ B. [7]

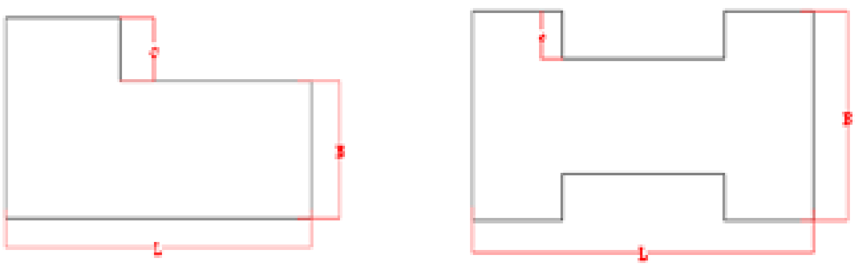

Fig 6- Illustration of KTP-N2 recommendations

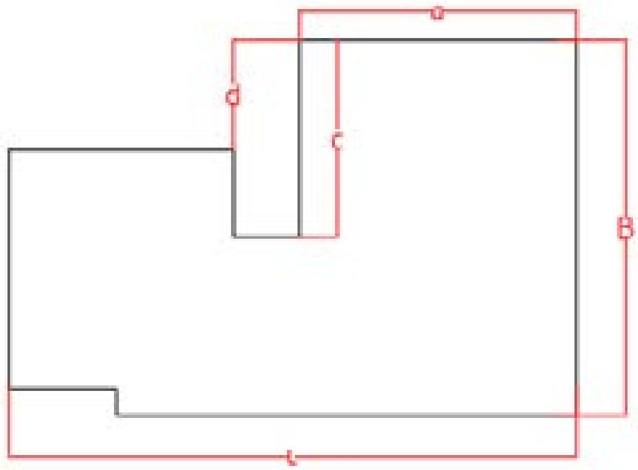

Fig. 7 Planimetric form of building 77/5

The main dimensions of the building are: L = 18.60; B= 14.24; c = 7.45 ; d= 4.15

Limitations of KTP: d/B = 4.15/ 14.25 = 0.29 > 0.25, but: 7.45/14.25 = 0.52 >>0.25 !!!

- The recommendation 4, page 11 of KTP-89 states that, if the condition **e/L < 15%** is met, the eccentricity is considered insignificant, where **e** is the eccentricity in one direction, i.e.:

  e = (10.21- 8.38) = 1.83 m and in our case, we have: 1.83/18.65 = **9.81%.** The condition is met.

- Let verify **[EC 8, pg.48]**: **4.2.3.2 Criteria for regularity in the plan:**

“for each fracture, the surface that is included between the contour of the intersection and a convex polygonal line, does not exceed **5%** of the intersection area.

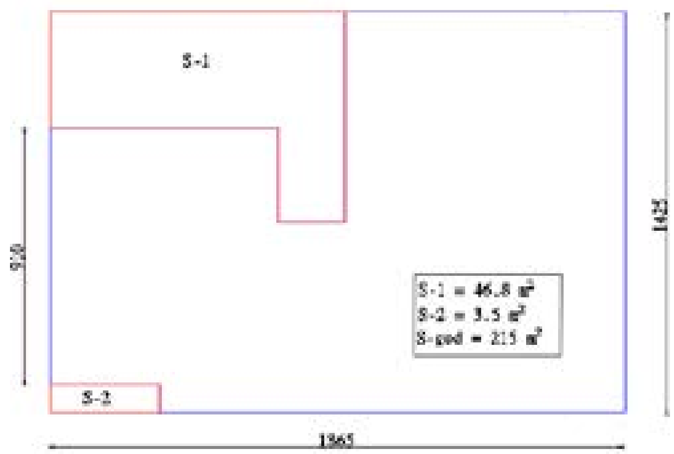

Fig. 9 The missing areas (1,2,3)

The missing areas to the total (full rectangular shape), is **51.4m2**, while the total area is 265.3 m2.

In percentage, the missing area, to the total is: 50.3 / 265.3 = **19 %!!**
Criterion is not respected, the value exceeds 5**%**
The mass and the gravity center are defined, and they are as below:

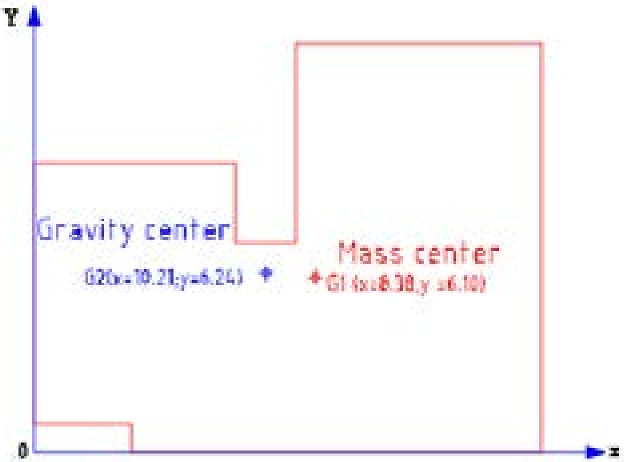

Fig.8 Mass and gravity center

Gravity center	**Mass center**
X_C = 10.21 m	X_C = 8.38 m
Y_C = 6.24 m	Y_C = 6.10 m

THE ETABS DATA INPUT

Loads

After calculation, the slab dead load is **200 kN/m²**.

Based on the EC, we accepted these loads:

Live = 200 kN/m2; Additional dead load = 200 kN/m2

Also, based on the Department of Seismology, Institute of Geosciences data, for the case of Tirana land, is accepted :

- land type - category C,
- acceleration ag = 0.25g.

Seismic data

Since we want to analyze the most unfavorable case, we choose from the type of elastic response spectra, the type "1" of the earthquake, based on the EC recommendation [6], with magnitude MS > 5.5. So, we used spectrum type 1, the masonry ductility factor **$q = q_0\ k_w \geq 1.5$**, and **3%** extinction. [6]

Below we present the tables with the data that have been entered in the Etabs program, on the basis of which have been obtained the respective results.

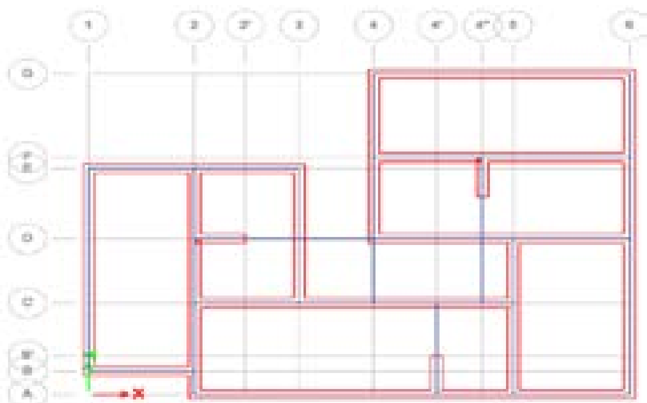

Fig. 10 - Plan type, input in Etabs

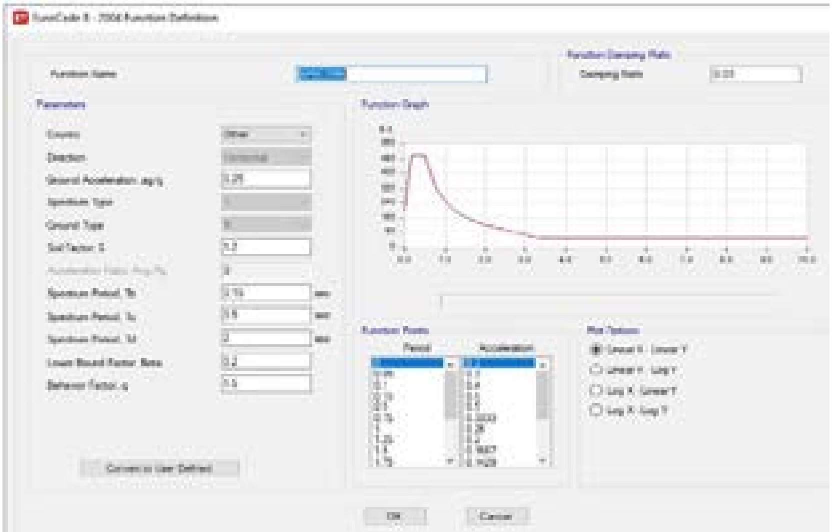

Fig.11- Seismic spectra, periods and accelerations

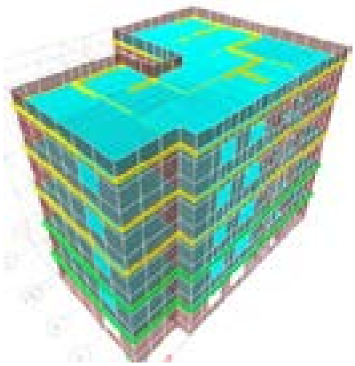

Fig. 12- Etabs 3D

(Yellow- concrete parts)

Fig. 13 - Etabs brick data

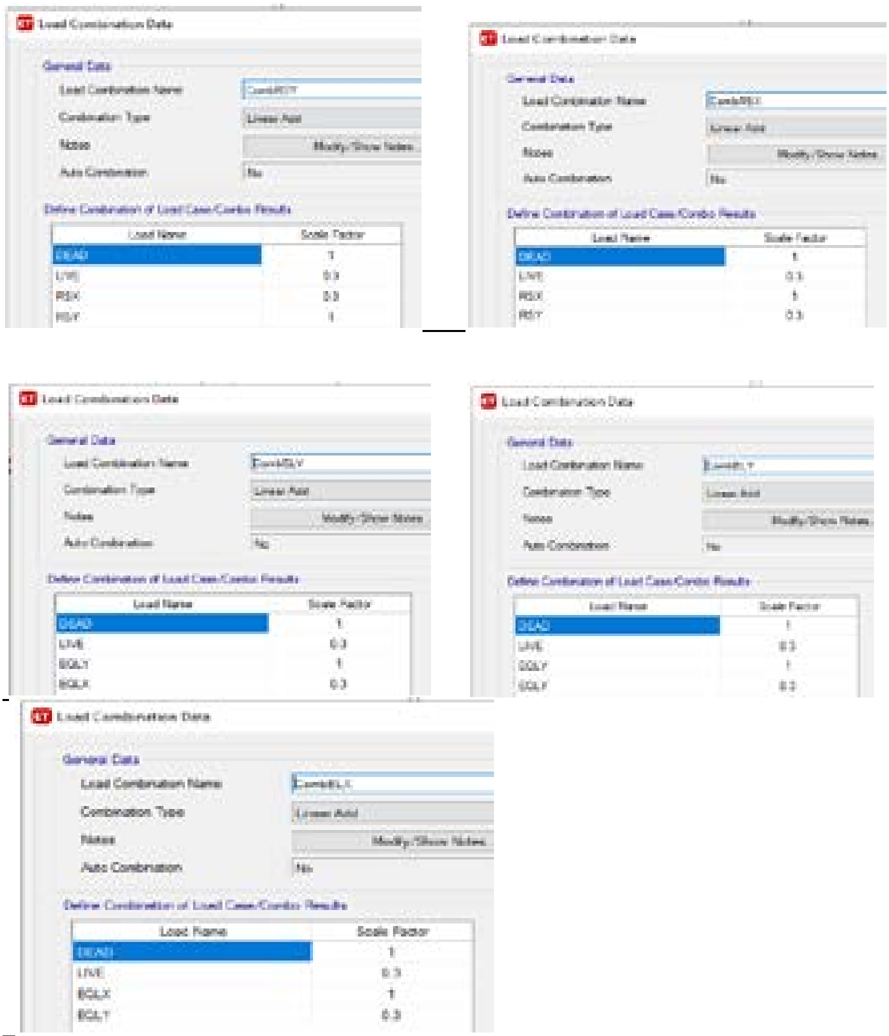

Fig. 14 Etabs main load combinations

The period of personal oscillations, according to the recommendations of EC 8 should be:

$$[T] = 0.05 \times Hg^{0.75} = 0.05 \times 14.2^{0.75} = 0.366 < T = 0.728 \text{ s !!!}$$

where **0.728 s** is the period in the first form of oscillation.

LOCAL CONTROLS:

Based on the Etabs analysis we have selected the walls with the greatest stresses and deformations. Some from the stresses and deformations results, which exceeds the allowed values are presented below, through respective screenshots.

Looking at the stress diagrams, we see that, the **upper area**, [upper floors], suffer mainly under the effect of tensile stresses, while the lower part (1-3) mainly, is under the effect of compressive surface stresses.

(The blue arrow indicates the analyzed elements)

AXIS 2-2

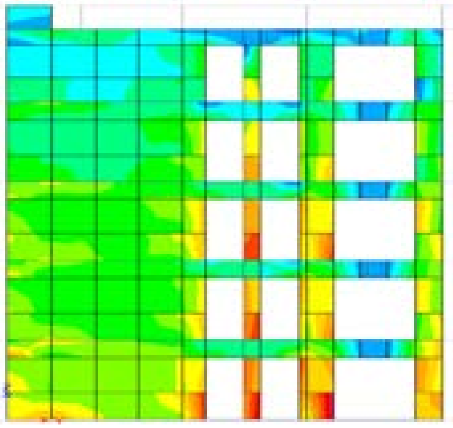

Fig. 15- Axis 2-2 elevation. The blue arrow indicates the most stressed element W828

Fig. 16- W828 element detached from axis2-2

Fig. 17- The analyzed stress **s**2-2 in W88 element

The max value on this axis is the compression **s**2-2, with value = -**1.46 MPa** > **- 1.1MPa**

AXIS 6-6

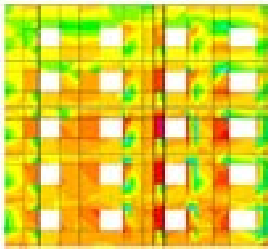

Fig. 18 - Axis 6-6 elevation. The blue arrow indicates the most stressed element W559

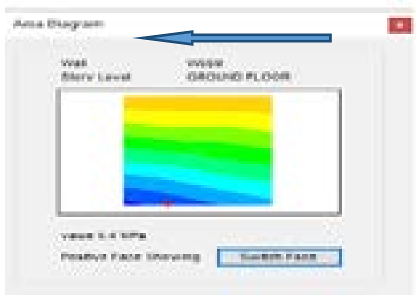

Fig. 19- Analyzed stress **s**1-1 of element W559 **Fig 20** - W88 element detached from axis2-2.

The max value on this axis is the traction **s** 1-1 with value = 0.4 MPa

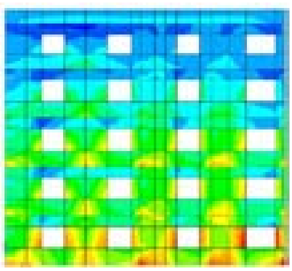

Fig. 21 - Axis 6-6 elevation. The blue arrow indicates the most stressed element **W1039**

Fig 22 – W1039 element detached from axis2-2. **Fig. 23 -** Analyzed stress **s** 2-2 of element W1039

The max value on this axis is **s** $_{2\text{-}2,}$ in compression, with value = -1.42 MPa

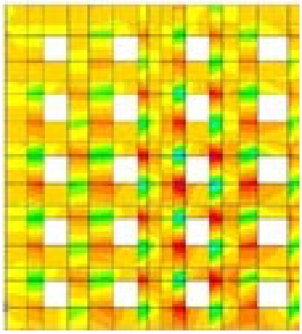

Fig. 24 - Axis 6-6 elevation. The blue arrow indicates the most stressed element W1053

Fig. 25 - Analyzed stress **s** 1-1 of element W1053 **Fig 26 –** W1053 element detached from axis 6-6

The max value on this axis is **s**$_{1\text{-}1,}$ in traction, with value = 0.79 MPa

<u>AXIS 6-6 Deformation</u>

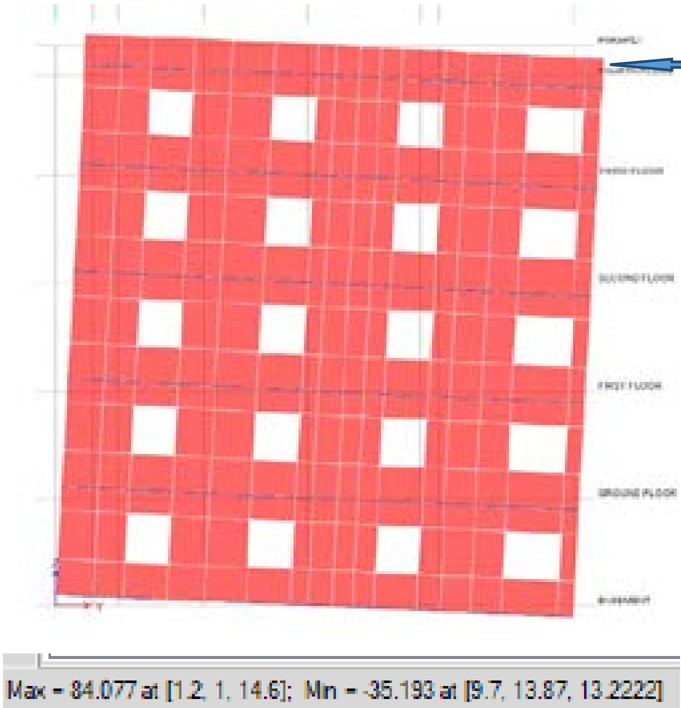

Fig. 27 - Deformed shape of axis 6-6, for EQLY combination

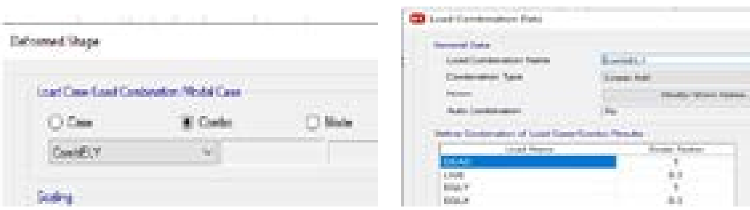

Fig. 28 – The ETAB s ELY combination, and its components

Max displacement = **8.41 cm > > 4.73 cm**

The recommended allowed displacement of the building on the top must be 1/300 H = **4.73** cm

CONCLUSIONS:

The building presents these main problems:

1- Its period 0.728 sec and not 0.336 as recommended by EC8 for masonry buildings
2- The building has mural elements, in which the values exceed [**s**] pressure = - 1.1 MPa
3- The most problematic are the elements that suffer in tractions, since the masonry is very sensitive to it. Thus, the allowable tensile values for masonry are [**s**] pressure = 0.1 MPa, while in the whole masonry of the building, tensile stresses greater than 0.1 MPa occur. This is also the main weakness of the building, which requires reinforcing surface interventions throughout the masonry.
4- Also, the building has significant displacements, which exceed those allowed

However, the next steps of analysis, (the non-linear ones) will highlight the other weaknesses of this building.

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