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SEISMIC ANALYSIS OF A URBAN BLOCK IN FAIAL ISLAND - AZORES

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SUMMARY

The paper deals with numerical seismic analysis of an urban building block located in Faial island, Azores, hit by an earthquake on July, 9th 1998. It includes two different construction types, mostly traditional masonry structures and a reinforced concrete structure building in one block side. The analysis was supported by laboratory and *in-situ* tests, namely ambient vibration measurements on three traditional houses and on the RC building. Experimental mode shapes and vibration frequencies allowed mechanical parameter calibration for numerical models. Finite element time–domain dynamic analyses were made using the accelerograms recorded during the earthquake. This paper describes the seismic analysis of the global block as is and also parametric studies to assess the effect of possible reinforcing solutions in floors/walls and of the RC building. The most vulnerable block zones were identified as well as their interaction with the remaining structures. When possible reinforcing schemes were considered, results allowed assessing their efficiency on improving the block behaviour. Interdependency between different block elements was evidenced, showing that reinforcing may be done by means of single interventions in localized zones. However, stress redistributions caused by interventions must be taken into account because stiffness variations may increase stresses in reinforced zones, which means that individual building might not be appropriate.

1. INTRODUCTION

Numerical seismic analysis of existing structures constitutes a major challenge due to unavoidable difficulties on evaluating realistic data concerning the actual material and structural characteristics. This task becomes even more demanding when complex and old structures are involved, such as traditional masonry constructions where different types of structural components and materials form the whole structural assemblage.

It is therefore of utmost importance that any reliable numerical prediction be preceded and complemented by suitable experimental activities in order to provide relevant data for numerical model calibration concerning material properties, both mechanical and physical, and structural details such as supports and connections that must be adequately taken into account. For traditional masonry structures this aspect is particularly important because the structural behaviour is not only dependent on the material quality but also on the adopted construction techniques, fabric and workmanship.

In the present work context [Neves, N. 2004], the basic characterization of material properties was supported by an experimental campaign carried out in the Faial island, directly performed on existing structures hit by the July 9th 1998 Azores earthquake [Costa, A. 2002], and by laboratory tests on samples taken *in-situ* from those structures. In addition, the evaluation of the whole structural behaviour and characteristics was done by recourse to ambient vibration measurements on several structures of an urban building block of Horta, the major town of

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Faial island, Azores. For traditional masonry buildings, satisfactory results were obtained concerning the determination of one or two of the first vibration modes shapes and frequencies. Finite element numerical models were thus set up for those buildings and a calibration procedure was carried out on the basis of information experimentally obtained; the basic scope was therefore to achieve adequate numerical representations of the vibration modes obtained from the *in-situ* vibration tests and to adopt the so calibrated models as representative ones for the seismic analysis of the building block.
It is worth mentioning that this paper mainly focus on part of a vast work [Neves, 2004] in which all details are addressed, namely the whole set of experimental activities, all the numerical models and result discussion. Moreover, several aspects are already addressed elsewhere [Neves *et al*, 2004] concerning the finite element modelling of some key structures and their calibration process based on vibration measurements. This paper is thus essentially devoted to the discussion of the seismic analysis results of the whole building block, although a brief review is also provided concerning the main issues of model calibration based on experimental data and of seismic response of the tested structures.

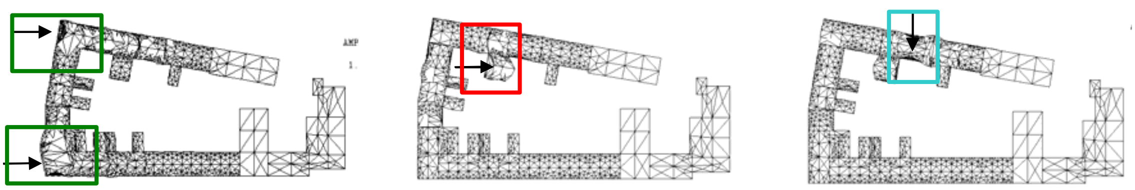
2. MODEL CALIBRATION BASED ON EXPERIMENTAL MEASUREMENTS

The block under study (Figure 1-a) is located near the Town Hall of Horta and mainly consists of traditional stone masonry buildings (medium grey in Figure 1-a) and of the Post Office building in the South side (dark grey in Figure 1-a) with a reinforced concrete frame structure. Four constructions were selected for vibration tests, namely three masonry houses (n. 15, 16 and 24, indicated with light grey in Figure 1-a and shown in Figures 1-b and 1-c) and the Post-Office building (shown in Figure 1-d).

a) Schematic layout plan b) House n. 15 c) Houses n. 16 and 24 d) Post-office building

Figure 1 – Building block: general layout and tested buildings

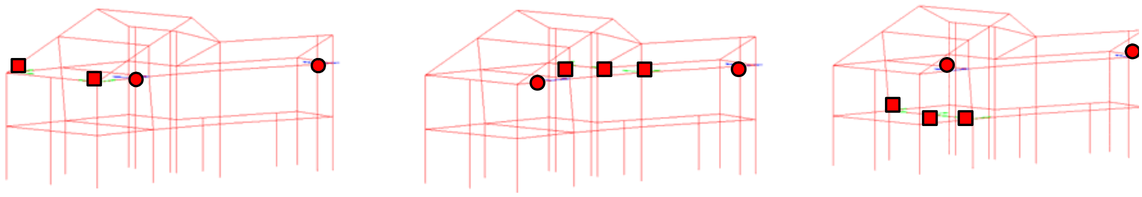
The selection for vibration tests was based on preliminary numerical modal analysis of the whole block [Neves, 2004] from which the most vulnerable points concerning the block dynamic behaviour were identified. Indeed, the three mode shapes shown in Figure 2, where block corners as well as in-plan and in-height irregularities take significant relevance in the whole deformed shapes, led to the selection of the above mentioned masonry houses. In addition, the RC frame building was also selected for vibration tests since it was considered to be likely to influence and affect the whole block behaviour in spite of being separated from the adjacent buildings by construction joints. However, for some reasons this building experimental results were not conclusive and, therefore, only those concerning the other buildings were considered.



a) Corners b) In-plan irregularities c) In-height irregularities

Figure 2 – Global block vibration mode shapes and identification of most vulnerable zones

Tests of ambient vibration measurements were carried out with five GEOSYG strong motion recorders (seismographs), GSR-12 and GSR-16 models of 12 and 16 bits resolution, respectively, equipped with three direction accelerometers (two horizontal and one vertical). Having some storage capacity, these seismographs can record accelerations that are then transferred to an auxiliary computer where a first analysis of the collected material can be carried out. In each house, records were made in series of measuring stations, each with three movable seismographs and two of reference; some of such stations are shown in Figure 3 for the house no. 15, with the location of the seismographs.

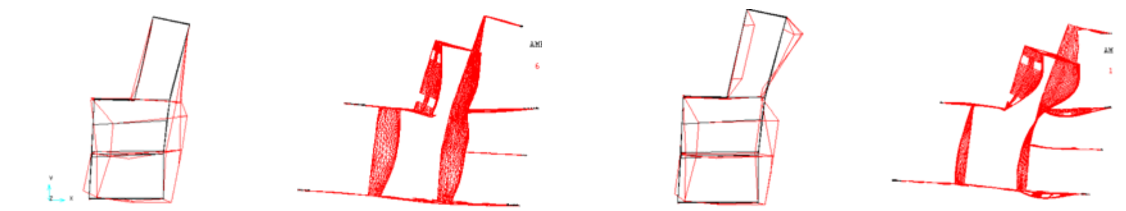


a) Station no. 1 b) Station no. 4 c) Station no. 7

Reference Seismograph Movable Seismograph

Figure 3 – Some ambient vibration measuring stations for house no. 15.

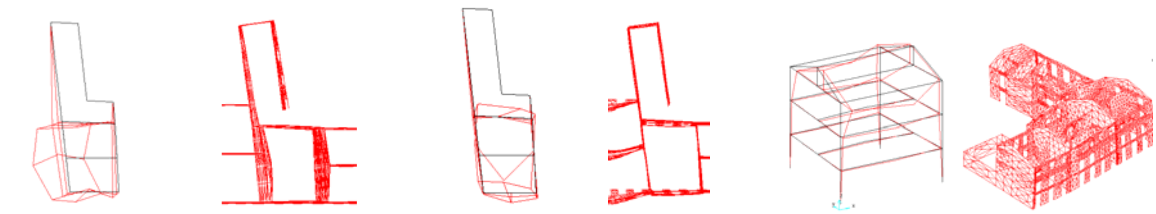
After careful analyses of the results obtained from the vibration measurements, it was possible to assess the two first vibration modes for both houses no. 15 and 16. The numerical models were then calibrated by adjusting the values of the elasticity modulus of the stone masonry walls and good agreement between numerical and experimental results for those vibration modes was achieved, as can be seen in Figure 4 and Figure 5. The same procedure was adopted for house no. 24, but only the first mode could be obtained from the experimental measurements; the comparison with the corresponding numerical vibration mode is also shown in Figure 5-c. The unit weight and the elastic modulus values thus considered for the stone masonry walls are listed on Table 1.



Experimental - 4.00 Hz Numerical - 4.06Hz Experimental – 5.18 Hz Numerical - 5.11Hz

a) 1st mode b) 2nd mode

Figure 4 – House no. 15. Vibration modes: Experimental *vs* Numerical



Exp. - 4.39 Hz Num. – 4.42 Hz Exp. – 5.66 Hz Num. – 5.74 Hz Exp. – 4.29 Hz Num. – 4.54 Hz

a) House no. 16 - 1st mode b) House no. 16 – 2nd mode b) House no. 24 - 1st mode

Figure 5 – Houses no. 16 and 24. Vibration modes: Experimental *vs* Numerical

Table 1 – House no. 15, 16 and 24. Stone masonry wall properties

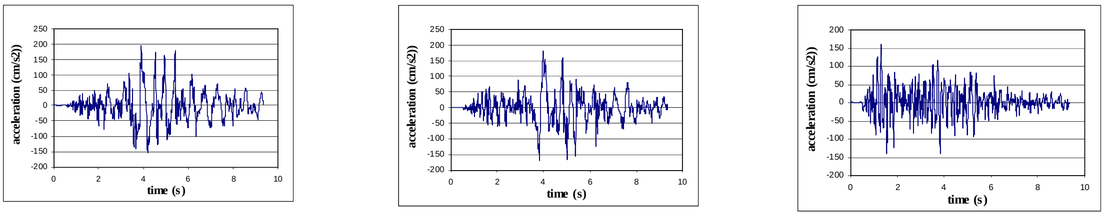
House no.	15	16	24
ρ (ton/m3)	1.8	1.8	1.8
E (GPa)	0.65	0.8	0.8

3. SEISMIC ANALYSIS OF TESTED BUILDINGS

According to the results of the calibration process, seismic analyses were made for the houses no. 15 and no. 16 and using CAST3M [CEA, 2003], a finite element based general-purpose structural analysis computer code. The structural modelling included 3-node shell elements for walls and 2-node bar elements for joists and wooden beams. All structural components that could condition the structural behaviour were modelled. Besides the seismic input referred bellow, vertical loading was also considered including dead loads (18 kN/m^3 for stone masonry walls, 2.7 kN/m^3 for wood and 0.5 kN/m^2 for additional dead loads) plus a quasi-permanent fraction (0.4) of live loads (2 kN/m^2), in agreement with Portuguese National Standards [RSA, 1983].

Since an earthquake in fact hit these constructions, a seismic analysis based on this specific action is of utmost importance. It allows a comparison between the numerical results and the actual impact of the earthquake over the block buildings and provides an additional validation of the conditions adopted for the modelling. In this case, the seismic input consists of the accelerograms registered on July, 9th 1998, at the foundation level of the Prince of Monaco Observatory, located in Horta - Faial Island, at an epicentre distance of approximately 10 to 15 km. This earthquake was generated in a tectonic fault, not being related with volcanic activity, and was classified with M5.8 in the Richter scale and 8 in the modified Mercalli intensity scale.

The obtained records show peak acceleration values of about 0.40g that are considerably above the peak ground acceleration subjacent to the seismic response spectra of the Portuguese National Standards. This apparent discrepancy is due to the fact that the Prince of Monaco Observatory is located on the top of a hill at 60 m altitude, much higher than that of the building block, which means that the recorded signals are affected by site amplification effects. For this reason, and in order to comply with the peak ground acceleration foreseen in the relevant code standards, the records were scaled to about 180 cm/s^2 of the maximum peak acceleration of the horizontal components as shown in Figure 6. The corresponding power spectra registered for its three components are illustrated on Figure 7. According to the block orientation, the XX direction is considered as coincident with the longitudinal axis and ZZ corresponds to the vertical direction.



a) XX Component b) YY Component c) ZZ Component

Figure 6 – Accelerograms registered on July, 9th 1998, and scaled for analysis input

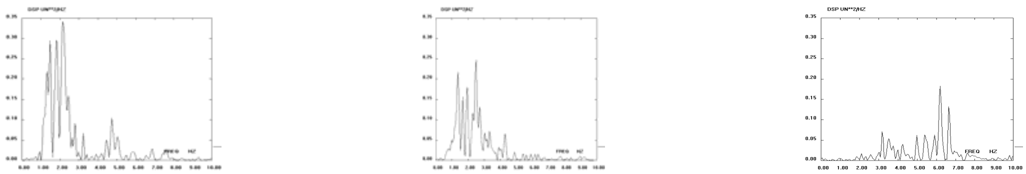

a) XX Component b) YY Component c) ZZ Component

Figure 7 - Power spectra densities of each component of the July, 9th 1998 earthquake.

The power spectra analysis shows that the earthquake horizontal components (XX and YY) have greater content in the frequency range between circa 1 Hz and 2.5 Hz, whereas the vertical component (ZZ) is more intense between 6 Hz and 7 Hz. Considering the spectral contents for the relatively high frequencies (higher than 2 Hz), this earthquake can be associated with Portuguese code standard action type 1 [RSA, 1983], whose characteristics are moderate magnitude and short focal distance. It is also worth mentioning the significant vertical component of these records that, for unreinforced masonry based structures mainly stabilized by their self-weight, normally induces important structural damage. In this case the peak vertical acceleration has preceded the horizontal ones, which means that the damaging effects of the horizontal inertia forces were amplified due to a prior release of the vertical compressive (stabilizing) stresses.

Results from the structural seismic response were obtained from linear elastic analysis by time integration using the Newmark method, considering viscous structural damping according to Rayleigh formulation (proportional to mass and rigidity matrices) and calibrated to ensure that the damping coefficient does not exceed 5% in the frequencies range relevant for horizontal and vertical components.

A vast set of results was obtained and is fully described in [Neves, 2004]. Besides the structural analyses with the calibrated models, some variants were additionally considered aiming at assessing the influence of the floor/roof stiffness (usually made of wood beams/joists and boards), of the group effect (i.e. the effect of adjacent structures, normally sharing a common structural wall) and of variations of mechanical characteristics of masonry walls. From comparative analyses the following main conclusions were drawn: *i)* floor/roofs should be provided with adequate in-plan stiffness and effective connections to the bearing walls in order to force the structure to behave as a whole; *ii)* the influence of adjacent structures is quite apparent and when not considered may lead to local underestimates of stresses; *iii)* the relative importance of mechanical properties of walls is increased when floors/roofs are not behaving with enough stiffness, requiring particular attention to wall corners.

Using the calibrated model it was possible to observe compressive stress levels compatible with the available structural element strength; the same is not valid for vertical or horizontal tensile stresses that are likely to reach larger values due to wall flexure and out-of-plane motion, depending on the effectiveness of the in-plan stiffness of floors/roofs. The inter-storey drift stays well within safety levels (0,1%) ensuring meaningless pathologies when floors/roofs have stiffness deficiency, but they can increase significantly putting the structural safety at risk in the limiting case of not existing floors (simulating a situation of very deficient connections between floors and walls). The influence of adjacent structures is generally positive, although it must be evaluated case-by-case depending on the wall location.

4. SEISMIC ANALYSIS OF THE WHOLE BLOCK AND STRENGTHNING MEASURES

4.1 Introduction

Numerical simulations were carried out aiming at two main objectives: *i)* to obtain the seismic response of the building block as is (reference scenario) in order to estimate peak values of several parameters such as displacements/drifts and principal stresses; *ii)* to perform parametric studies allowing to assess the effect of some issues, namely the stiffness characteristics of the houses' floors and walls, and the presence of the RC Post Office building in one side of the block. The later, in particular, is likely to affect the global block behaviour due to its location and significant stiffness that can be responsible for relevant eccentricities between the stiffness and the mass centres, thus increasing the earthquake impact on masonry buildings due to block global torsion. Concerning the floor and wall stiffness effects, the basic idea is to introduce some localized modifications of their characteristics in selected zones so as to simulate possible strengthening interventions.

After having calibrated the mechanical properties of materials and the modelling criteria according to the tested houses study (in particular of the house no. 15), the whole block numerical analysis was carried out under the same seismic input already described. A block global model (Figure 8) was developed, including all buildings. Considering the model dimension, it was decided to refine the mesh only for the stone masonry structures and to adopt a less discretized mesh for the reinforced concrete building. Using the already referred computer code, all components were simulated with shell elements, including wooden floors, since the simulation of main beams and joists with beam elements would lead to an extremely expensive model. All mechanical properties of materials were considered as for the house no. 15 assessment and the so obtained numerical model is referred in the following as the reference scenario.

4.2 Numerical seismic response of the block for the reference scenario

Based on the "*in-situ*" survey process and on extrapolation of material data listed in Table 1, traditional masonry building walls were given average global values of elasticity modulus as indicated in Figure 8. As for the wooden floors, their simulation was done by 1.5cm thick shell elements with 3.8GPa for the elasticity modulus (typical of criptomeria wood species) and equivalent unit weight to convey the actual floor weight. That thickness was previously tuned to simulate an in-plane floor stiffness equivalent to that of the real floor (wood boards nailed to main wooden beams). Finally for the RC Post Office building, usual concrete properties were adopted (γ = 25 kN/m^3 and E = 29GPa) and 15cm thick shell elements were considered for the walls in order to account for wall openings; based on "*in-situ*" observations, similar shell elements were also considered for slabs.

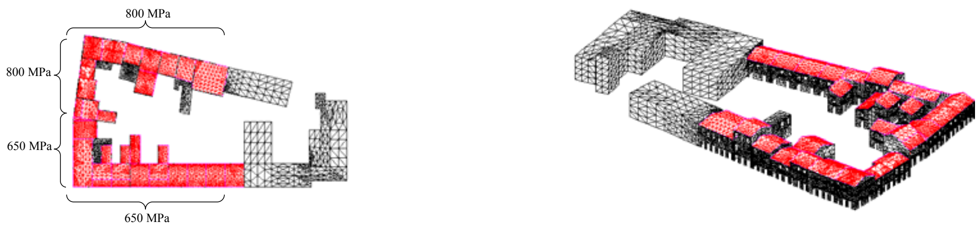
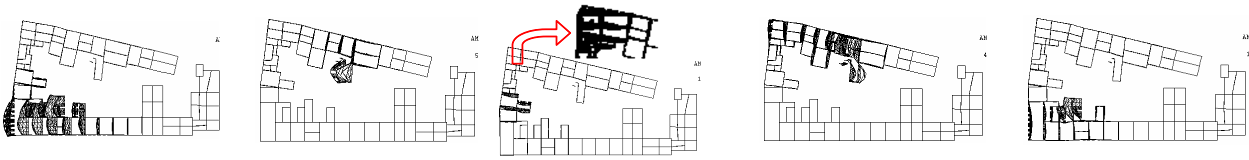


Figure 8 – Block global model: plan and perspective views.

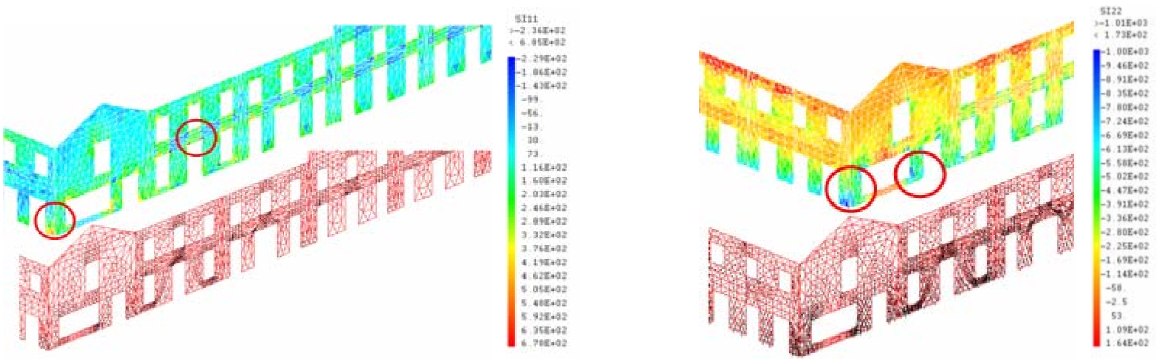
Numerical modal analysis allowed obtaining the vibration modes and frequencies shown in Figure 9 where references are also included to the corresponding vibration modes obtained from individual analyses of the tested houses (no. 15, 16 and 24); this modal correspondence has shown reasonably close frequency values obtained in local analyses of each house and in the global block analyses. Particular attention is drawn to mode no. 3 that, despite mobilizing more than one house of the left block side, also involves the houses of the top-left corner where the house no. 24 is included. These results stress again the fact already mentioned that the most seismically vulnerable zones are the block corners (modes no. 1, 4, 5 and 6, 8, 9,10 - not shown), the in-plan irregularity zones (mode no. 2) and the height irregularity zones (modes no. 4 and 6, 9, 10 - not shown).



1st mode: 4.52HZ (1st mode of house 15) | 2nd mode: 4.63HZ (1st mode of house 16) | 3rd mode: 4.75HZ (1st mode of house 24) | 4th mode: 5.09HZ (2nd mode of house 24) | 5th mode: 5.53HZ (2nd mode of house 15)

Figure 9 – Vibration modes and frequencies of the building block for the reference scenario.

From time domain analysis, principal stress distributions were obtained for the masonry walls as shown in Figure 10 for the zones where peak tensile (Figure 10-a) and compressive (Figure 10-b) stresses occur, both in terms of direction and intensity. Note that these peaks stresses are mainly localized near the corners (maximal values indicated in the figures) although equally high values can be found in house no. 16, particularly in the connections to adjacent lower houses and near the wall openings. It is worth mentioning that compressive stresses are reasonably compatible with masonry strength limits while tensile stresses show that some cracking can be found, though without affecting structural stability. Generally speaking it is clear that the corners and the top alignments of buildings are the critical zones; however, despite some stress concentration in these localized zones, the block shows a good stress distribution, possibly due to the reasonable homogeneity of houses.



a) Principal tensile stresses (max. 678kPa) b) Principal compressive stresses (max. 1010kPa)

Figure 10 - Distribution and direction of principal stresses (block reference scenario).

The reduced deformation capacity of stone masonry walls is usually a critical issue for this type of structures under seismic loading. Therefore, maximal displacements were evaluated in six zones (Figure 11) of the main façades of the block buildings, both in the longitudinal (x) and transversal (y) directions.

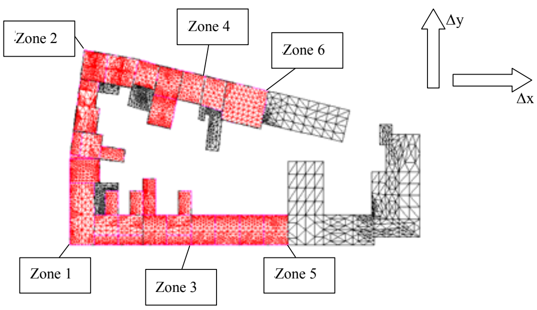
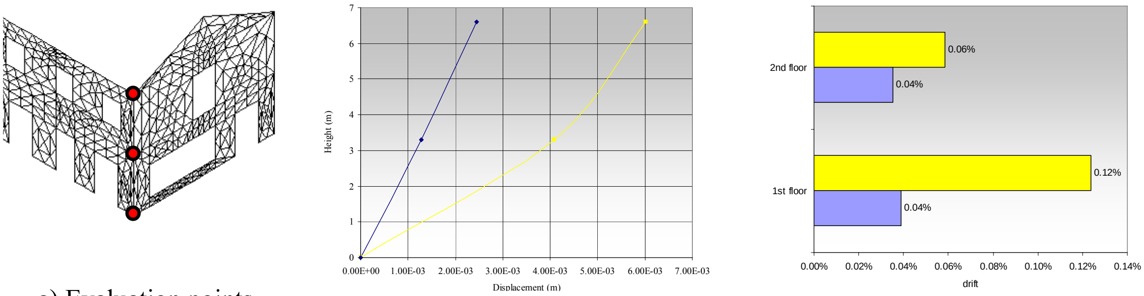


Figure 11 – Zones where displacements were evaluated.

In this paper only the results for zone 1 are presented (**Figure 12**) where the largest displacements are observed in the longitudinal direction and in clear agreement with the obtained modal configurations. Such displacements refer to intersections of wall alignments and are associated with drift values in the range 0.04% to 0.12%, therefore not likely to affect the structural stability.



a) Evaluation points b) Displacements c) Drift values

Long. direction (X) Transv. direction (Y)

Figure 12 - Zone 1 – Displacements and drifts (block reference scenario)

4.3 Parametric study of the floor stiffness influence on the block seismic response

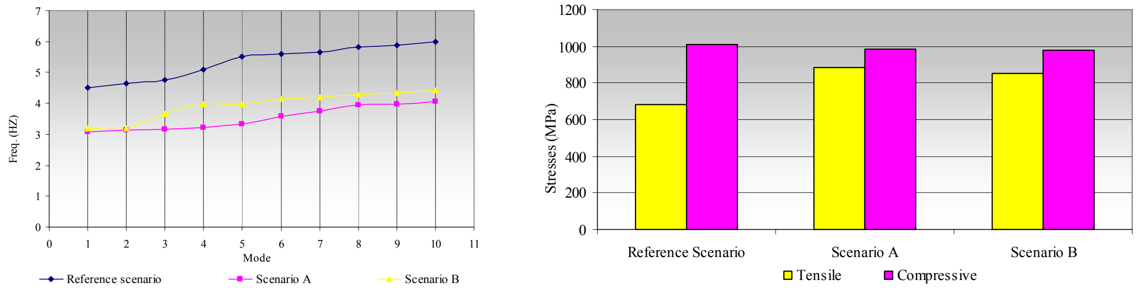
Floor characteristics as adopted for the block reference scenario are valid for the testing conditions, i.e. under low vibration intensity. In case of earthquake occurrence, accelerations significantly increase and, therefore, the tuned floor characteristics may not be appropriate unless efficient connections between floor elements and walls are provided to ensure that global behaviour of structure can be actually mobilized. Thus, aiming at assessing the influence of floor characteristics and of possible localized reinforcing interventions, two other alternative scenarios were also considered as follows:

- Block model where wooden floors are assumed very inefficient concerning diaphragm stiffness, as it were constituted just by main beams providing only localized restrictions to wall movements. This configuration was artificially simulated with 0.15cm thick shell elements and is referred in the following as the no stiff floor scenario or floor scenario A.
- Based on the previous scenario, some houses were considered with floor and roof stiffening to improve the in-plan diaphragm effect. These houses were chosen according to criteria of seismic vulnerability, namely in the block corners (zones 1 and 2), in the height irregularity zones (zone 4, house no. 16) and in a house in the middle of zone 3, a continuous and homogeneous alignment, for comparison purposes. For each reinforced house, 1.5cm thick shell elements were adopted for the floors whereas the wall characteristics were kept as in the reference scenario. The so obtained configuration is referred as the reinforced no stiff floor scenario or floor scenario B.

By comparing the vibration mode frequencies for the three scenarios (reference, A and B) as shown in the chart included in Figure 13-a, it is clear that the simulated floor reinforcing solution has no effect in the first two modes because these modes mainly mobilize not reinforced and more flexible structures. By contrast, in the remaining upper modes a slight frequency increase is observed, thus indicating a global block stiffness increase instead of just the reinforced houses, where modal configurations are also clear (not shown herein) concerning the type of wall deformations.

With regard to peak values of principal stresses in the main façade walls, the chart included in Figure 13-b for the same three scenarios allows to conclude that compressions (normally in the ground level and mainly due to structural weight) are not very affected, although some localized significant variations can be found but not critical for the structural stability. However, for tensile stresses, more significant increases are observed when no stiff floors are considered, because wall flexure is more intensely mobilized. The simulated floor reinforcement has little influence on peak stress values; however, in the reinforced zones it leads to good results by significantly reducing the stresses.

Displacements were also obtained for the zones already described and for the three scenarios. Figure 14 shows the results for zone 1 where larger displacements can be observed along the longitudinal direction X, in accordance with the 1st mode that mainly develops along that direction. It is worth noting that displacements are larger for the reference scenario, i.e. for floors with the tuned stiffness. This can be explained by the fact that floor stiffness tends to increase a global block behaviour effect in the zone 1 corner, leading to more interaction of adjacent structures and to stress and displacement increase.



a) Block vibration frequencies b) Peak tensile and compressive stresses

Figure 13 – Block frequencies and peak stresses for different floor stiffness scenarios

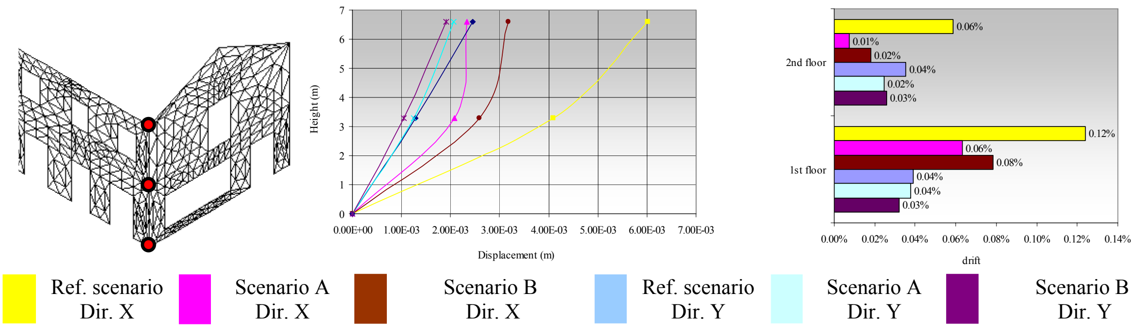


Figure 14 - Zone 1 – Displacements and drifts for different floor stiffness scenarios

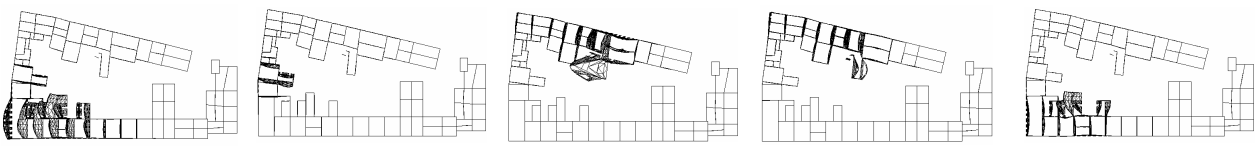
In summary, the floor stiffness influence was seen to increase with the building height due to the lower relative wall stiffness compared to floor stiffness in the upper floors. For the cases under analysis, the observed deformations are not significant and correspond to inter-storey drifts about 0.12% which are not likely to introduce serious damage in structural elements.

4.4 Parametric study of the wall stiffness influence on the block seismic response

The influence of possible wall reinforcements on the block response was also addressed by considering two other scenarios in which the floor stiffness was considered as in the reference scenario and the wall stiffness was locally increased according to the following:

- Scenario C – The house in the zone 1 corner was reinforced by introducing a new masonry wall in the longitudinal direction to give additional support to more flexible transversal walls.
- Scenario D – The same houses as for scenario B were considered as reinforced, adopting the floor stiffness as for the reference scenario and walls with E=1.2GPa instead of the values shown in Figure 8.

Scenario C was very localized and, consequently, has shown very little differences in the global block behaviour, in comparison to the reference scenario. Therefore, the attention is herein mainly focused in the scenario D results of which Figure 15 includes some vibration modes and frequencies as for the reference scenario.

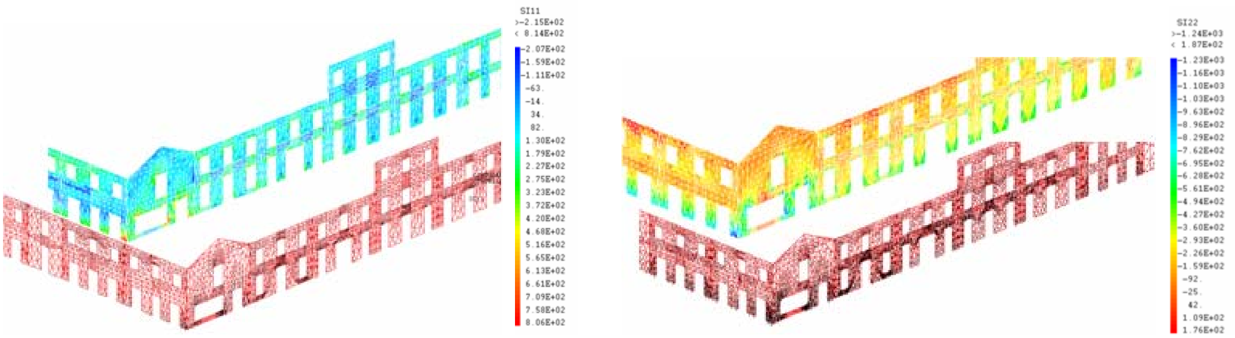


1st mode: f = 4.72HZ 2nd mode: f = 4.85HZ 3rd mode: f = 5.16HZ 4th mode: f = 5.46HZ 5 th mode: f = 5.60HZ

Figure 15 - – Vibration modes and frequencies of the building block for scenario D

By comparing the results of Figure 15 with those of Figure 9, some changes are found concerning the global block behaviour which are first of all related with the general increase of frequencies. Although not very significant, such changes lead to visible stress and displacement variations in some walls as described below.

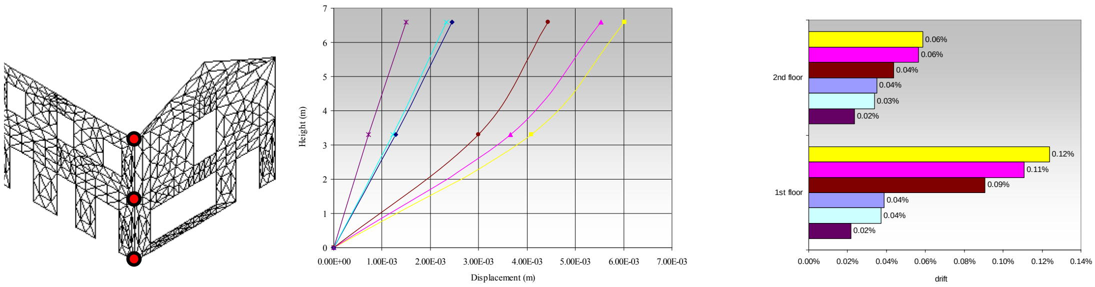
Stress distributions and displacements were obtained and analysed for the same block region as in the previous section. Thus, Figure 16 shows stress results concerning the scenario D from which about 20% stress increase can be found in comparison to the reference scenario results. This increase is associated with the stiffness increase that "attracts" more stresses and is essentially located in the reinforced zones. Thus, the pertinence of such reinforcing strategy may sound questionable unless the available strength and ductility increase is adequate for the new demands. In any case, for this kind of structures it is particularly relevant the importance of this type study in order to assess the dependency between local and global behaviour.



a) Principal tensile stresses (max. 814kPa) b) Principal compressive stresses (max. 1250kPa)

Figure 16 - Distribution and direction of principal stresses (block scenario D).

Concerning displacements and drifts similar results are included in Figure 17 for scenarios C and D, as those previously presented in Figure 14 for the reference scenario. It can be observed that the reinforcement in the scenario D is more efficient, even in the zone 1 where the scenario C reinforcement was considered. Despite the relatively low values of displacements, the scenario D reinforcement greatly contributes to the displacement reduction (about 25%).



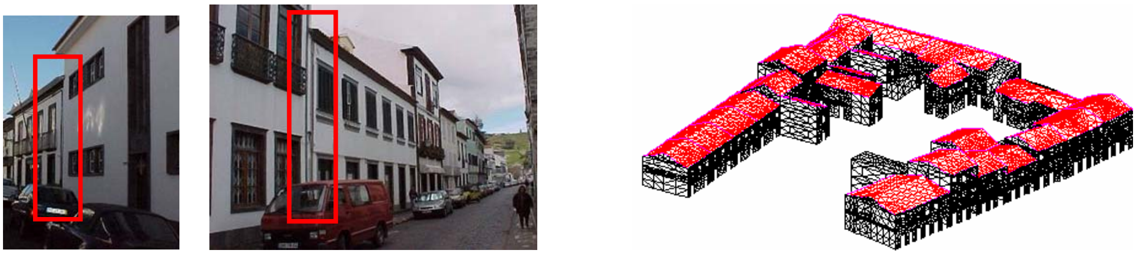
Ref. scenario Dir. X | Scenario C Dir. X | Scenario D Dir. X | Ref. scenario Dir. Y | Scenario C Dir. Y | Scenario D Dir. Y

Figure 17 - Zone 1 – Displacements and drifts for different wall reinforcement scenarios

4.5 Study of the RC Post Office building influence on the block seismic response

Although the presence of the RC Post Office building was expected to introduce relevant changes in the overall block behaviour, a simple modelling simulation modification was first considered to provide a rough evaluation of the result trends. Since a dry joint exists (Figure 18-a) between the RC building and the traditional masonry houses, a stiff and full connection between them would not appear as the most adequate modelling simulation. In fact, this is a unilateral contact type connection that only activates under compressive stress states in the joint. Therefore, the simplest way to assess the RC building was to delete its portion from the original finite element global model used for the analyses described so far. The resulting model mesh is illustrated in Figure 18-b, with which several analysis variants were done, namely concerning the effects of floor stiffness as before.

Regardless of the floor behaviour type, results have shown much lower influence of the RC building on the global behaviour than what could be expected, in particular in the most stressed zones [Neves, 2004]. Despite the adopted modelling simplification, it is hardly expectable that an increased influence can be induced by a more correct representation of the interface between the two types of buildings; this means that the simple strategy allowed assessing the essential effects resulting from the presence of the RC building. From the observed it appears that the whole stiffness of the longitudinal wall alignments is sufficient for the lot of masonry houses to stand almost “independent” of the Post Office building, therefore not being affected by its presence or absence.



a) Transition between RC and masonry buildings b) Model mesh without the RC building (perspective)

Figure 18 - RC Post Office building influence on the block seismic response.

5. FINAL REMARKS

Numerical analyses of the several block models allowed finding the most vulnerable zones, particularly concerning peak stress values. Such zones are mainly located in the block corners and where in-plan and height irregularities exist, and are more easily mobilized by vibration modes with lower frequencies. However, for the seismic action the block has been subject to, compressions are compatible with masonry strength while tensile stresses, though clearly exceeding the masonry capacity, are very easily absorbed by a suitable reinforcing scheme of walls. As for wall displacements, drift values in the range 0.04% to 0.12% are obtained, which are not likely to affect the structural stability.

Simulations of different possible reinforcing strategies were carried out involving either masonry walls or wooden floors. Reasonable efficiency was found on the global block behaviour improvement, evidencing the interdependency of all structural elements of the block even when the reinforcement is done locally in selected zones. This means that individual studies of each isolated house or building may not be appropriate, because the global behaviour is not accounted for. Contrarily to the expected, the presence of the RC building in one side of the block did not show significant influence in the most stressed zones.

Although the analyses were done considering a simple linear elastic behaviour (calibrated on the basis of experimental *in-situ* measurements), in the authors opinion the results provide valuable information if they are seen in the reinforcement design perspective. Thinking as for common RC design, if the obtained compressions are comfortably within strength limits and if adequate reinforcing measures are adopted to absorb tensile stresses (for instance by recourse to wall plasters reinforced with any adequate material like steel or polymers), then linear elastic analysis results are perfectly applicable. The most important question then comes on the realism and adequateness of material data and modelling assumptions adopted for the analysis! This topic is in fact of major concern and, to some extent, it has been accounted for thanks to the campaign of *in-situ* vibration tests.

6. REFERENCES

CEA (2003), Manuel d’utilisation de Cast3m, Commissariat à l’Énergie Atomique; Pasquet, P., www.cast3m.cea.fr.

Costa, A. (2002), Determination of mechanical properties of traditional masonry walls in dwellings of Faial Island, Azores. *Earthquake Engineering & Structural Dynamics*.

Neves, N. (2004), Identificação Dinâmica e Análise do Comportamento Sísmico de um Quarteirão Localizado na Cidade da Horta – Ilha do Faial, *Tese de Mestrado* (in portuguese), FEUP, University of Porto, Portugal.

Neves, N., Costa, A. and Arêde, A. (2004), Seismic Analysis of a Building Block Located in an Urban Area of the Faial Island of Azores, *Proc. of the 13th World Conference on Earthquake Engineering, Paper 863*

RSA (1983), Regulamento de Segurança e Acções para Estruturas de Edifícios e Pontes, Dec.-Lei n. 35/83, *Imprensa Nacional - Casa da Moeda*, Lisboa.