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Some general considerations about the behaviour and retrofitting solutions for the existing buildings with masonry structures

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Summary

Being in countries with high seismic risks and vulnerability or mining subsidence we have the legacy of an existing buildings stock (with masonry and gravitational frame structures) which must became safety from all the view points.

Depending of each historical period of erection, buildings and structures are tributary to the knowledge level of that time including small or big mistakes, or spectacular technical solutions.

Passing from country to country, especially in Europe, you may observe a lot of similarities between architectural époques or structural styles, especially because the gravitational rules were imposed, using French, Italian or German Design Codes, more or less with the same content.

This may be a worse point but also an interesting idea, because being anywhere in Europe (Germany, France, Italy, Romania, Greece or Poland) for the same historical periods you may find the same architectural style of buildings and the same design for their structures. More or less it mean that will be a good reason to extend some of the information obtain for a country to the others, taking into consideration the each item.

In the followings, only the buildings with masonry structures were considered in the idea to optimize the possible retrofitting solutions.

Incremental-iterative procedures were used for approximate assessment of the response of unreinforced masonry buildings to seismic loads. Working within the framework of the shear-type structural behaviour, we propose a constitutive model for the resisting elements able to account for both the variation in lateral stiffness induced by cracking, and their sort of failure (i.e. ductile or brittle). Application to a practical case demonstrates how, when modelling walls with generic windowing, the manner in which they are subdivided into smaller resisting elements strongly influences the predicted structural response and the manner of failure of each wall.

KEYWORDS: masonry structures, structures, earthquakes, ductility, stiffness, strength, deflections, drifts

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1. INTRODUCTION

Almost in the entire Europe the existing buildings stock may be divide in several typological groups as followings:

- Very old buildings (included in the national heritages) with timber and/or masonry (stone or solid bricks) structures made without any rules excepting the traditions transmitted as an experience from masters to masters;
- Buildings erected until the very beginning of the 20^{th} century, without specific design codes, with timber and/or masonry structures which sometimes may include also structural elements from simple or reinforced concrete;
- Buildings erected during the world wars, designed with very poor codes (only gravitational) with masonry and gravitational frame structures;
- Buildings erected immediately after the second world war in concordance with some seismic pre-codes with masonry and frame structures;
- Buildings erected during the 70's period in concordance with some former seismic or mining subsidence design codes(actually passed as knowledge – technically and scientifically) with masonry and frames structures (cast in place or pre-cast technology);

Each typological group consist in a large number of different possibilities and masonry structures group is not different from this point of view.

Is very difficult for a small team to carry out in a short time all these possible cases and in the followings just a part of the entire entity is presented.

In these papers our attention was focused only to the masonry structures, included in the 2^{nd} typological group of buildings, previously presented.

Depending of the computational tools of each period, after about 20 years of work and about several hundred case studies of different building types (from different typological groups) the authors attend to find some general considerations about the behaviour of the buildings with masonry structures and their optimal possible retrofitting solution. In this trend several idealised study cases were used.

2. STATE OF THE ART AND OF THE PRACTICE FOR THE DESIGN PERIOD OF THESE TYPES OF BUILDINGS

For the preparedness of the idealised study cases, the state of the art and of the practice tributary to that period were considered:

- These buildings present a lot of irregularities including architectural and structural ones, in the plan or in the elevations;

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- The masonry structures, in parallel with the gravitational frame structures are probably the most irregular cases.
- From the general considerations the masonry structures may be:
  - Unitary – meaning practically one building, made from the beginning and kept in this shape;
  - Attached – meaning that in time the owners (different or not) attached in plane (other bodies of the building) or on the vertical (other levels);
- From the structural point of view the masonry structures may be:
  - Optimum developed in plane and space – with similar stiffness characteristics on each principal direction and on the vertical;
  - „in lane or wagon type" buildings – where usually there is a huge difference between the stiffness characteristics on principal directions.
- Without any specific design tools, masonry structures were more or less the architects appendage. The masonry walls thickness were generally in accordance with the solid brick dimensions (7x14x28 cm) made from a good class of bricks (about C50-C100) and a worse class of mortar (between M4-M10);
- There was not any computations for these kind of structures, excepting the centric compression stresses;
- Usually there are not any small columns or ties in these masonry structures meaning that we may consider a URM solution;
- The coupling beams (lintels) are from masonry and timber or steel at the bottom;
- The considered slabs thickness was about l/40 (where l=minimum dimension of the slab, from both directions);
- Always the foundation soil was considered with a conventional pressure between 200-300 KPa.

3. STUDY CASES FOR THE BEHAVIOUR OF THE EXISTING GRAVITATIONAL FRAME STRUCTURES AND THEIR POSSIBLE RETROFITTING SOLUTIONS

Taking into consideration all this "rules" (well known) 48 idealised study cases were carried out to obtain some general considerations about the masonry structures. The idealised cases consist into one exterior masonry resistance line (one way frame) with 2 equal spans of 500 cm) with the tributary areas for masonry piers and coupling beams and also for the level weights.

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The used parameters were the levels number and the horizontal acceleration (PGA). Three cases of PGA were considered (0.16g, 0.20g and 0.25g) and three types of building behaviour (masonry piers with virgin coupling beams, masonry piers with double hinged coupling beams and masonry piers with middle span hinged coupling beams). For the levels number 6 variants were considered, between 1-6 levels, with a level height of 300 cm).

All the beams were considered with 28x140 cm transverse section and for the piers were considered two different sections: a „L" section for the both marginal piers and a „T" section for the median one, with 28 cm thickness.

The chosen seismic excitation was INCERC NS 1977 scaled for different types of PGA (as mentioned before).

In order to assess the seismic response of simple masonry buildings, one of the most widely used structural models is the shear-type, in which floors behave as rigid diaphragms elastically connected by interposed walls. If the vertical displacements of the structure induced by horizontal actions are ignored, the model's degrees of freedom are extremely reduced. In such a case, the quality of the information obtained by applying the model depends upon the accuracy of the assumptions regarding the behaviour of the single walls. In this regard, satisfactory descriptions can be provided of the walls' behaviour during the brief initial loading stage (phase-I), where each results uncracked and behaves elastically (Figure 1, line O-A), as well as when they reach the corresponding limit condition (line A-B). On the other hand, very little is known about the subsequent stage (phase II).

If the resisting element (either a wall or a masonry element of walls with openings) is restrained well enough to its bases and is not excessively thin, the cracking which forms where tensile stress is at a maximum ceases almost immediately, and a new elastic phase may precede ultimate failure (line B-C). Under such conditions, the state of stress begins to grow again and reach values well above those of the preceding stage. Finally, collapse of the element may come about through brittle failure (line C-D), if, that is, the compression exceeds the material's strength, or, more frequently, through 'slow frictionful slipping of one part over another (line C-E), in which case, the failure turns out to be ductile.

In the transition from phase I to faze II, the element's stiffness undergoes an abrupt decline; thus, the stress distribution among the elements may vary considerably, with consequent worsening of the state of stress in the as yet uncracked elements. Such a situation favours a more uniform stress distribution throughout the entire structure while this is still behaving elastically and wall cracking is not evident. Greater uniformity of the stress distribution may delay failure of the elements. In any case, their manner of ultimate failure, either ductile or brittle, depends on the material's strength parameters and on the panel-aspect ratio.

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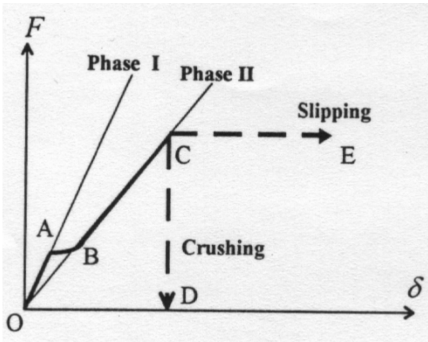

Figure 1 – Two-phase behaviour

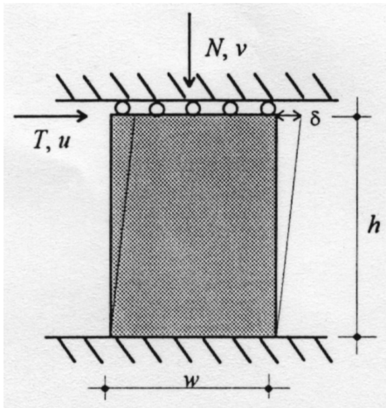

Figure 2 – A masonry panel under compression and shear

With the aim of accounting for the essential features of both phenomena described in the foregoing, in the following we propose a simple mechanical model able to reproduce the phase-II elastic equilibrium state attained in rectangular masonry elements. This permits the overall seismic response of a building to be deduced via standard incremental procedures. In doing so, each of the walls with generic windowing is treated as an assemblage of elements, some of which may be cracked. In conclusion, the main characteristics of the resulting algorithm are demonstrated in an example application.

A rectangular panel of unreinforced masonry is fixed at its bases, the other sides being load-free (Figure 2). Let $\boldsymbol{w}$ be the width, $\boldsymbol{h}$ the height and $\boldsymbol{t}$ the thickness, assumed constant and small with respect to $\boldsymbol{w}$ and $\boldsymbol{h}$. The panel is subjected to a relative displacement δ between its bases and parallel to them.

Faze I -In the absence of perceptible cracking, the stress distribution in the panel can be determined, within a technically acceptable margin of error, through linear elastic analysis by disregarding the effects of the material's dishomogeneities and anisotropies. In particular, by extending the beam model to the panel [1] and denoting by $\boldsymbol{E}$ and ν the material's Young's modulus and Poisson's ratio, the lateral stiffness results to be

$$K^{I} = \frac{Et}{[2\chi(1+\nu)+\eta^{2}]\eta} \tag{1}$$

Where χ is the shear factor (6/5 for a rectangular-shaped cross-section) and $\eta = \boldsymbol{h/w}$. In order to account to some extent for the actual degree of constraint, a

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reduced value of K^I is usually assumed for masonry elements located near the top of the building.

Phase II - When cracking becomes evident, the simplest mechanical model for assessing the stress state in the panel is represented by the graph in figure 3 below.

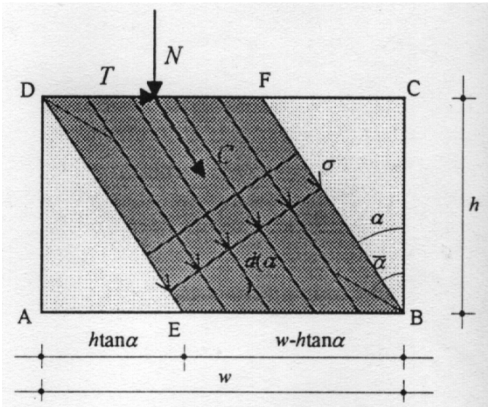

Figure 3 – The load-bearing zone

In particular, it is assumed that the reactive elastic zone is made up of the parallelogram E-B-F-D, whose dimensions for given values of w and h are determined solely by angle α. Within the zone, the state of stress is constituted by a uniform compression σ, with resultant $C(\alpha) = \sigma t\, d(\alpha) = \sigma t\,(w\cos\alpha - h\sin\alpha)$. The triangular regions A-E-D and B-C-F are instead held to be stress-free. The lower the material's tensile strength, the closer such a model will conform to reality. In order to determine α, we assume a value such that the lateral stiffness of the reactive zone is at a maximum:

$$K^{II} = \frac{E_t \sin^2\alpha \cos\alpha(\cos\alpha - \eta\sin\alpha)}{\eta} \tag{2}$$

Thus, angle α coincides with the maximum point of the function $f(\alpha) = \frac{\sin^2\alpha\cos\alpha(\cos\alpha - \eta\sin\alpha)}{\eta}$, with $o \le \alpha \le ά$, where □$= \arctan(w/h)$.

Figure 4 shows how $\alpha(t)$ is monotone decreasing between values 45° and 0°. Figure 5 instead plots the stiffness of the wall elements in phases I and II, from which it can be seen how the loss of stiffness is always significant, as long as $\eta < 2$.

In deducing the interaction curves for the panel, we consider only cases in which failure occurs either by crushing of the weaker masonry components, or by slipping. Denoting by $N=C\cos\alpha$ and $T=C\sin\alpha$, the vertical and horizontal components of C, respectively, and by v and u, the corresponding displacements (Figure 2), the interaction curves traced in the N-T plane are deduced by recourse to the static theorem of limit analysis. In doing so, we first deal with elements for which N can be considered a constant, and then an element for which v is zero and N is unknown.

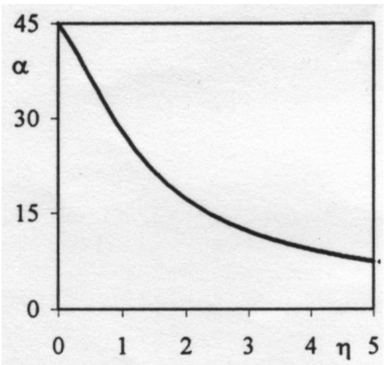

Figure 4 – Angle α versus ratio ***h/w***

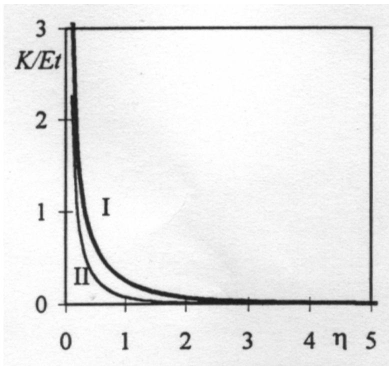

Figure 5 – Panel stiffness

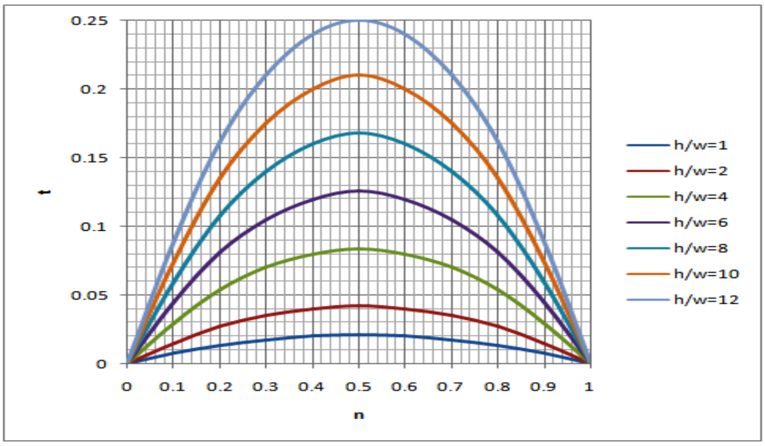

Figure 6 – Masonry crushing interaction curve for f_m=50 tf/m^2

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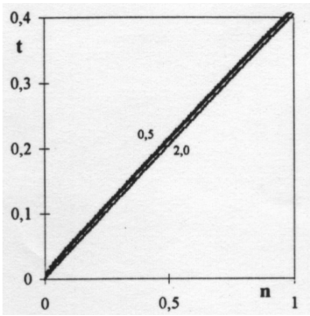

Figure 7 – Interaction curves for slipping

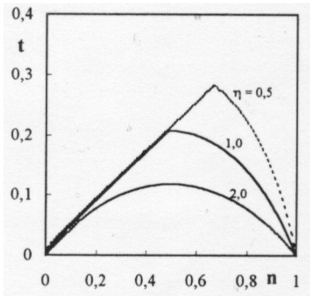

Figure 8 – The limit strength domain for generic masonry panels

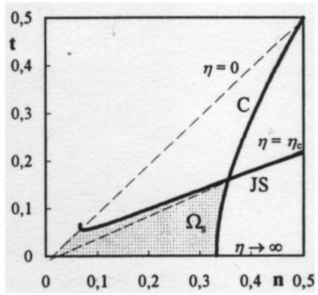

Figure 9 – The limit strength domain for maximum-stiffness panels

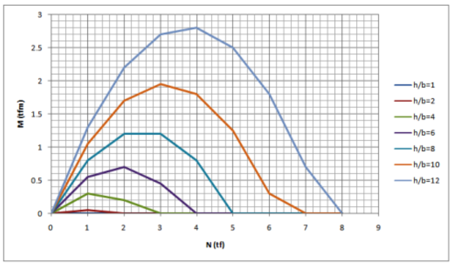

Figure 10 – Interaction curve N-M for fm=1 tf/m^2

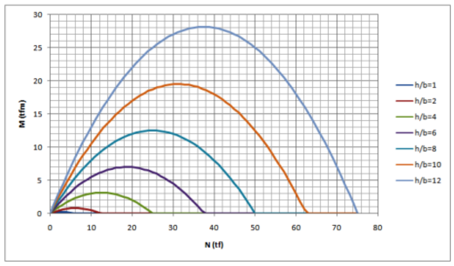

Figure 11– Interaction curve N-M for fm=10 tf/m^2

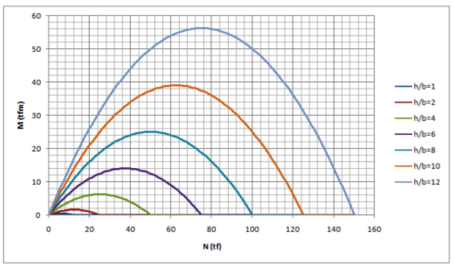

Figure 12– Interaction curve N-M for fm=20 tf/m^2

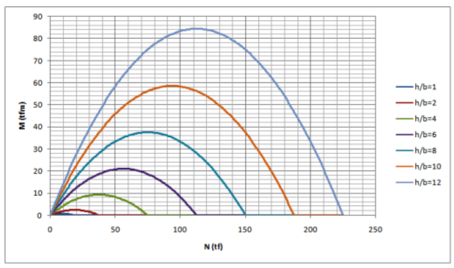

Figure 13– Interaction curve N-M for fm=30 tf/m^2

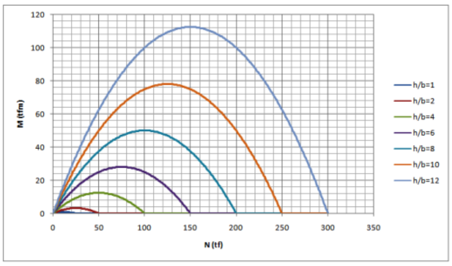

Figure 14– Interaction curve N-M for fm=40 tf/m^2

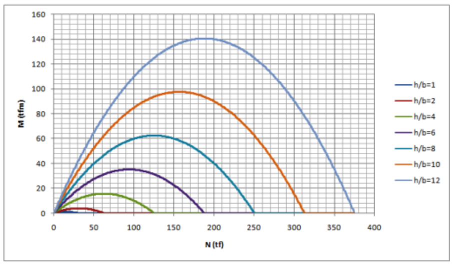

Figure 15– Interaction curve N-M for fm=50 tf/m^2

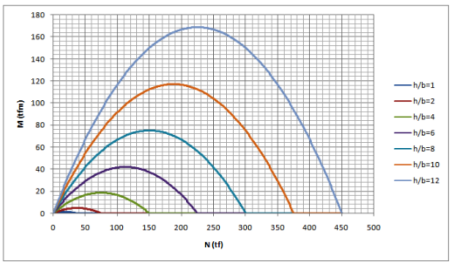

Figure 16– Interaction curve N-M for fm=60 tf/m^2

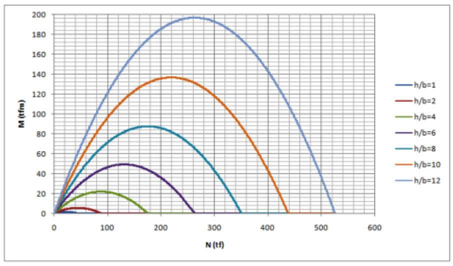

Figure 17– Interaction curve N-M for fm=70 tf/m^2

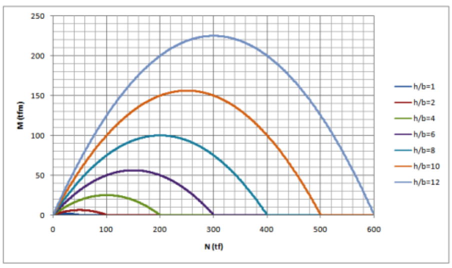

Figure 18– Interaction curve N-M for fm=80 tf/m^2

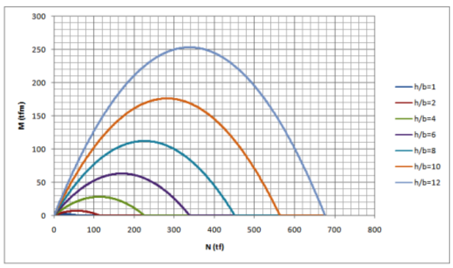

Figure 19– Interaction curve N-M for fm=90 tf/m^2

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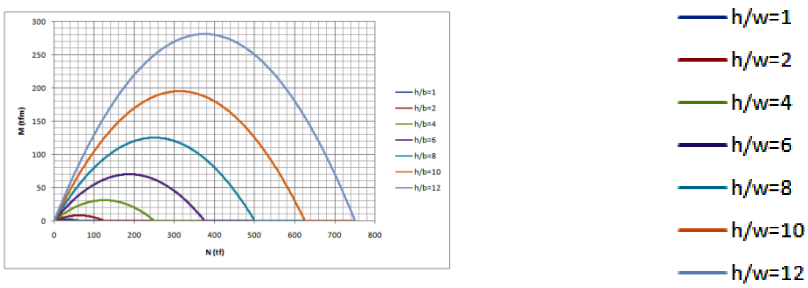

Figure 20– Interaction curve N-M for fm=100 tf/m^2

One type of retrofitting solution using RC jackets on both sides of the piers and coupling beams was accepted.

Of course there are also other theoretical retrofitting solutions (which will be study in the following stage) like: steel systems, base seismic isolators (almost all the time is not enough to use these kinds of dampers without to execute some retrofitting operation also at the superstructure), FRP systems and RC frame system implants.

For the existing structure, in each type of case the gravitational and seismic safety were verified:

- Gravitational safety

The cross sections for the masonry piers were determined in concordance with the state of the art and state of the practice for that period, like a "correct" designer. In this situation, for all the study cases the masonry piers offer gravitational safety and from this point of view there is not necessary to take supplementary measures.

- Brittle failure mechanisms: $t \geq 0.5$ with $t=\tau_o/R_t$ and $\tau_o = Q/A_{shear}$

For all the study cases we looked always both at the strength capacity and the stiffness of the building structures.

From the stiffness point of view starting with the proper period of vibration (figure 21) and ending with the drifts (figure 22 a, b and c) we observe that the lateral stiffness of the existing structures is very rich. As a consequence the drifts are normal or in the normal limits.

The masonry structures, because of the structural walls offer always a medium to large lateral stiffness for the horizontal actions, but not also enough strength. So for these type of structures usually all the problems are coming from the strength capacities.

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The solution of RC jacketing of all the masonry structural elements, without to take any other measures seems to be adequate or reasonable for low and medium rise buildings (1-6 levels) only, because as we know the masonry structures are conceptually stiff structures.

In figure 21 are presented the fundamental period of vibrations for the existing structures and for the retrofitting case studies.

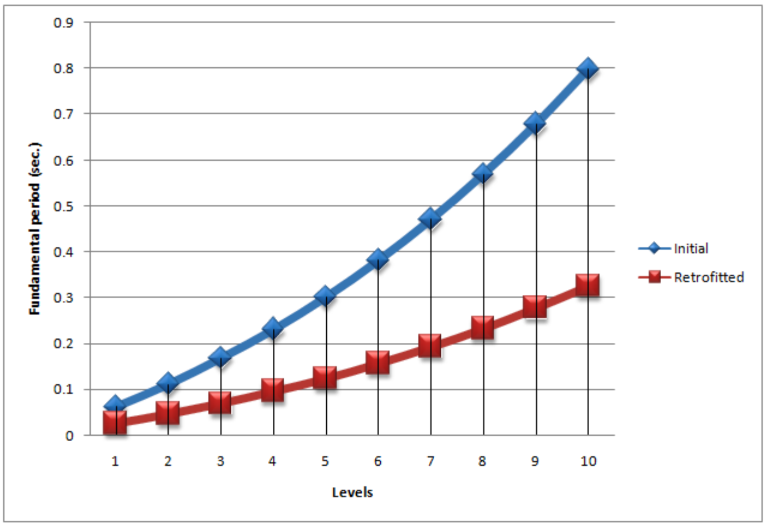

Figure 21 - Fundamental period of vibrations

The ductility demands highlight that for PGA less than 0.25g there will be not any plastic hinges in the masonry piers. Increasing the number of the levels, plastic hinges will occur in the masonry coupling beams.

This was exactly the reason to choose 3 series of models. Depending of the ratio between the height and the length of these masonry coupling beams, the occurrence of plastic hinges (which mean practically severe"X" cracks) may be at both ends or in the middle spans of these coupling beams. In this situation the immediate behaviour of the initial uncraked structure will be like the considered models with plastic hinges at the edges or in the middle span.

For the retrofitted model we considered a RC jacketing at one or both faces of the masonry piers and coupling beams and to rehabilitate the initial model with functional coupling between the masonry piers.

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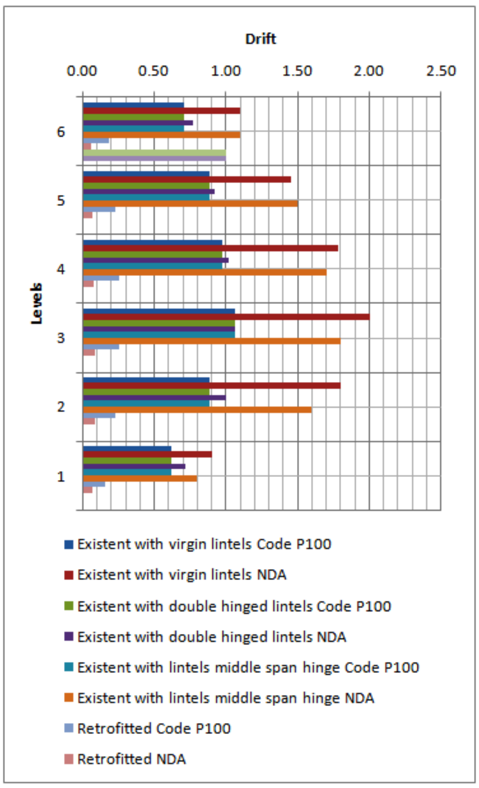
In the figures 27-28-29 (a, b and c) there are presented base seismic shear coefficient and both C_{BNL}/C_{BL} and Safety indexes (NL=non linear, L=linear and B=base) for all the study cases, for all three seismic intensity values.

The C_{BNL}/C_{BL} increase with the seismic intensity increasing and always the lowest value is for the middle span plastic hinges models in time when the higher values are touch by the double hinged coupling beams models.

Similar commentaries concerning the safety index which have the values less than 0.500 always for the existing situations.

And generally speaking the best behaviour is of the RC jacketing structural walls retrofitting solution.

Drifts (‰)



Deflections (cm)

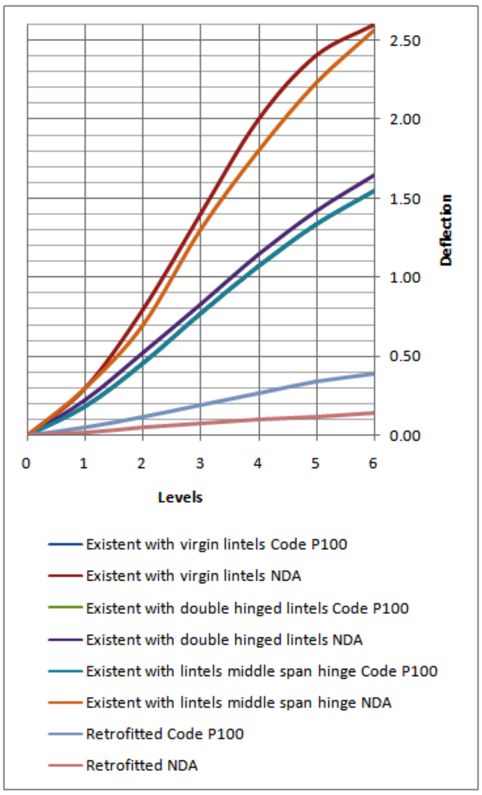


Figure 22a – Drifts and deflections for PGA=0.16g

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Drifts (‰) Deflections (cm)

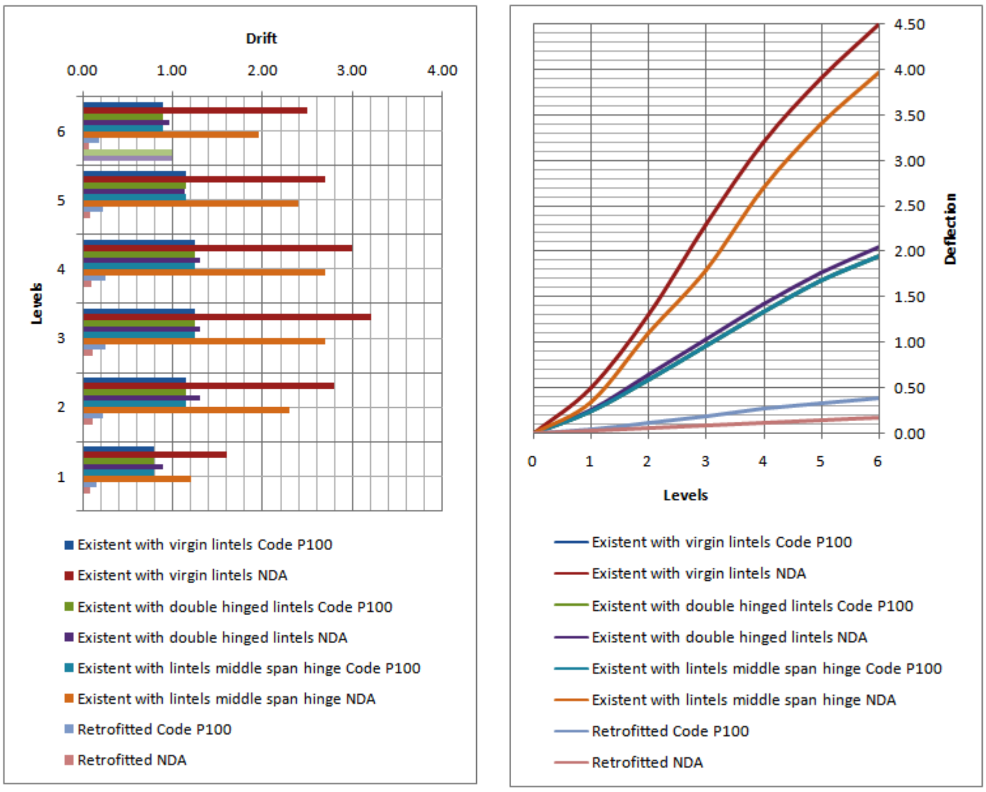

Figure 22b – Drifts and deflections for PGA=0.20g

By applying the shear-type model, determination of the structural response of a building to seismic loads can be performed via sub-structuring techniques, by which the main structure is comprised of floors and the walls are secondary. In such a framework, in order to account for possible openings in the walls, each wall can be thought of as an assemblage of smaller rectangular elements, each of which satisfies the constitutive equation illustrated in Figure 1 and the limit strength conditions deduced in the foregoing. Finally, with regard to the transition between the two phases, although the cessation of uncracked stage should more properly be evaluated in the context of fracture mechanics theory (in order to better account for failure of the brick-mortar interface), here for the sake of simplicity, considering the extremely low levels of stress, it was assumed that cracking starts when the maximum tensile stress component reaches the material's flexural strength.

Under the hypothesis that the loads increase proportionally to a single multiplier λ, the structural response can be obtained by proceeding incrementally. In each step,

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an iterative procedure must be used in order to assure that the stiffness considered for each resisting element corresponds to its actual state. The resulting algorithm can be easily implemented.

Drifts (‰) Deflections (cm)

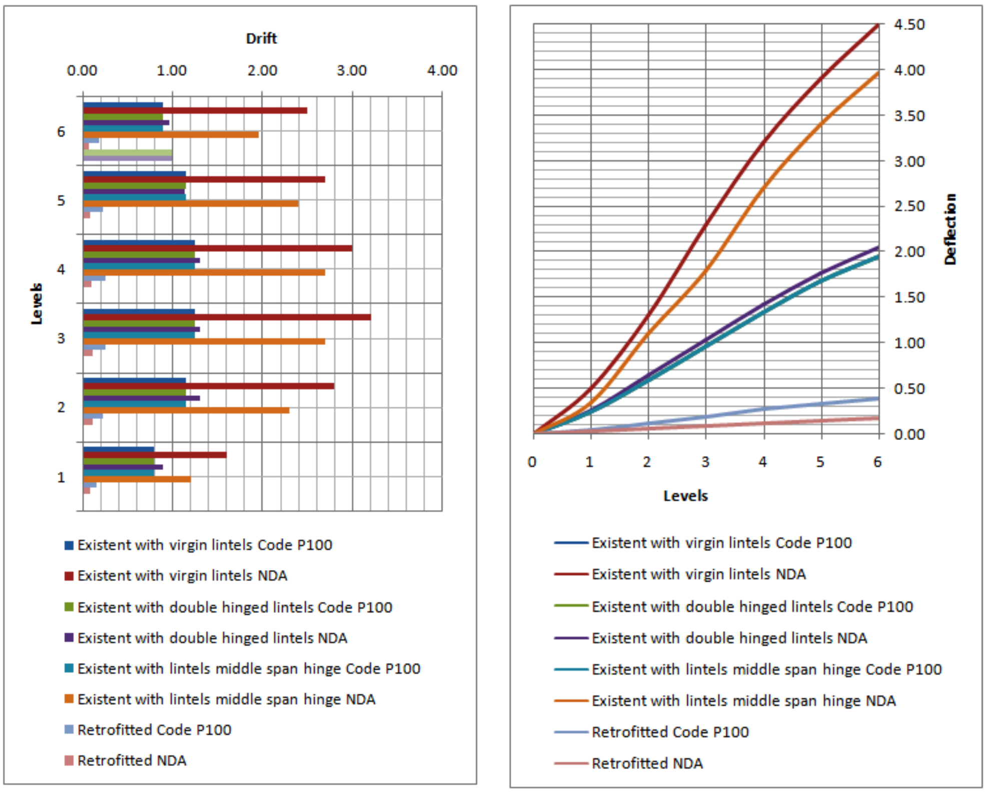

Figure 22b – Drifts and deflections for PGA=0.20g

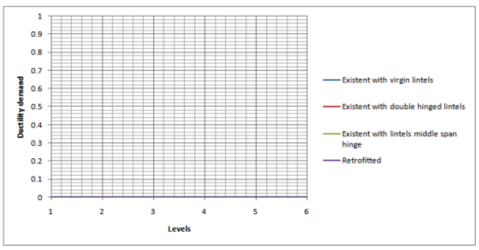

Figure 23a - Ductility demands for masonry piers – PGA=0.16g

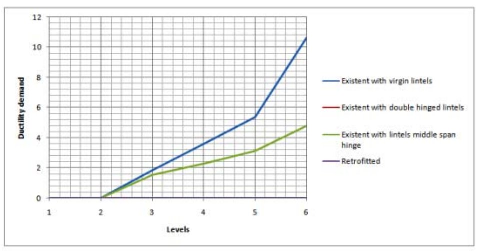

Figure 23b - Ductility demands for masonry coupling beams – PGA=0.16g

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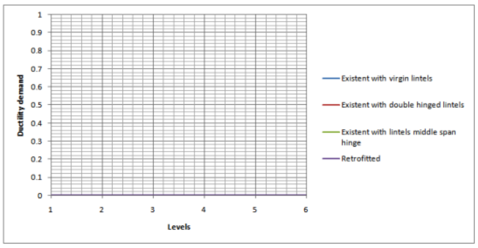

Figure 24a - Ductility demands for masonry piers – PGA=0.20g

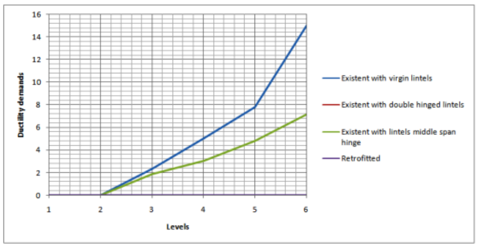

Figure 24b - Ductility demands for masonry coupling beams – PGA=0.20g

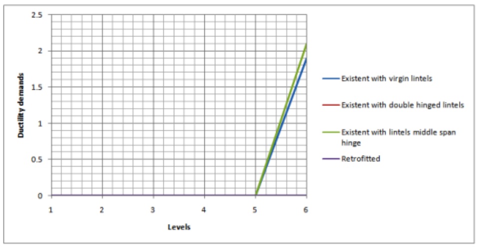

Figure 25a - Ductility demands for masonry piers – PGA=0.25g

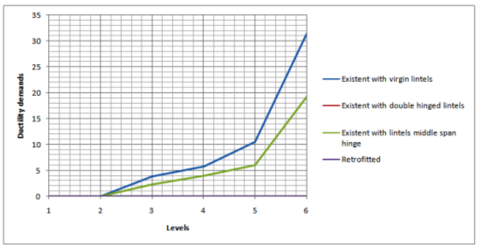

Figure 25b - Ductility demands for masonry coupling beams – PGA=0.25g

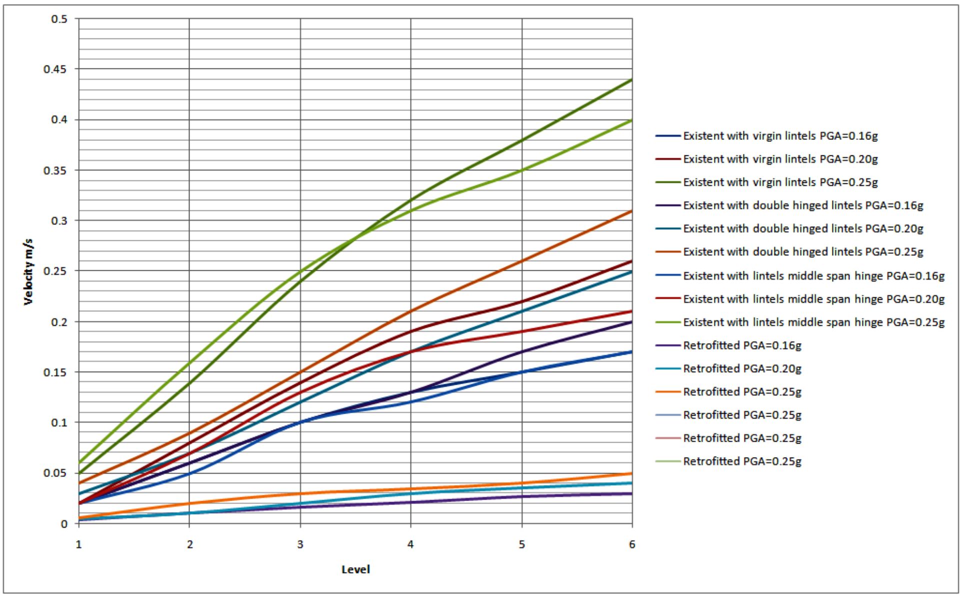

Figure 26 – Velocities (m/s)

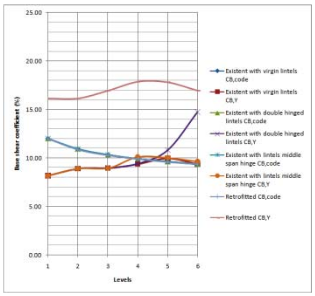

Figure 27a– C_B for PGA=0.16g

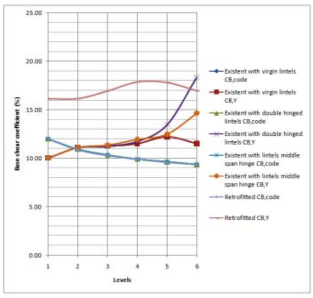

Figure 27b– C_B for PGA=0.20g

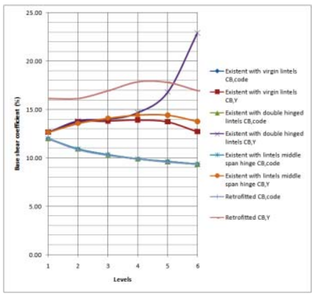

Figure 27c– C_B for PGA=0.25g

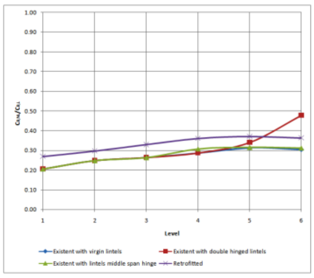

Figure 28a – C_{BNL}/ C_{NL} index for PGA=0.16g

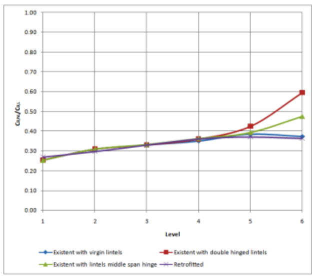

Figure 28b – C_{BNL}/ C_{NL} index for PGA=0.20g

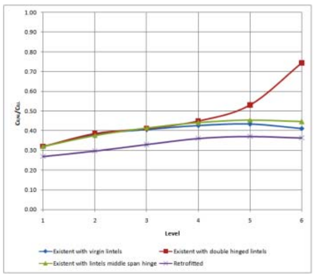

Figure 28c – C_{BNL}/ C_{NL} index for PGA=0.25g

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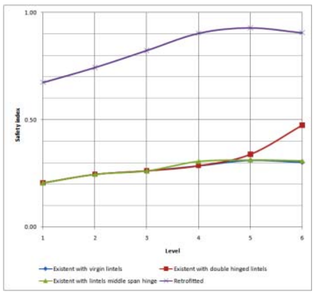

Figure 29a – Safety index for PGA=0.16g

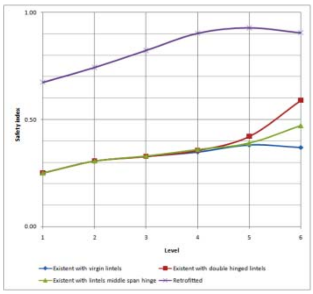

Figure 29b – Safety index for PGA=0.20g

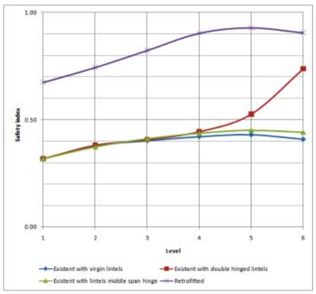

Figure 29c – Safety index for PGA=0.25g

4. CONCLUSIONS AND RECOMMENDATIONS

This ample study is just at the very beginning, but even after the things presented above, we shall say that the buildings with **masonry structures** are stiff structures. If normally in a simplified manner we appreciate the fundamental period of vibration for a frame structure at ***T= (0.03-0.1) n***, for these existing structures we found an average period of ***T=0.081n+0.069*** (where **n** is the storeys number). For the retrofitted masonry structures buildings the fundamental period of vibrations is about ***T=0.033n+0.028*** (sec.).

Usually the worse responses aspects became because of the mortar quality and not from the solid bricks.

Several retrofitting solutions are viable to increase the strength capacities (because usually the stiffness is more or less enough) but in these studies just one (extended to two) was carried out.

When a designer must expertise and retrofit this type of structures, from the beginning he will know the structural responses and expectations of them in comparison with all the demands.

Having these ideas will be more comfortable and easy to have discussions with the architects involved in the same project and of course with the owner of the building, in the idea to choose the better way to put in safe the existing building structure. Sometime other problems may occur such are the soil characteristics, the neighbourhood buildings.

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