# ANALYSIS OF A DAMAGED BUILDING IN THE NORTH REGION OF PORTUGAL

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ABSTRACT: A damaged building in the North Region of Portugal is analysed. Geotechnical characterisation of the soil foundation is presented. This soil has a low bearing capacity and the water level is near the ground surface.

The reinforced concrete structural skeleton of this building consists of a mat foundation and slabs joined by walls. The façades of the building are in air brick masonry.

Instrumental devices to measure the settlements of the building were installed. To observe the behaviour of the developed cracks, a monitoring system was also applied. The water table was observed with piezometers.

A tridimensional numerical linear-elastic analysis using a finite element computer program considering soil-structure interaction is done. The soil is modelled using springs distributed continuously and the values of the modulus of subgrade vertical reaction are estimated from the geotechnical properties. The numerical results reveal the deformations and the damages of the building. These results are compared with the field observations.

A rehabilitation structural solution is analysed in order to decrease the differential settlements and the soil pressure, and sustain the crack development.

## 1 - INTRODUCTION

To build a building in the suburbs of a city in the North of Portugal the geotechnical survey indicated a base soil with low bearing capacity and very sensitive to water content. Furthermore, the water table level was detected near the ground surface.

Two solutions for foundations were considered: pile foundations or mat foundation. The mat foundation was chosen because it was less expensive.

The building has underground technical galleries, ground floor and first floor. To give more stiffness to the structure, the construction was built in tunnel with a mat foundation, reinforced concrete walls in one direction and reinforced concrete slabs. In the other direction the walls were in air brick masonry. The building was monitored with markers to observe its settlements.

In spite of its stiffness, the building began to crack, mainly, in the air brick masonry. When cracks began to appear markers were placed to observe their opening and closing movement. The water table was also observed in piezometers.

To analyse the building structural behaviour, a tridimensional numerical analysis using a finite element computer program considering the soil structure interaction is done.

A competitive structural reinforcement solution is proposed to sustain the damages of the building.

# 2 - GEOTECHNICAL CONDITIONS

## 2.1 - Introduction

The geologic formations at the site are granitic with alteration and alluvial soils in the upper layer. The geotechnical survey has been done in four wells. Samples of the soil were taken from the walls of the wells for granulometric, Atterberg limits and other characteristic measurements. Fourteen dynamic penetration heavy tests were done. In four of them the bedrock was reached. The minimum depth was 11.4 m and the maximum was 18.6 m.

## 2.2 - Soil characteristics

In the alluvial soils, the number of blows in the dynamic penetration tests ranged from 0 to 2. These results reveal soils with low compactness. Seven soil samples were analysed. Three were sand (most of the particles have sizes between 2 mm and 4.76 mm) and four were silty-clayey soils. For the sand the percentage of passing the 200 sieve (ASTM standard) varies from 14 to 30. In these soils the silty particles (with size between 2 $\mu m$ and 74 $\mu m$) are in more quantity than the clay particles (size smaller than 2 $\mu m$). Therefore, the soils are non-plastic. In the case of the silty-clayey soils the percentage of passing the 200 sieve varies from 58 to 86. Most of these soils are plastic with plasticity index between 11 and 14. According to the Unified Soil Classification system, the sand can be classified as SM and the silty-clayey soil can be classified as CL and ML.

## 2.3 - Water table level

The water table level was observed in seven piezometers from September 1996 to April 1997. It was observed that, even in the rainless months, most of the measures were above the floor of the underground galleries. This high water level can be justified by the insufficient drainage system of deep waters. As these soils are very sensitive to water content, these high water levels contribute to the reduction of the shear strength of the soils.

## 2.4 - Modulus of subgrade vertical reaction

To evaluate the modulus of subgrade vertical reaction of the soil, $k_s$, the following equation is used (Bowles 1988).

$$k_s = 40 q_a F_s \tag{1}$$

where $q_a$ is the allowable bearing pressure of the base soil and $F_s$ is the safety factor. The ultimate bearing pressure divided by the safety factor gives the allowable bearing pressure.

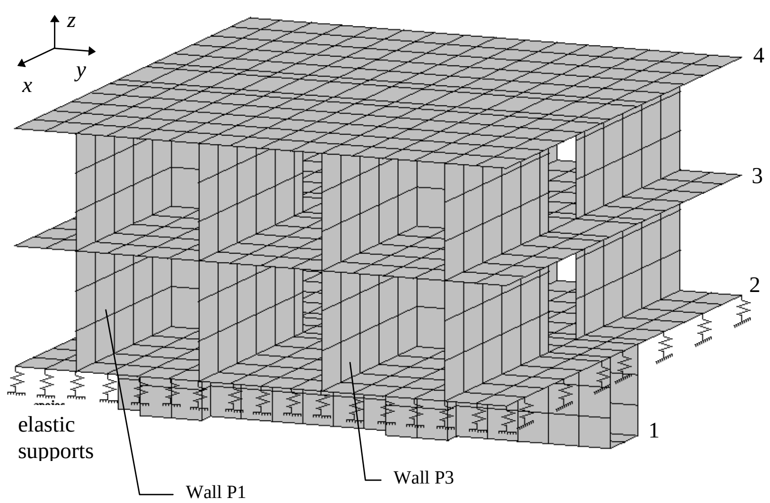
The allowable bearing pressure was estimated equal 50 kPa. Substituting this value in equation (1) and using a safety factor equal 3, the modulus of subgrade vertical reaction is equal to 6000 kN/m$^3$.

In the next section the modulus of subgrade vertical reaction will be taken between 2000 kN/m$^3$ and 10000 kN/m$^3$.

# 3 - NUMERICAL ANALYSIS

## 3.1 - Introduction

A representative module of the building structural behaviour is discretized by eight nodded Ahmad shell finite elements (Zienkiewicz and Taylor, 1989). The finite element mesh is illustrated in Figure 1.




1 - GALLERY; 2 - MAT FOUNDATION; 3 - 1$^{ST}$ FLOOR; 4 - 2$^{ND}$ FLOOR

Figure 1 - Finite element mesh.

A tridimensional numerical linear elastic analysis is done using a finite element computer program named FEMIX, release 2.3 (Azevedo and Barros, 1997). In this analysis the soil-structure interaction is accounted for, and the soil is modelled as being a surface rigidity - springs distributed continuously - (Barros, 1996). The values for the modulus of subgrade vertical reaction were evaluated in section 2.4.

Amongst the studies carried out for the simulation of the structural behaviour, only the two most representative will be described. In the first study (analysis A) the soil is considered homogeneous, with the same $k_s$ value in all points, and the structural concrete is taken as undamaged. This study is intended to simulate the behaviour before occurred the first damages. In the second study (analysis B) the soil is considered non-homogeneous with two different $k_s$ values, and the concrete rigidity is reduced in order to simulate the damage due to concrete cracking.

A structural rehabilitation solution proposed to sustain the damages is also analysed.

## 3.2 - Analysis A

In this analysis the structure is considered without any damage and the soil is assumed homogeneous. In the uniaxial compression tests carried out with concrete cylindrical specimens extracted from the structure, it was verified that this structure was built by a C20/25 concrete (CEB-FIP, 1993). The soil was simulated with a $k_s$=10000 kN/m$^3$, which is the higher value of the range proposed in section 2.4. The self weight of the structure and the surcharges on the slabs are actions taken according to the Portuguese legislation.

The deformed mesh with scaled displacements (×300) is represented in Figure 2. In this case the building settles almost uniformly. The settlements normal to the floors (in direction of $z$ axe) are equal to 0.011 m. This value is very close to that measured four months after the construction (0.013 m). The pressure on the soil under the mat foundation and under the gallery is approximately 110 kPa.

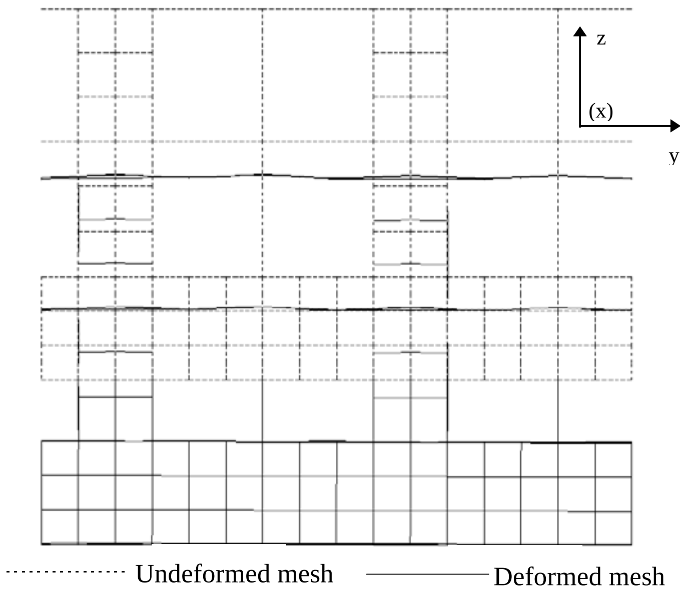



Figure 2 - Deformed and undeformed mesh for analysis A.

Other analysis were done with different combinations of actions and maintaining a uniform value for the modulus of subgrade vertical reaction. This modulus was changed from 2000 to 10000 kN/m$^3$. All these analysis indicated that, considering an uniform modulus of subgrade vertical reaction, the differential settlements between the reinforced concrete walls were so reduced that can not justify the macrocracks developed in the air brick masonry.

## 3.3 - Analysis B

In this analysis the soil is considered heterogeneous and two different modulus of subgrade vertical reaction are used. The soil between the walls P1 and P3 (see Figure 1) is simulated by $k_s = 2000$ kN/m$^3$, while the remaining soil is characterised by $k_s = 10000$ kN/m$^3$. In order to simulate the lost of rigidity due to cracking, a reduced concrete Young modulus of 20 GPa is used. The displacements in $z$ axis (orthogonal to the floors) are represented in Figure 3. A maximum differential settlement of 0.005 m is obtained, which cause the development of compression and tension zones in the air brick masonry (see Figure 4).

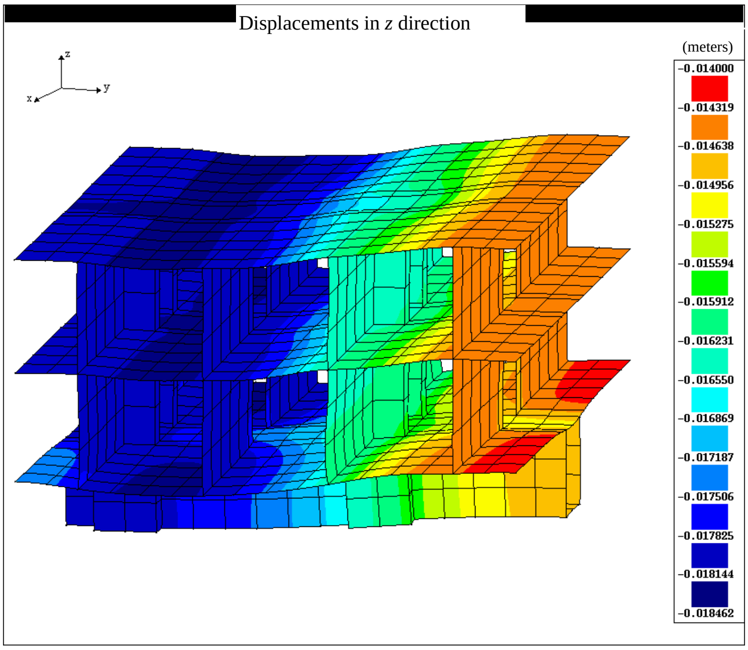



Figure 3 - Displacements in *z* direction for the analysis B.

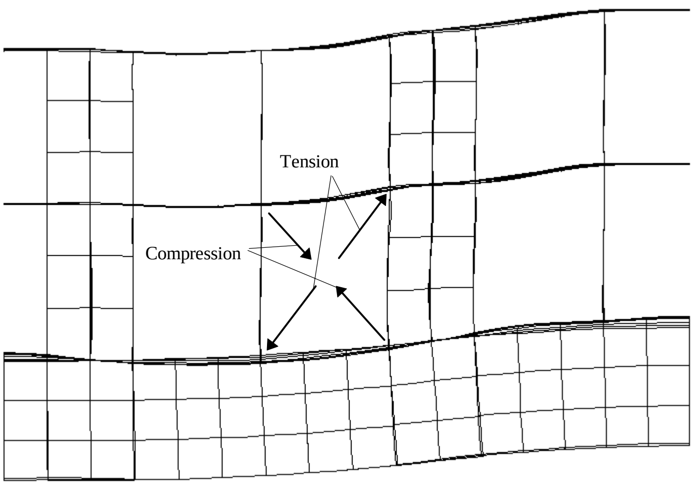



Figure 4 - Deformed mesh for the analysis B.

The soil pressure varies from 30-40 kN/m$^2$ in the low $k_s$ value zone to 180 kN/m$^2$ in the remaining soil (see Figure 5).

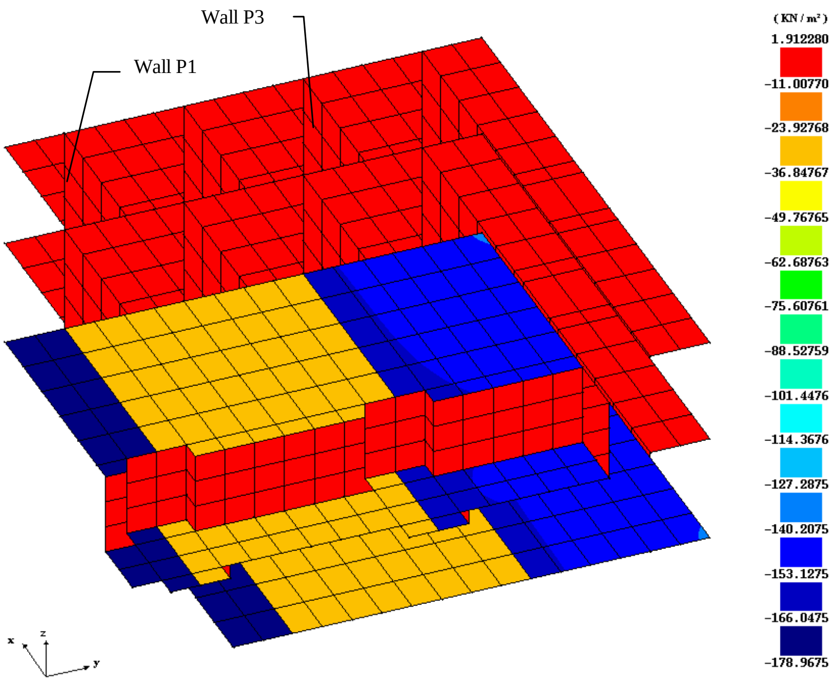
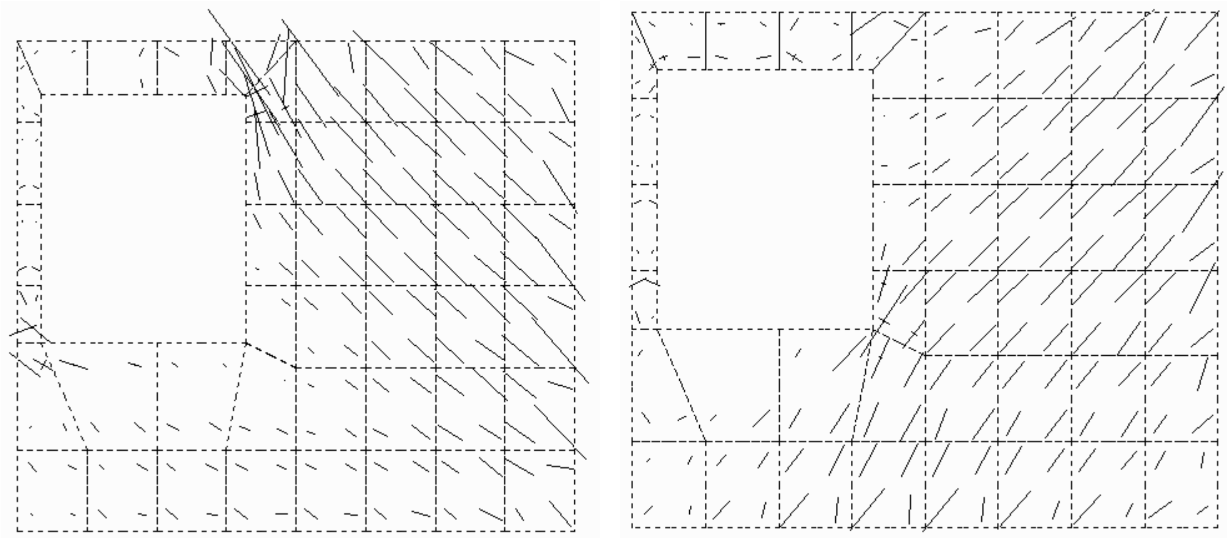



Figure 5 - Soil pressure for the analysis B.

## 3.4 - Air brick masonry

A panel of air brick masonry between concrete walls and floors is analysed. This panel corresponds to that submitted to greater shear forces introduced by the structural border elements (see Figure 4). It is considered a panel of 0.25 m of thickness with an homogeneous behaviour. The analysis is linear elastic and it is taken a Young modulus of 1 GPa. The panel is discretized by eight nodded Ahmad shell finite elements. The actions considered are the generalised displacements introduced by the structural border elements in the contour of the panel.

The in-plane principal stresses developed are represented in Figure 6. It is possible to detect the existence of tension and compression zones. The maximum tension and the maximum compression are about 0.5 MPa and 0.9 MPa, respectively. These values are higher than the corresponding material strength (Carvalho, 1990).
A schematic representation of the damage zones is shown in Figure 7. The concentration of the maximum tension and maximum compression occurs in the corners of the window. This is in accordance with the damages observed in the building.



(a) (b)

Figure 6 - Compression (a) and tension (b) in-plane principal stresses in the panel of air brick masonry.

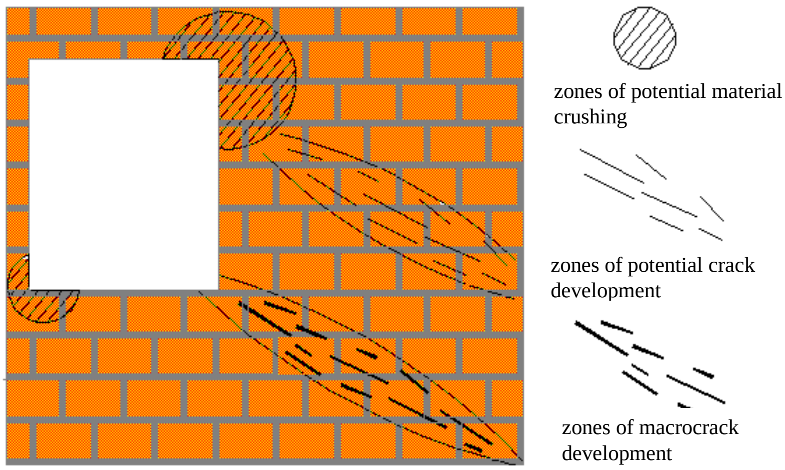



Figure 7 - Damage zones in the air brick masonry.

## 3.5 - Analysis of a structural rehabilitation solution

In order to rehabilitate the building, the interior panel of the façades air brick masonry is replaced by a reinforced C25 concrete wall of 0.10 m thickness. This new model is analysed with the same conditions taken in the analysis B (section 3.3). The displacements in $z$ axis direction are shown in Figure 8. The maximum differential settlement is reduced to 0.002 m.

The compression and tension membrane principal forces in the new reinforcing walls are illustrated in Figure 9. The maximum compression and the maximum tension stress are about 3.6 MPa and 3.2 MPa, respectively.

A wire mesh with a steel yield stress of 348 MPa is applied at interior and exterior surfaces of these walls (with a 0.015 m of concrete recover). The area (per unit length) of this reinforcement is represented in Figure 10. The reinforcement at interior and exterior wall surfaces is practically equal. Near the corners of the windows, each wire mesh must has 8.4 and 10.6 $cm^2/m$ in *y* and *z* axis direction.

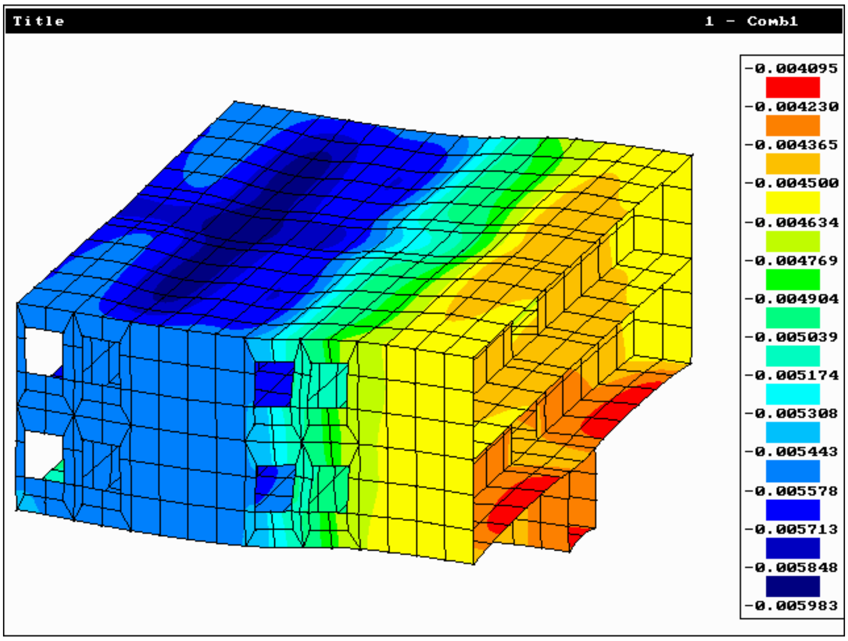
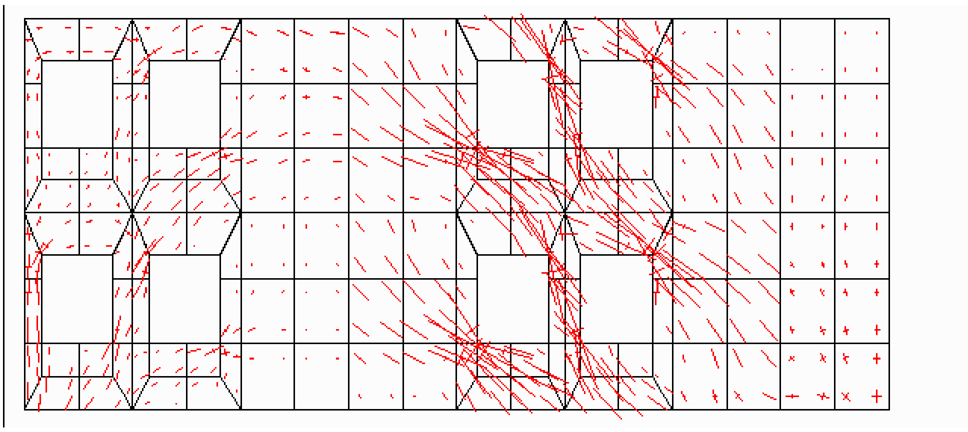
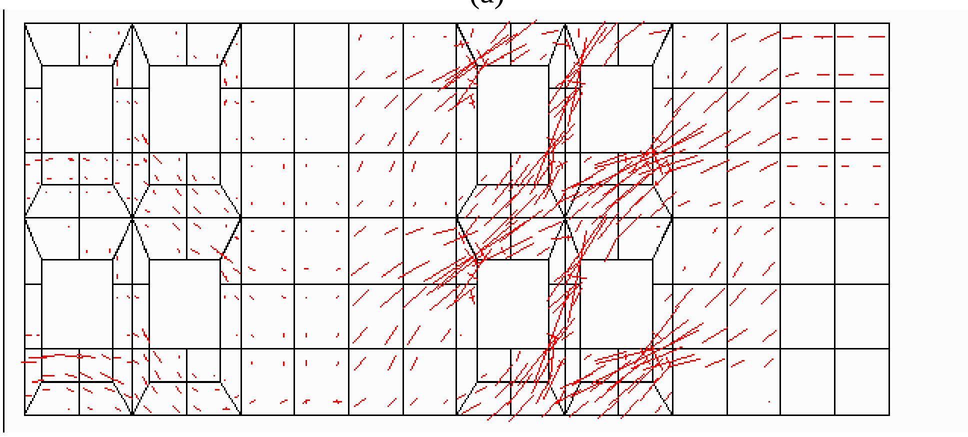



Figure 8 - Displacements in *z* direction for the analysis of the reinforced structure.

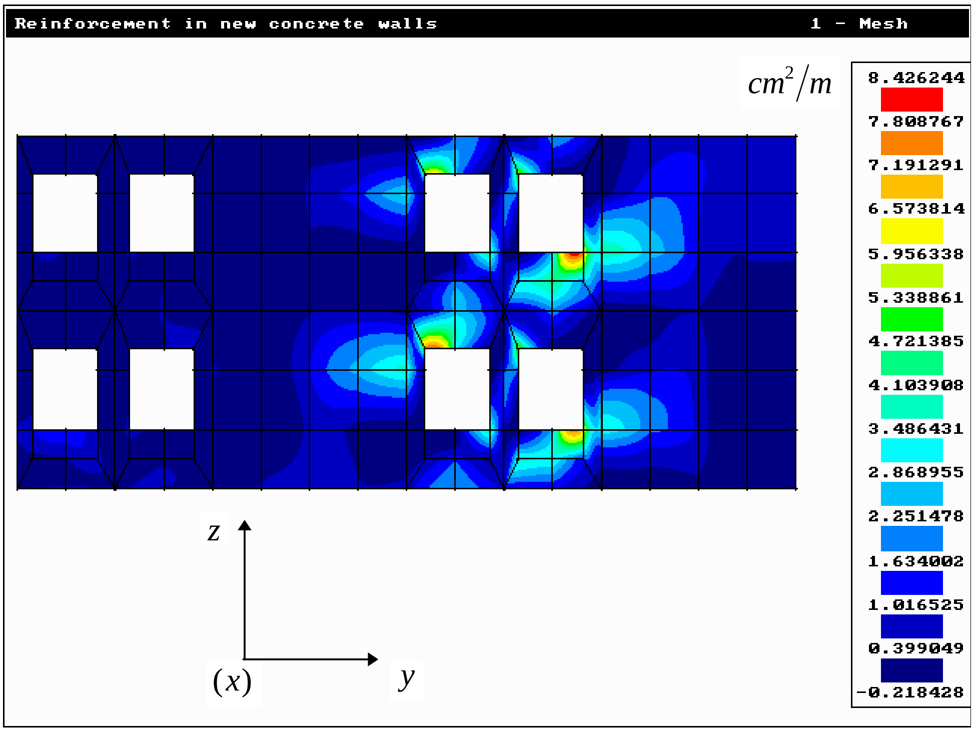


(a)

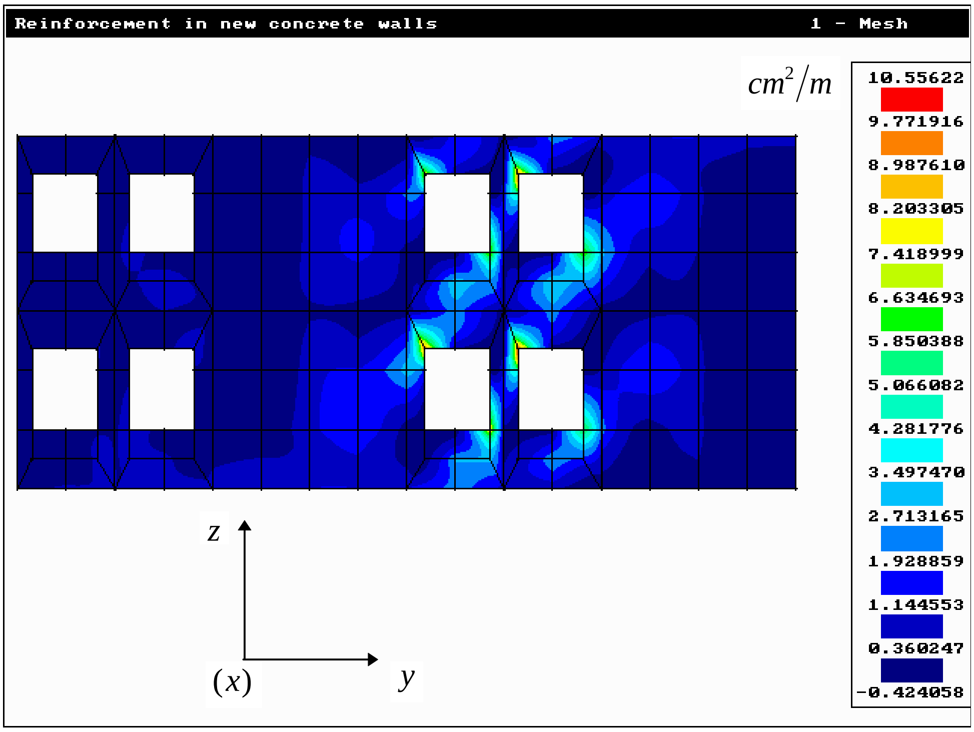


(b)

Figure 9 - Compression (a) and tension (b) membrane principal forces in the reinforcing walls.




(a)




(b)

Figure 10 - Reinforcement area at interior surface in the *y* (a) and *z* (b) directions (almost equal to the area at exterior surface).

The soil pressure varies from 10 to 55 kN/m$^2$ (see Figure 11). The maximum value of this range is about the allowable bearing pressure estimated from the geotechnical analysis.

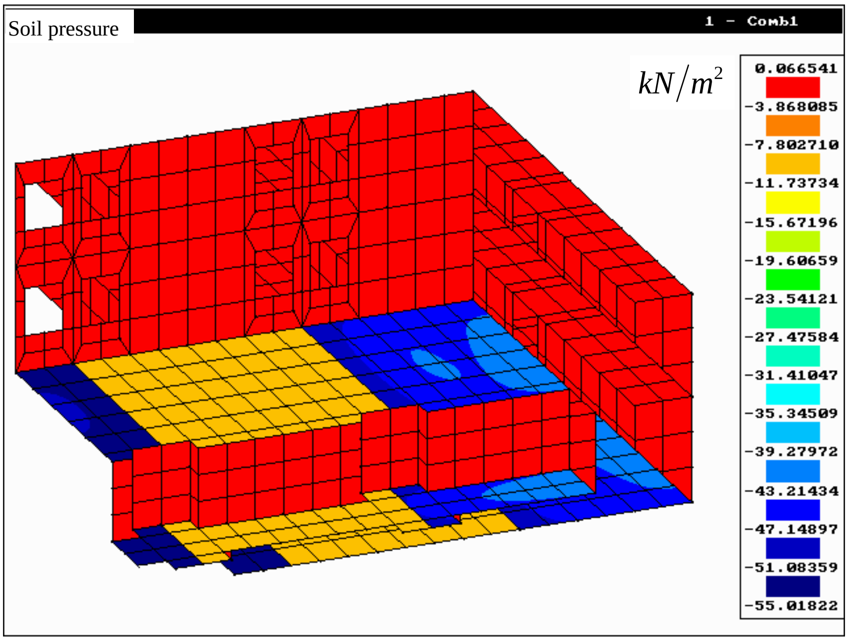


Figure 11 - Soil pressure in the reinforced structure.

## 4 - CONCLUSIONS

In this work a structural analysis of a damaged building is performed. The soil and concrete properties were evaluated from experimental tests.

Considering the soil homogeneous, the building settles uniformly and it doesn't suffer any damage. However, if the support conditions are heterogeneous the building suffers non uniform settlements, which cause the development of cracking in the air brick walls. The numerical results obtained with the computer program are in accordance with the deformation and cracking observations in the building.

In order to sustain the damage evolution, a rehabilitation solution is proposed. The interior panel of the building façades in air brick masonry are replaced by reinforced concrete walls. With this reinforcement the structural rigidity is increased, the cracking in the façades of the air brick masonry is sustained and the soil pressure is reduced. This rehabilitation is technically and economically competitive comparatively to other rehabilitation solutions.

## 5 - REFERENCES

Azevedo, A. F. M., Barros, J. A. O., Manual do programa FEMIX - versão 2.3, FEMOPEN Consultoria e Software de Engenharia, Lda, Porto, Portugal, 1997.

Barros, J. A. O., Comportamento do betão reforçado com fibras - análise experimental e simulação numérica, Doctoral thesis presented at the Civil Eng. Dept. Faculty of Engineering, University of Porto, Portugal (in Portuguese), 1995.

Bowles, J. E., Foundation Analysis and Design, 4th ed., **McGraw-Hill Book Co**, New York, 1993.

Carvalho, F. R., Comportamento mecânico de alvenarias. Influência de alguns parâmetros nas suas resistências à compressão e ao corte. LNEC, Lisboa, 1990.

CEB-FIP Model Code, Comite Euro-International du Beton, Bulletin d'Information nº 213/214, Ed. Thomas Telford, 1993.

Zienkiewicz, O.C.; Taylor, R.L., The finite element method, Fourth Edition, 1989.

## 6 - ACKNOWLEDGEMENT

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