CHARACTERIZATION AND STRENGTHENING OF A "GHOST" BUILDING

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ABSTRACT

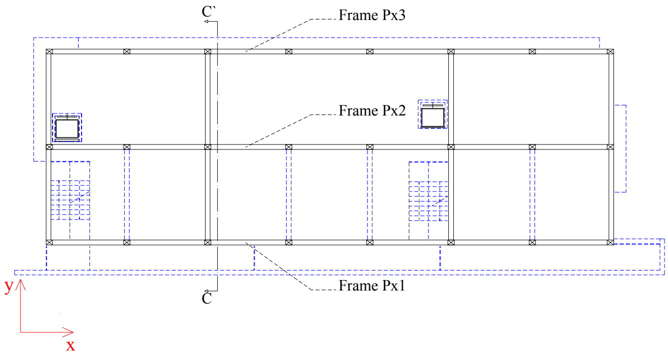
In the middle of the eighties it was intended to build a one family dwelling at a North Portuguese Region, but a much bigger edification was constructed, without any design elements. At the end of the nineties this construction was acquired, and another architectonic and functional configuration was designed for this space. Since there were not any elements available for the existent construction, it was carried out several strategies for its geometrical, structural and material characterization. These elements gave the indispensable information for analysing the structural stability of the building, which revealed to be necessary to strengthen foundations, beams and columns. The procedures for characterizing the construction, the structural stability analysis and the strengthening strategies are described in the work.

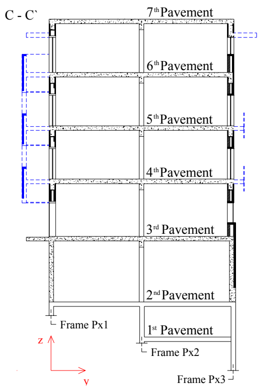
1. INTRODUCTION

Figure 1 represents a frontal view of one family dwelling, designed in the middle of the eighties to be built in a North region of Portugal. However, instead of this little house it was built the construction shown in Figure 2. A structural floor and a cross section of this building are represented in Figures 3 and 4. According to the knowlegment of the autors of the present work, the construction was done without any design element, and was interrupted in the phase when the brick walls were setting up, in 1992. Since then, the construction was abandoned, up to the end of the nineties, when it was bought, and a new configuration was planned for the space occupied by this building. The new architectural design foresaw the use of the existent areas of the building, with the construction and demolition of some structural elements. To assess the ability of the existent structure for supporting the new exigencies, a study was demanded to the Laboratory of the Civil Engineering of Minho University, envolving the determination of all the elements necessary to verify the structural stability. To perform this task, the geometry of the structural elements was measured, some properties of the soil foundation, concrete and reinforcing bars were estimated using experimental tests. The structural stability has revelead that, a large strengthening intervention should be done on the existent construction. The present work describes, briefly, the studies performed from the building characterization up to the proposed strengthening solutions.

Figure 1 – Expected building.

Figure 2 – Existent construction.





Legend: dashed line – new elements; solid line – existent elements.

Figure 3 – Typical structural plant of the building.

Figure 4 – Transversal section of the building.

2. ASSESSING THE CHARACTERISTICS OF THE BUILDING

2.1 Properties of the soil foundation

The construction was supported on a too weathered and fractured gneissic-schistous laminate formation, and residual clayey soil of variable compacity. From the dynamic penetration heavy cone tests (DPH) it was verified that significant part of the soil foundation was poorly compacted, with high variety of properties in plant and in deepness. Based on the results obtained from DPH and taking into account the level of the bulding foundations, it was estimated a 0.2-0.3MPa for the soil allowable stress under service loads.

2.2 Geometry and materials of the structural components

2.2.1 Geometry

To perform the analysis of the structural stability of the building it was necessary to measure the dimensions of the beams, columns and foundations. Due to the difficulties for assessing the foundations, the dimensions of the unassessed footings were estimated using assumptions, like the structural symmetry of the building. Figure 8 includes the dimensions of the footings considered in the structural analysis. A thickness of 0.75 m was assumed for all the footings. All the beams have a cross section of 0.3m width and 0.5m height. The columns from the footings up to the fourth pavement have a cross section of 0.5m and 0.3m in x and y direction, respectively (see Figures 3 and 4). From the fourth up to the seventh pavement, the columns have 0.4m and 0.3m dimensions in x and y direction, respectively.

2.2.2 Concrete

The concrete compression strength of the structural elements was estimated using two approaches: uniaxial compression tests on cores and Schmidt esclarometer tests. With the aim of obtaining a correlation between the results of these two approaches, the Schmidt test was carried out on the concrete to be extracted for doing the specimens for the uniaxial compression tests. To avoid the influence of the weathered concrete surface on the results obtained from Schmidt eslarometer, a thin layer of the concrete surface was removed. The number and the places selected for performing the Schmidt esclarometer tests and for drilling the cores were established with the aim of obtaining representative results of all the building. To avoid that the core drilling procedure introduces significant injuries on the structural elements, it was given special preference for the elements that was predicted to be demolished and for the places with low impact on the structural stability. The uniaxial compression tests were carried out on cylindrical concrete cores, according to BS 6089 Standard (1981). A very low correlation was obtained between the Schmidt esclarometer index and the uniaxial compression strength, indicating that the Schmidt esclatometer index is influenced by several factors, giving a qualitative indication of the compression strength, only. This qualitative indication was used for estimating the concrete class of the elements where it was not drilled concrete cores. Using this methodology it was estimated the concrete classes indicated on Table 1.

Table 1 – Concrete classes of the structural elements.

<table>
<tr><th></th><th>Beams</th><th>Columns</th></tr>
<tr><td>Foundations</td><td>C12/16</td><td rowspan="4">C12/16</td></tr>
<tr><td rowspan="2">1st pavement</td><td rowspan="2">-</td></tr>
<tr></tr>
<tr><td rowspan="2">2nd pavement</td><td rowspan="2">C12/16</td></tr>
<tr><td rowspan="2">C12/16</td></tr>
<tr><td rowspan="2">3rd pavement</td><td rowspan="2">C12/16</td></tr>
<tr><td rowspan="2">C12/16</td></tr>
<tr><td rowspan="2">4th pavement</td><td rowspan="2">C20/25</td></tr>
<tr><td rowspan="2">C20/25</td></tr>
<tr><td rowspan="2">5th pavement</td><td rowspan="2">C20/25</td></tr>
<tr><td rowspan="2">C20/25</td></tr>
<tr><td rowspan="2">6th pavement</td><td rowspan="2">C25/30</td></tr>
<tr><td rowspan="3">C20/25</td></tr>
<tr><td rowspan="2">7th pavement</td><td rowspan="2">C20/25</td></tr>
<tr></tr>
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2.2.3 Reinforcement

For assessing the reinforcement arrangement of the beam and column elements it was done slits on these elements, according to the scheme represented in Figure 5. The percentage of stirrups on beams was evaluated from the slits made on the lateral faces of the beams, near the columns, where the shear forces are higher. The longitudinal reinforcement on top and on bottom surfaces of the beams was assessed from the slits on top, near column, and bottom, at midspan, respectively. The amount of hoops on columns was evaluated on the longitudinal slits introduced in these elements, while the longitudinal reinforcement was estimated from the transversal slits. The details of the reinforcement arrangement obtained with this procedure were represented elsewhere (Barros 2001).

To evaluate the type of some steel bars applied in the structural elements, uniaxial tensile tests were carried out on a servo-controlled machine with specimens extracted from elements that were predicted to be demolished. Figure 6 represents the stress-strain relationship registered in these tests and includes the average data obtained. The results obtained are representative of the ribbed steel class A400 NR (REBAP 1999), that was assumed on the analysis of the building structural stability.

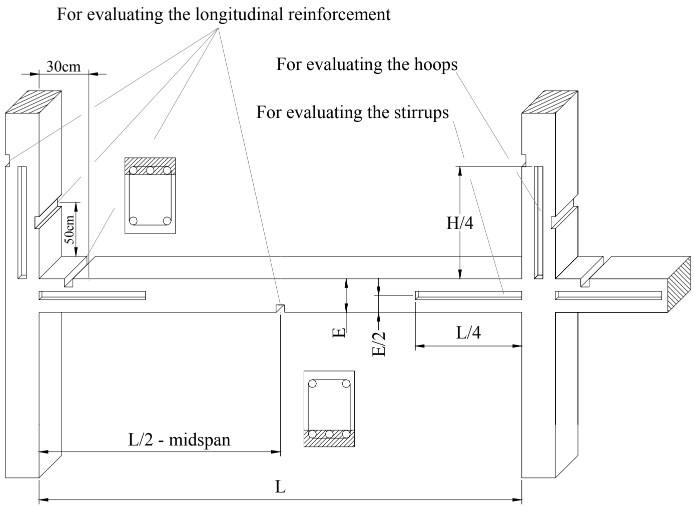
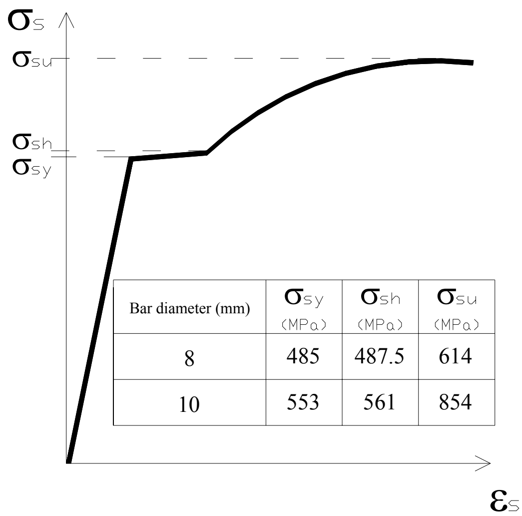


Figure 5 – Slits applied on beam and column elements for assessing the reinforcement arrangement.



Bar diameter (mm)	σ_{sy} (MPa)	σ_{sh} (MPa)	σ_{su} (MPa)
8	485	487.5	614
10	553	561	854

Figure 6 – Results obtained on the uniaxial tensile tests on steel bar specimens.

3. STRUCTURAL STABILITY

3.1 Introduction

The data obtained in the procedures described in previous chapter have given the indispensable information for analysing the structural stability of the building. A linear analysis was performed with a finite element computational code, discretizing the beam and columns with 3D Timoshenko bar elements, the pavements by rigid diaphragms, and the elevator reinforced concrete compartments with shell elements. In this analysis the new structural elements were also integrated. The ultimate and the serviceability limit states were verified for the most unfavourable combinations of the load cases prescribed in the Portuguese Code (RSA 1983).

3.2 Beams

Comparing the positive and the negative resistant design moments of the cross section beams with the corresponding highest actuating design moment, the beam cross sections that need to be strengthened were evaluated. The same was made for beam shear resistance. In this safety analysis it was also considered the minimum amount of reinforcement and the maximum distance between reinforcing bars, according to the Portuguese Code (REBAP 1999). From the results obtained it was verified that about 15% of the beams needs to be strengthened for the positive moments (insufficient reinforcement at bottom surface), 72% for the negative moments (insufficient reinforcement at top surface), and 56% for the shear forces.

3.3 Columns

The safety of the column cross sections was estimated using the expression proposed in the Portuguese Code (REBAP 1999):

$$\frac{M'_{Sd,x}}{M_{Rd,xo}} + \frac{M'_{Sd,y}}{M_{Rd,yo}} <= 1 \tag{1}$$

where $M'_{Sd,x}$ and $M'_{Sd,y}$ are the actuating design bending moments in the principal inertia axis of the cross section, $M_{Rd,xo}$ and $M_{Rd,yo}$ are the resistant design bending moments in these axis, evaluated in plane bending with the design axial force N_{Sd}. Applying the expression (1) to the critical sections of all the columns, it was obtained the cross sections that do not accomplish this condition. From the results obtained it was concluded that, excluding some columns of the highest floor, the remainder should be strengthened.

3.4 Foundations

Taking the data obtained in the geotechnical prospection and the data indicated in specialized bibliography (Fang 1991, Branco and Correia 1990) it was estimated an angle of internal friction of 10^o, a cohesion of 0.02 MPa, a poisson ratio of 0.35 and a Young modulus of 30 MPa for the soil foundation. A footing was considered stable if its load bearing capacity (Bowles 1993), its resistance to punching and its resistance to wide-beam shear were simultaneously verified for the most unfavourable loadings transferred by the columns. Figure 7 represents the safety factors obtained for the footings, corresponding to the load bearing capacity ($lbc = q_{Rd} / q_{Sd}$), punching ($pun = p_{Rd} / p_{Sd}$) and wide-beam shear ($wbs = w_{Rd} / w_{Sd}$), where the subscript *Sd* represents the design actuating value and *Rd* the design resistant value. From the results it was verified that any of the footings of the frames Px1 and Px2 fulfill the safety requirements analysed. The main problem affecting the footings of the frame Px3 is its reduced thickness, do not assuring the safety for the punching and wide-beam shear. The shorter footings of the Px3 frame do not also fulfill the safety requirements for the load bearing capacity.

4. STRENGTHENING STRATEGIES

4.1 Introduction

The analysis of the structural stability of the building has revealed that, the major part of the columns and footings should be strengthened, as well as, a significant number of beams. Due to the low class of the concrete, considering that, in general, the concrete cover of the structural elements are in poor conditions, and taking into account the amount of reinforcement to be applied, the use of fiber composite polymers (FRP) for increasing the load bearing capacity of these elements is technically and economically disadvantageous, when compared with other reinforcing strategies (CEB-FIP 2001).

After analysing the possible scenarios, the owner of the building has opted for a strengthening solution. A micro-concrete of low shrinkage, high workability and strength class C20/25 was proposed for the strengthening campaign. The weathered concrete cover of the elements to be strengthened should be removed before procede with the reinforcing operations, while the concrete in good conditions should be punctured with a proper device.

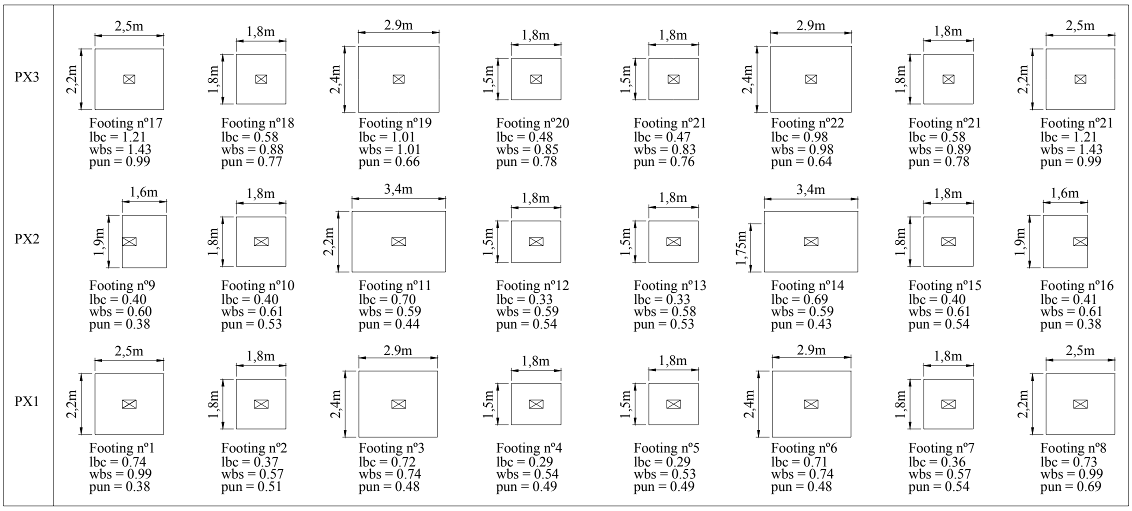


Figure 7 – Safety parameters for the footings.

4.2 Numerical model

To evaluate the behaviour of a strengthened cross section it was used a computational code, supported on the fiber section model (Barros and Sena 2001). This model can be applied on a cross section of any shape and submitted to an axial force and two bending curvatures (see Figure 8). The cross section can be composed of different concrete, steel and composite materials. To be useful for strengthening design, the model can simulate materials with a given initial strain and stress. The finite element mesh discretizing the cross section is dependent on the geometry and on the materials composing this cross section. The moment-curvature relationships are obtained taking into account the material constitutive laws, the equilibrium and the kinematic equations. Appropriated constitutive laws simulate the monotonic and the cyclic behaviour of the aforementioned materials.

The monotonic compression behaviour of plain concrete (PC) is simulated by the expressions proposed by the CEB-FIP Model Code (1993). The confinement provided by conventional stirrups or hoops is taken into account using the model developed by Scott *et al.* (1982). The formulation proposed by Thompson and Park (1980) was used for governing the unload-reload branches. Up to cracking, the PC tensile behaviour is simulated by the Young modulus and the axial tensile strength, whereas the post-cracking behaviour is modelled by the fracture parameters (Barros and Figueiras 1999). The tensile stiffness of the unload-reload branches of the cracked PC is considered constant and equal to the tensile stiffness of the uncracked PC. The tension-stiffening model, that simulates the post-cracking behaviour of the concrete under the influence of the reinforcing bars, takes into account the reinforcement main properties and the concrete fracture parameters (Barros 1995). Steel rebars are modelled using the laws proposed by Menegotto and Pinto (1973).

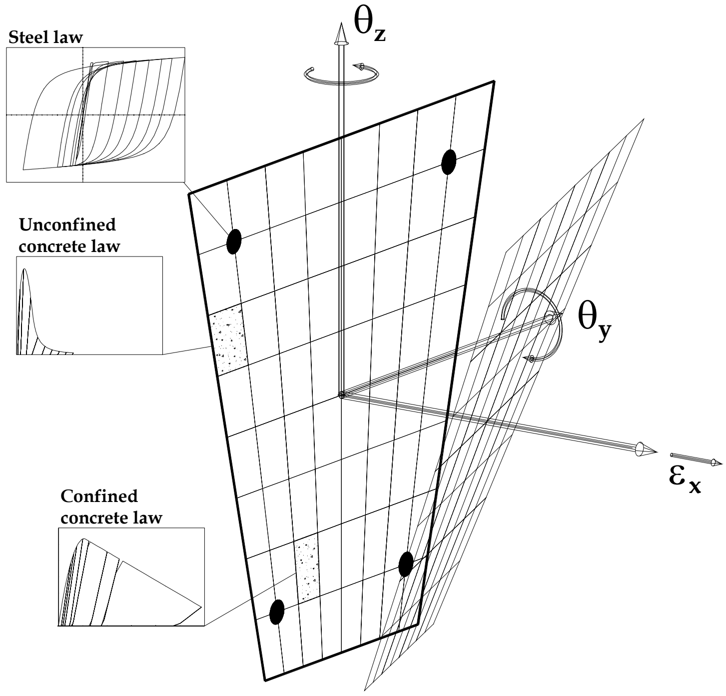


Figure 8 – Cross section model representation.

4.3 Beams

Figure 9 represents the reinforcing strategy proposed for the cross sections that needs to be strengthened for the negative bending moments and shear forces. The top and bottom concrete cover was increased 0.05m and 0.01m, respectively. An overlayer of 0.05m of lighweight concrete was proposed for the plates of the pavements in order to avoid geometric discontinuities between plates and strengthened beams. This new overlayer was linked to the concrete layer of the existent plates using a bounding compound. The top concrete layer of the existent plates should be scraped before applying the bonding compound, in order to increase the aggregate interlock between these concrete layers. A AQ50 (ϕ5#100) wire mesh should be placed at middle surface of the lighweight concrete overlayer for avoiding cracks due to shrinkage and temperature variation. Extra longitudinal bars were applied in cross section bottom surface for supporting the reinforcing stirrups. For accommodating the reinforcing stirrups, the lateral concrete cover increased 0.05m. Figure 10 illustrates the solution proposed for increasing the resistance to positive and negative moments, as well as, to shear forces. This solution is similar to previous one, but a higher internal arm was obtained by adding a layer of 0.1m to the bottom cross section.

Figure 11 represents the column-beam join, showing the continuity assured for the reinforcing longitudinal bars introduced into the beams.

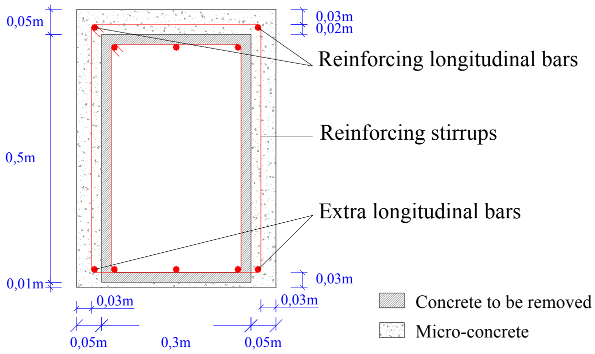


Figure 9 – Scheme of the strenghening strategy for increasing the resistance to negative moments and shear forces.

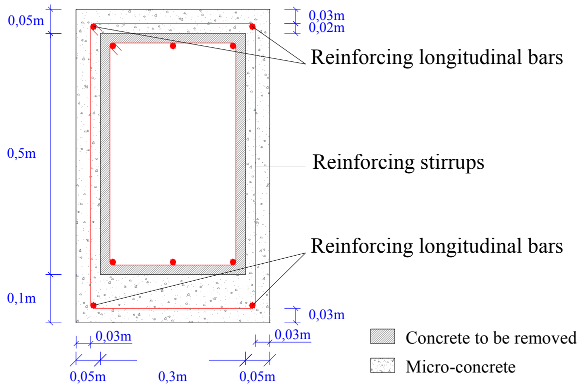


Figure 10 – Scheme of the strenghening strategy for increasing the resistance to positive and negative moments, and to shear forces.

4.4 Columns

After a previous estimation of the strengthening solution for each critical section of the columns, the moment-curvature relationship of these cross sections was evaluated using the model descrived in section 4.2, from which it was obtained the design resistant moment, and the stress and the strain in each concrete (existent and added) fibrous and steel bars (existent and added), for the most unfavourable load combinations. Due to the reduced cross section area, low concrete class and high axial compression forces, the cross section of a column to be strengthned was enlarged and extra longitudinal and transversal reinforcement were added, according to the scheme shown in Figure 12.

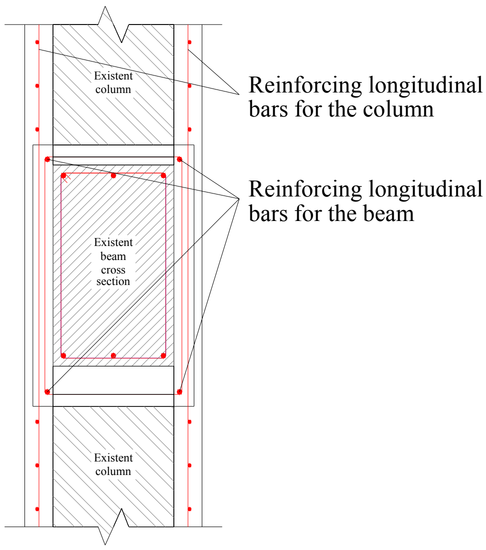


Figure 11 – The continuity of the reinforcing bars of the beams was assured.

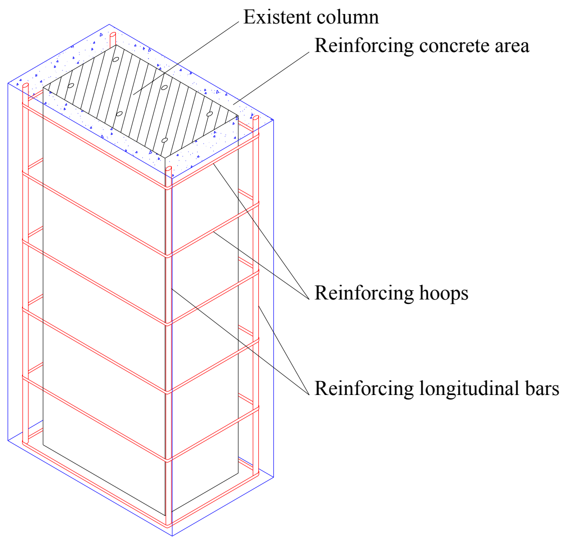


Figure 12 – Strengthening solution for columns.

4.4 Foundations

Chapter three has shown that all the footings of frame Px1 and Px2 do not fulfil the safety requisites of the load bearing capacity, the punching, and the wide-beam shear. The strategy proposed for accomplishing these safety requirements was transform the isolated footings to a continuous footing. In the larger footings of the frame Px3, that failed by punching and wide-beam shear, the safety was assured increasing its thickness. For fulfilling the safety requirements of the shorter footings of this frame, it was necessary to enlarge and to increase the thickness of these footings. The analysis of the strengthened strategy was performed with a finite element computacional code (Álvaro e Barros 2000). The finite element mesh was conceived in order to simulate the state corresponding, not only, to the phase before converting the isolated footings to one continuous footing (phase I), but also, the final state (phase II). In phase I it was considered the loading corresponding to dead weight of the structural elements existing in this phase. The strain and stresses induced in the footings and in the soil by the loading of the phase I was assumed as an initial state on the analysis corresponding to phase II. The soil reaction modulus was obtained with the data estimated from the geotechnical study and considering the recommendations of Gazetas and Hatzikonstantinou (1988). Figure 13 represents the soil pressure at the end of the phase I (only the dead load was considered), and Figure 14 illustrates the increment of the soil pressure due to the live load (design values) to be applied on the building, for the footings of frame Px1. Adding the soil pressures corresponding to these two phases, it can be verified that, the maximum soil pressure is lower than the design value of the resistant soil pressure (≅450 kPa).

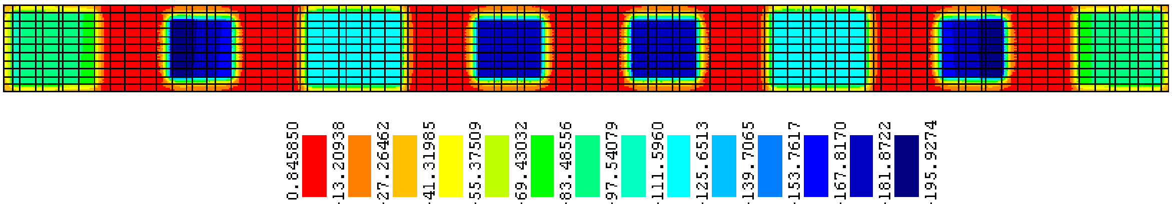


Figure 13 – Soil pressure (kPa) at phase I, at footings of frame Px1.

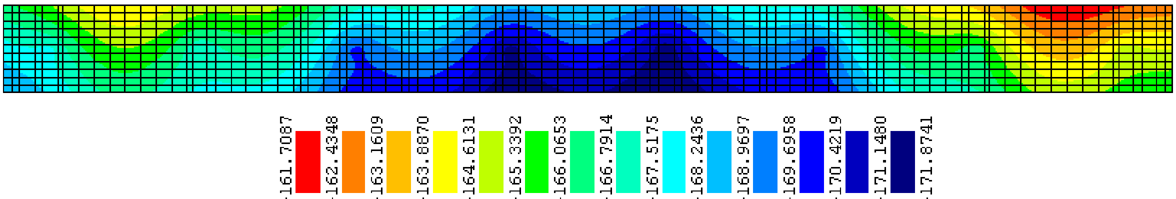


Figure 14 – Soil pressure increment (kPa) corresponding to phase II, at footings of frame Px1.

Steel connectors were used for linking the existent footings to the enveloping concrete (see Figure 15). The connectors applied on the top surface of the footings were designed for taking the force resulting from the difference between the axial force transmitted by the column (N_{sd}) and the resultant force due to soil pressure on footing bottom surface (P_s).

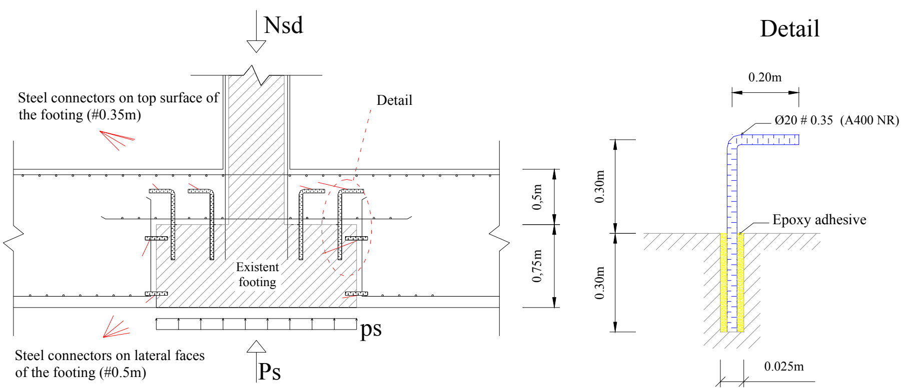


Figure 15 – Strategy for strengthening the footings.

5. CONCLUSIONS

To assess the structural stability of an abandoned building, without any design elements available, it was carried out several procedures for obtaining the indispensable characteristics of this construction. To accomplish this task several experimental tests were performed. The structural analysis of this building, submitted to the new architectural exigencies, has revelead the necessity of strengthening significant number of their structural elements. A numerical model, that is able of predicting the behaviour of a strengthened cross section, was used for designing the strengthening strategies proposed, and described in the present work.

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