Seismic Assessment of an Old Reinforced Concrete Building in Lisbon

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ABSTRACT: The construction history in Lisbon can be divided into the following five periods, *Pré-pombalino*, *Pombalino*, *Gaioleiro*, a mix between reinforced concrete and wood/masonry and the reinforced concrete period. The latter, started during the mid 1950's when the first buildings with only a reinforced concrete structure started to appear, and on the same period so did the first codes.

The codes have suffered several revisions, making them more demanding, especially regarding the seismic design criteria. Within this context the present work is developed, where a reinforced concrete building from the 1960's in Lisbon was seismic assessed and the main vulnerabilities identified. A survey of the building's characteristics, both architectural and structural, was done with the purpose of evaluating its seismic capacity and seismic performance according to the current seismic codes.

The building and its surroundings were modelled in the software SAP2000 and submitted to a seismic evaluation, based on a nonlinear static analysis and considering the N2 method. According to the Part 3 of Eurocode 8, the most vulnerable structural elements were identified, as well as the impact on considering the stairs on this type of models, which is not a common practice in design offices.

According to the results obtained, several retrofitting solutions were suggested and discussed and the most suitable one is proposed to overcome the deficiencies identified in the seismic assessment.

Keywords: Reinforced concrete building, seismic assessment, seismic retrofitting, nonlinear static analysis, Part 3 of Eurocode 8

1. INTRODUCTION

Due to its dimension and vast diversity of buildings, Lisbon may represent the construction history in Portugal. Most of this city's built-up area was construct before the first seismic codes. This evolution can be divided into five different periods:

- (Until 1755) – *Pré-pombalinos* buildings
- (1755 – 1880) – *Pombalinos* buildings
- (1880 – 1940) – *Gaioleiros* buildings
- (1940 – 1960) – Reinforced concrete and wood/masonry buildings
- (1960 – Nowadays) – Reinforced concrete buildings

The first period represents all the buildings built until the earthquake of 1755. This construction period can be mostly found near the S. Jorge Castle and can be characterized by vertical elements made with various stone rubble masonry, and wooden elements. The flooring is supported by wooden beams that are simply supported on the walls. The roof type used in this period is the gable roof with a wooden structure.

After the earthquake of 1755, where a significant part of the buildings in Lisbon were destroyed, the reconstruction of the city was based on a series of standard procedures, with the purpose of being a faster and more efficient construction. The *Pombalinos* buildings can be characterized by their architectural impositions, like the limited number of floors and simpler façade designs. On the ground floors the buildings were built with masonry walls with direct foundations to the subsoil. On the upper storeys the Saint André's cross was introduced, a wooden structure filled with masonry allowing a better seismic response of the structure. Regarding the flooring, the support structure continued to be made with wooden beams supported on the walls.

By the year 1880, Lisbon's urban area expanded significantly and neighbourhoods like the *Avenidas Novas* and *Campo de Ourique* appeared. This fast growth lead to a more reckless construction with both taller and wider buildings, where the strong masonry walls used on the ground floors were replaced by a poorer quality masonry or by *tabique* walls, which are composed by a slim wooden structure filled with masonry rubble. On the upper storeys the support structure became simpler with the wooden beams being simply supported on the walls.

By 1940, reinforced concrete started being used on construction in specific areas, such as balconies, kitchen and sanitary installations.

Starting on 1955, the first examples of reinforced concrete (RC) buildings appeared, together with the first seismic code. Based on the knowledge at that time and the requirements proposed by the code lead to RC structures poorly design with inadequate amount of longitudinal and transversal reinforcement.

Since the first code has been developed, there has been constant revisions that lead to updated versions. In this context, a study was then developed for a reinforced concrete building, of the 1960's, where the seismic performance was evaluated and retrofit solutions proposed.

2. CASE STUDY

The case study on this dissertation is a building located in the *Avenida da Igreja*, in the *Alvalade* area. It is a reinforced concrete framed structure and can be considered as a representative example of this typology, i.e. RC framed buildings built before 1970. The building was built in 1967 and has five storeys above ground and one bellow with 2483m^2 of gross construction area and 331.2m^2 of deployment area.

Figure 1 - Case study building

This building has a total of eleven units, nine for residential purposes and two retail units located on the ground floor. On the rooftop there is the technical area for the two elevators as well as the storage units for both the apartments and the condominium. The basement is composed by two storage areas to be used by each retail unit.

With a floor to ceiling height of 3.06m in each storey, the building has a total height of 20.5m above street level and in plan, the length of 27m and a depth of 10.55m.

I. Structural characterization

The structure design of the building was made accordingly to REBA [1] with the support of the technical tables supplied by LNEC [2].

The foundations of the building were design accordingly to the simplified Mörsch theory which is based on the compression struts that are still nowadays the bases for this type of design.

As showed in the next figure, the vertical elements in the middle of the building have isolated foundations and the ones on the outer perimeter are embedded in the retaining wall.

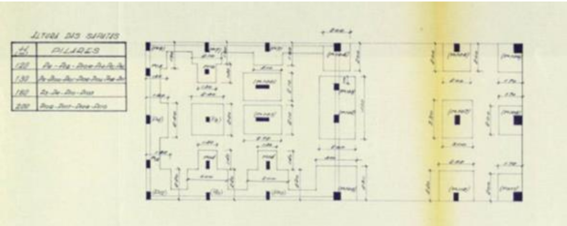

Figure 2 - Foundations plan

The reinforced concrete frame, columns and beams, were designed using the worst case scenario between hypotheses I and II defined in REBA [1], that differ mainly on the loads considered in the structure. The cross sections of the vertical elements vary from 0.30x0.30m to 1.15x0.20m, and the beams vary from 0.20x0.40m to 0.65x1.20m. The distribution of this elements may be verified on Figure 3.

The stairs of the building are approximately on its centre and were built with reinforced concrete and with a thickness of 0.15m.

The slabs on the building, with the exception of the balconies, and the stairs landing, are composed by prefabricated slabs with 0.17m thickness aligned in the directions showed in Figure 3.

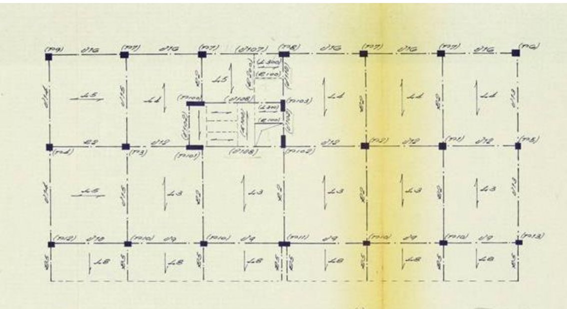

Figure 3 - Typical structural floor plan

II. Seismic vulnerability

A number of general building characteristics have been identified as being responsible for localized component deficiencies. These are identified in the following.

At a global level:

(i) Asymmetries in plan – When a horizontal load is applied on a building, a rotation around it's stiffness centre will have a significant impact on the performance of the vertical elements of the structure, specially on the ones in the borders;
(ii) Soft-storey – It occurs when one of the storeys of a building has lower stiffness than the others. This may lead to the concentration of the seismic damage on this storey.

At local level, which may be divided into two categories:

(i) Lack of confinement and ductility
(ii) Lack of shear capacity

Both these insufficiencies may cause a premature collapse of the elements, putting in risk the seismic safety of the building.

3. COMPUTATIONAL MODELLING OF THE BUILDING

A 3D model of the case study building was developed using SAP2000 [3], where it was evaluated both the dynamic characteristics of the building and its seismic behaviour. Every element and material used on the building's structure were defined according to the information acquired on the Technical Specifications of the Project [4].

I. Materials

Concrete

On the technical specifications of the project, three different types of concrete used on the buildings structure were identified: the B180 was used for the slabs and stairs; B225 was used for the beams and columns; and B300 was used for the foundations. Nowadays these types of concretes are no longer available. With the support of REBA [1] and EC2-1[5] a comparison was made to find the most similar concretes used in the current market. These comparisons indicated that the B180, B225 and B300 should be treated as C15/20, C20/25 and C25/30 respectively. Due to the lack of confinement verified in the RC elements of the building's structure, it was assumed a stress-strain relationship of an unconfined concrete.

The mechanical characterization of the concretes used is represented on the Table 1.

Table 1- Mechanical properties of the concretes used

REBA (1967)	B180	B225	B300
EC2-1 (2010)	C15/20	C20/25	C25/30
Weight density [KN/m^3]	25	25	25
Modulus of Elasticity [GPa]	29	30	31
Poisson's Ratio	0.2	0.2	0.2
Mean value for tensile strength [MPa]	24	28	33
Compressive strain at peak	1.9	2.0	3.5
Ultimate strain	3.5	3.5	3.5

Steel

The steel used as rebar in the reinforced concrete of the building is a A40 T_L from Heliaço, which, according to REBA, is a cold hardened smooth steel. Comparing its mechanical parameters, provided by the *Tabelas Técnicas para Engenharia Civil* [2] with the current code, it appears to be similar to a A400 steel. According to Caruso et al. [6], to get the accurate mechanical parameters for this type of steel, it is necessary to apply some reduction factors. The factors are related to the facts that smooth steel does not guaranty the same level of connection between the concrete and the rebars, which implies that the stresses that should be distributed along the rebar are all concentrated at the anchorage points. These anchorages however, do not have the same capacity to mobilize the similar stresses as standard systems. The reduction factors where applied to both the Elasticity modulus and the yield strength of the steel. The mechanical characterization of the steel used is represented on the Table 2

Table 2- Mechanical properties of the steel used

REBA (1967)	A400 T_L
EC2-1 (2010)	A400
Weight density [KN/m^3]	0
Modulus of Elasticity [GPa]	120
Poisson's ratio	0.3
Yield strength [MPa]	202
Ultimate strength [MPa]	259

Yield strain [‰]	0.028
Ultimate strain [‰]	3.5

Infill Masonry

According to the architectural technical specifications, all the infill masonry was done with hollow ceramic bricks, in a single panel specifically for the interior walls and on the external walls a double panel. For the definition of the mechanical parameters of this elements it was used the Italian norm [7]. The characteristics of the infill masonry walls are defined in Table 3.

Table 3- Mechanical properties of the infill masonry used

Type of masonry	Hollow ceramic bricks (percentage of interior puncture less then 45%)
Weight density[kN/m^3]	12
Mean Compressive strength [MPa]	4.0
Shear strength [MPa]	0.3
Modulus of Elasticity [GPa]	3.6
Distortion modulus	1.08

II. Modelling of structural and non-structural elements

Due to the existence of the reinforced concrete retaining wall, all the columns that are embedded in it were just modelled from the street level connection to the top. To replicate the restrictions that the retaining wall imposes on the columns, the translations and rotations in all directions at the connection were all restrained (X, Y, Z). For the foundations of the columns that are connected to the foundations of the retaining wall, due to the stiffness of this elements, all the translations and rotations were also restrained. For the isolated foundations only, the translations in the X, Y and Z directions were restrained, and the rotation stiffness was calculated based on the dimensions of each element and the soil characteristics.

The columns, beams and stairs of the frame structure were all modelled as linear elements. For the cross section of each element, it was considered the rebars of the end section of the elements and modelled using the Section Designer tool of SAP2000 [3].

The prefabricated slabs were modelled as *Shell thick* elements with no mass, due to the specific solution used, being the weight defined as a dead load of 2.4 kN/m^2.

The infill walls were modelled as two struts located at the diagonals of the column-beam frame. This struts can only carry compression loads as defined by [8].

III. Nonlinear modelling strategy

The modelling of nonlinear behaviour of elements can be done in several different ways. As shown in Figure 4 there are two main groups, the concentrated plasticity in the extremities of the elements by means of plastic or nonlinear spring hinges and the distributed plasticity that are defined by the finite length hinges, fiber sections or finite elements along the element.

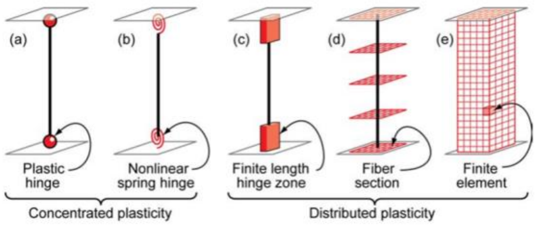

Figure 4 – Plasticity models for linear elements (adapted from[9]*)*

The concentrated plasticity models, used in this thesis for beams and columns, are based on the simplification that the nonlinear behaviour is concentrated on the extremities of the elements, either on vertical or horizontal elements. According to Paulay [9] definition of the length of the hinge will lead to values that are approximately 50% of the height of the cross section, but as Varum demonstrated on [10] the consideration of smooth rebars leads to values of around 25% of the cross section height, which can be calculated based on the following equation.

$$l_p = 0,04l + 0,011d_b f_y \qquad [1]$$

Were l_p stands for the length of the element, d_b for the diameter of the longitudinal rebar and f_y the yield strength of the steel.

Due to the fact that these are only subjected to axial loads, for the nonlinear behavior of the infills, a concentrated plastic hinge was placed in the middle of each element. The hinge was defined by a force/displacement relation that take the element to the collapse.

IV. Consideration of the contiguous buildings

The building used as a case study is part of a block with adjacent buildings on both sides. To take in consideration the impact of these buildings in the case study, they were also included in the model. Due to the lack of information on these buildings, it was considered that they both had the same characteristic as the case study. The 3D representation of all model is represented in Figure 5.

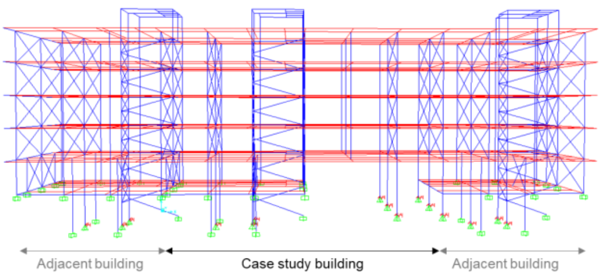

Figure 5 - 3D representation of the case study building considering both adjacent buildings

4. SEISMIC EVALUATION OF THE EXISTING STRUCTURE

According to EC8-3 [11] the seismic evaluation of an existing building can be made by two types of analyses, a linear and a nonlinear. On the present thesis, to adequately take into account the nonlinear behavior of the elements, the seismic assessment of the building was performed by nonlinear static analyses.

I. Performance Requirements and Compliance criteria

The performance verification of the elements considered on the seismic evaluation was made by the comparison of demand and capacity for the Significant Damage (SD) limit state, according to the definition on the Portuguese National Annex of the EC8-3[11]. The EC8-3[11] defines the following criteria for both Ductile and Brittle components:

- Ductile Components - $\theta_{SD} < \frac{3}{4}\theta_u$
- Brittle Components - $V_{Ed} < V_{Rd}$

Where θ_{SD} stands for the total chord rotation, θ_u for the ultimate rotation capacity as V_{Ed} and V_{Rd} stands for the design and resistant shear, respectively.

Ductile Elements

For the ductile elements EC8-3 defines that the analyses of the rotation capacity, θ, is characterized by the angle between the tangent to the element axis at the plastified extremity and the linear distance between that point and where M=0.

As Fardis defined on [8], the total chord rotation can be estimated by the following equation:

$$\theta = \chi_y \frac{L_v}{3} + (\chi - \chi_y) L_p \left(1 - \frac{L_p}{2L_v}\right) \qquad [2]$$

Where χ represents the imposed curvature, χ_y the yield curvature, $L_p$is the shear-moment ratio at the end of the element and L_v is the plastic hinge length.

The ultimate rotation capacity of the chord for the verification of the limit states can be calculated by the following equation:

$$\theta_{um} = \frac{1}{\gamma_{el}} 0{,}016(0{,}3^{v}) \left[\frac{\max(0{,}01;\omega')}{\max(0{,}01;\omega)} f_c\right]^{0{,}225} \cdot \left(min\left(9;\frac{L_v}{h}\right)\right)^{0{,}35} 25^{\left(\alpha\rho_{sx}\frac{f_{yw}}{f_c}\right)}(1{,}25^{100\rho d}) \qquad [3]$$

Were γ_{el} is equal to 1.5 for primary seismic elements and to 1.0 for secondary seismic elements, h the height of the cross section, v the relative normal force, ω and ω' the mechanical reinforcement ratio of the tension and compression longitudinal reinforcements, f_c and f_{yw} the concrete compressive strength and the stirrups yield strength, ρ_{sx} the transverse rebar ratio, ρ_d the diagonal rebar ratio and α the confinement effectiveness factor. According to the EC8-3 [11], for members with smooth rebars a 0.8 coefficient should be applied to the value of the ultimate chord rotation capacity.

Brittle Components

The shear capacity of each element was evaluated according to the shear capacity to resist cyclic loads of each element defined by EC8-3, to access the impact of brittle mechanisms in the structure seismic behavior..

II. Modal Analysis

The evaluation of the dynamic properties of the structure was made by means of a modal analysis. For the models with and without the consideration of the stairs, developed in SAP2000 [3], these analyses allowed a characterization of the main modes of vibration,$\emptyset_n$, the periods, T, and the contributions of the main effective masses on the directions X, Y and of the torsion around Z (M_{xn}, $M_{yn}$and R_{zn}).

In the analysis of the results obtained for both models, the stairs have a significant contribution on the dynamic behavior of the building. In fact, the stairs increase the rotation of the structure on both X and Y modes of vibration and reduce the period of the structure of the first main vibration mode. On Table 4, is presented the periods obtained for the main vibration modes for the structure with and without stairs.

Table 4 – Main periods of vibration for the structure with and without stairs

		Periods of vibration (s)		
Mode	**Direction**	**Without stairs**	**With stairs**	**Difference between periods (%)**
1	(X)	0,523	0,436	20%
2	(Y)	0,384	0,389	1,3%
3	(r)	0,320	0,346	7,5%

III. N2 Method

The N2 method is proposed by EC8-1 and consists in a simple method to calculate the target displacement of a structure. The method is divided into six steps:

- Step 1: Data analysis and definition
- Step 2: Single degree of freedom equivalent structure and capacity curve
- Step 3: Determination of the force/displacement relation
- Step 4: Determination of the period of the equivalent SDOF (Single degree of freedom) system
- Step 5: Seismic demand for the SDOF system
- Step 6: Seismic demand for the MDOF (Multiple degrees of freedom)

As the seismic assessment is for the Significant Damage (SD) limit state, the final ultimate displacement should be reduced by 25%. The values for both the target displacement and the reduced ultimate displacements are represented in Table 5.

Table 5 - Verification of the seismic performance

		Model with stairs				Model without stairs			
Direction	**Load**	d_u [m]	d_t [m]	$\frac{\frac{3}{4}d_u}{d_t}$		d_u [m]	d_t [m]	$\frac{\frac{3}{4}d_u}{d_t}$	
x+	Uniform	0,025	0,040	0,63	×	0,070	0,058	1,21	✓
x-	Uniform	0,031	0,039	0,79	×	0,074	0,052	1,42	✓
y+	Uniform	0,066	0,030	2,21	✓	0,067	0,029	2,28	✓
y-	Uniform	0,065	0,030	2,16	✓	0,063	0,029	2,23	✓
x+	Modal	0,032	0,049	0,65	×	0,073	0,071	1,03	✓
x-	Modal	0,026	0,051	0,52	×	0,075	0,062	1,20	✓
y+	Modal	0,070	0,037	1,89	✓	0,069	0,034	2,05	✓
y-	Modal	0,070	0,038	2,47	✓	0,065	0,033	2,67	✓

Regarding the Model without stairs and as shown in Table 5, the safety verification, for both directions and type of load, is verified.

The option of considering the stair in this type of analysis can be justified by the results obtained for the X direction. In fact, in this direction the effect of the stairs in the building's performance is evident as leads to the collapse of the columns which support the stairs; this occurs for both type of lateral load distributions adopted, i.e. the Uniform and Modal lateral loads distribution.

The results obtained also put in evidence the significant contribution of the infill masonry walls, especially during the elastic phase, albeit they were only modeled in the Y direction, the structure verified the safety parameters for both the models in this direction.

From the types of loads applied to the structure, the Modal X (-) was the one that lead the most severe results. For that reason, this load was defined as critical for the structure, and from now on, the structure and its elements are evaluated for this load.

Relative Displacements

EC8-1 defines, for the design of new structures, a limit for the relative displacements between stories.

This limit defines that a relative displacement for each storey, d_r, must be lower than 1.25% of the floor height, which in this case is equal to 3.06m.

Since the case study is for an existing building, this verification was done with the purpose of evaluate the impact of the stairs on the structure.

For each direction and lateral load distribution, the worst-case scenario was applied. On the Y direction, the effect of the soft-storey is clear, since the results on this storey are significantly higher than the ones above. In the X direction, the results clearer show that the stairs worsen the seismic behavior of the structure.

Effects of the stairs

By the analysis of the results obtained for the structures with and without stairs, it becomes clearer that the stairs should not be ignored when the seismic evaluation is performed, on either an existing building, or even in the seismic design of a new one.

The non-consideration of these elements is usually assumed as a conservative procedure, because allegedly the performance of the real structure will be superior than the one considered on the design. According to [12] this hypothesis can only be truthful when horizontal loads do not exist (e.g. in non-seismic areas).

The following figures demonstrate the damage distribution for the critical lateral load distribution and for the ultimate displacement of the building. In the model considering the stairs, it is evident that the damages are concentrated on the second storey, specifically at the connections between stairs and columns. The significant reduction of the rebars and cross section dimensions used, justify the damage distribution of the structure without stairs. The consideration of the stairs will aggravate the vulnerability of this storey; the columns were not design to take into account the landing slab of the stairs at half height which cause higher interstorey drifts on the columns and would lead to their premature collapse.

In the following figures the color scale represents the state of the plastic hinges for each element, were B stands for the yielding, IO for immediate occupancy, LS life safety limit, CP the collapse prevention and E for the collapse of the section.

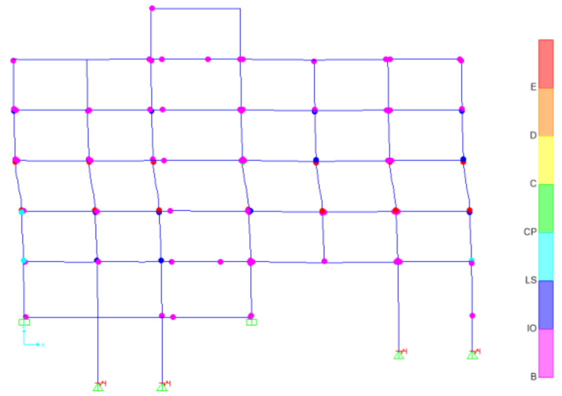

Figure 6 - Damage distribution for the structure without stairs

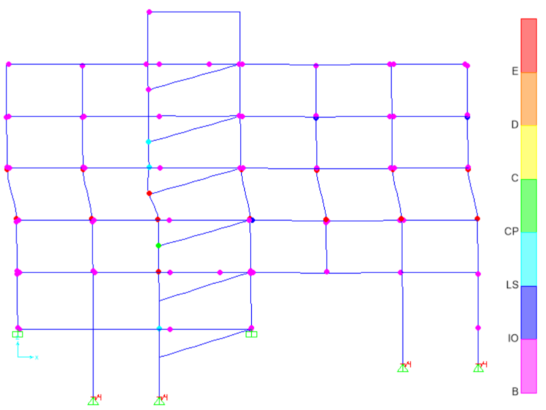

Figure 7 - Damage distribution for the structure with stairs

Safety verification of structural elements

According to Fardis [13], generally the main conditioning elements of an existing RC framed structure, are the columns, since the design of the beams usually do not present major problems. For that reason, Fardis suggests that the retrofit may gird to the vertical elements of a framed RC structure. On the present thesis, the safety verifications were checked primarily on the vertical elements.

Due to the low values of the interstorey drifts on the structure, the flexure demands on the vertical elements are reduced, with exception to the ones where the stairs land at half height. In Table 6 are represented the ductile safety verifications of the vertical elements, for each frame alignment shown on

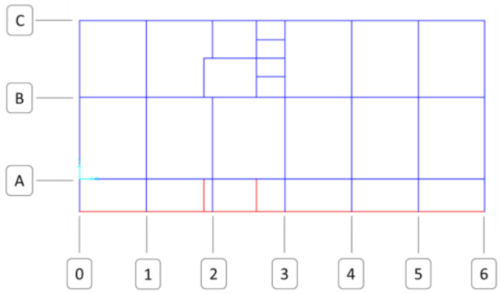

Figure 8 - Frame alignments considered on the structure

Table 6 - Ductile limit state verification of each vertical element

		Frame 0		Frame 2		Frame 3		Frame 5		Frame 6	
Alignment	Floor	θ/θ_{NC}	LS	θ/θ_{NC}	LS	θ/θ_{NC}	LS	θ/θ_{NC}	LS	θ/θ_{NC}	LS
A	0	2%	DL	2%	DL	1%	DL	2%	DL	1%	DL
	1	2%	DL	2%	DL	3%	DL	4%	DL	2%	DL
	2	2%	DL	2%	DL	3%	DL	4%	DL	2%	DL
	3	2%	DL	1%	DL	3%	DL	4%	DL	1%	DL
	4	1%	DL	0%	DL	1%	DL	2%	DL	1%	DL
B	0	1%	DL	4%	DL	2%	DL	2%	DL	0%	DL
	1	1%	DL	7%	DL	2%	DL	4%	DL	1%	DL
	2	1%	DL	959%	x	1%	DL	4%	DL	1%	DL
	3	1%	DL	12%	DL	1%	DL	4%	DL	0%	DL
	4	1%	DL	1%	DL	1%	DL	2%	DL	0%	DL
C	0	2%	DL	2%	DL	0%	DL	1%	DL	0%	DL
	1	2%	DL	2%	DL	4%	DL	3%	DL	2%	DL
	2	2%	DL	1%	DL	4%	DL	3%	DL	2%	DL
	3	2%	DL	1%	DL	3%	DL	3%	DL	2%	DL
	4	1%	DL	1%	DL	1%	DL	1%	DL	2%	DL

X – Element Collapse
DL – Verifies the Damage Limitation state
θ/θ_{NC} – Chord's rotation and ultimate rotation ratio

As expected, the columns where the stairs are supported, present ductile incapacity to withstand the rotations imposed, causing the premature collapse of these elements. For the others vertical elements, the Significant Damage limit state is verified.

In Table 7 are represented the shear safety verifications to the vertical elements.

Table 7 – Brittle limit state verification of each vertical element

		Frame 0		Frame 2		Frame 3		Frame 5		Frame 6	
Alignment	Floor	V_2/V_R	LS	V_2/V_R	LS	V_2/V_R	LS	V_2/V_R	LS	V_2/V_R	LS
	0	443%	X	669%	X	594%	X	346%	X	646%	X
	1	991%	X	1254%	X	933%	X	1328%	X	1113%	X
A	2	991%	X	1254%	X	933%	X	1328%	X	1113%	X
	3	169%	X	478%	X	259%	X	280%	X	266%	X
	4	34%	DL	77%	DL	148%	X	105%	X	18%	DL
	0	90%	DL	844%	X	817%	X	323%	X	747%	X
	1	737%	X	568%	X	1534%	X	2274%	X	959%	X
B	2	62%	DL	271%	X	1534%	X	2274%	X	959%	X
	3	207%	X	380%	X	528%	X	436%	X	240%	X
	4	737%	X	302%	X	249%	X	436%	X	40%	DL
	0	298%	X	332%	X	64%	X	261%	X	364%	X
	1	530%	X	729%	X	570%	X	824%	X	539%	X
C	2	530%	X	410%	X	570%	X	824%	X	539%	X
	3	168%	X	410%	X	99%	DL	280%	X	108%	X
	4	13%	DL	43%	DL	35%	DL	70%	DL	12%	DL

X – Element Collapse

DL – Verifies the Damage Limitation state

The results show that the majority of the vertical elements are conditioned by a brittle failure for low flexure demands, which can be justified by a lack of transverse reinforcement on these elements.

5. Seismic Reinforcement

IV. Intervention Strategy

For buildings were the structure showed incapacity to withstand the seismic demands a retrofitting solution should be analyzed to allow the verification of the seismic safety.

Given the vulnerabilities identified from the seismic evaluation, there are two different approaches which may be proposed:

(i) Global approach to the building which aims to reduce the seismic demands;
(ii) Local approach aiming to improve the deformation capacity of the structural elements.

If the number of structural elements which need to be retrofit is significant, a global approach should be taken, being the goal to improve the seismic behavior of the structure as a whole. If the insufficiency is located only on a few elements, a local approach should be taken.

Reduction of the seismic demands

The strategy to reduce the seismic demands for the existing structural and non-structural elements can be made by defining that the elements capacity is lower than the real one. This approach urges the necessity to include new elements to the structure so that it is withstands the seismic demands. From the different methodologies, the following three are distinguished:

(i) Strength and stiffness increase (inclusion of new elements, or retrofit of existing elements);
(ii) Seismic isolation;
(iii) Energy dissipation systems.

Strength capacity increase

During the past decades, a few studies have been developed around the best solutions to increase the strength capacity of a building as a whole. One of the solutions that has been gaining recognition along the years is the steel bracing. This solution can be applied on both the inside and the outside of the building, but it is on the outside that this solution is gaining recognition as it can be installed in a building without the need to relocate the inhabitants. In Figure 9 an example of steel bracing on a building is presented.

Figure 9 - Example of steel bracing on the exterior of a building (adapted from [14]*)*

Reinforced Concrete Jacketing

The reinforced concrete jacketing is the most common technique used in the retrofit of the RC elements, to increase stiffness and strength capacity. This technique consists in increasing the cross-section by means of a new layer of reinforced concrete. Assured by a well-known design and application techniques, this solution improves the element's:

(i) Stiffness
(ii) Deformation capacity
(iii) Shear strength resistance
(iv) Compression resistance
(v) Deficient lap-slices strength
(vi) Flexure resistance
(vii) Joint's shear resistance

The main disadvantages of this solution are: the increase of the cross-section of the element, possible change of the cross-section geometry and the structural weight.

Reinforced Steel Plate Jacketing

Due to the current design methodologies, materials and applications, this solution is also commonly used in the retrofitting of existing elements, especially when the elements are near collapse, because it can mobilize almost all its capacity in a short period of time. This solution can improve the element's:

(i) Shear strength resistance
(ii) Deformation capacity
(iii) Compression resistance
(iv) Deficient lap-slices strength

The solution's costs are higher than concrete jacketing and among the main disadvantages are cross-section limitations and fire safety issues.

Externally Bonded FRP Jackets

Comparing with the previous solutions presented, this is the more recent one on the market. The FRP, Fiber Reinforced Polymers, is a reinforcement fiber with a polymeric matrix material that can be applied on reinforced concrete elements to improve its resistance and deformation capacity. In terms of fibers, nowadays there are three types to consider: glass, aramid and carbon. One of the major advantages of this solution is that the jacket will be just a few millimeters thus causing a minimum weight increase. This technique, with a simple installation methodology, may improve the element's:

(i) Shear strength resistance
(ii) Deformation capacity
(iii) Deficient lap-slices strength
(iv) Low influence on the flexure resistance

The two major concerns on this technique are that the material used has an elastic behavior until it collapses, which means that there is no plastic reservation and a low fire resistance.

V. Retrofitting Proposal

The results of the seismic analysis indicated that the building's structure did not comply to the minimum requirements for the safety verification of the structure. It was verified that most the vertical elements had insufficient shear resistance. Moreover, for the columns of the stairs, the collapse is controlled by flexure. Due to the vulnerable seismic behavior of the building's structural elements the approach strategy to be followed should be global. This approach would include new elements in the structure to reduce the seismic demands upon the structural elements (see Figure 10).

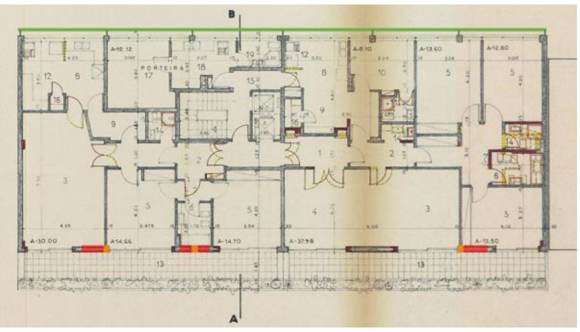

Figure 10 - Main Retrofit solution

The retrofitting proposal for the current building foresees a steel bracing on the back façade, represented in green in Figure 10), and the inclusion of reinforced concrete walls in the main façade, represented with yellow the existing columns and red the new walls. This solution, presented in Figure 10, has the advantage of having a minimum intervention in the building's architecture, being the steel bracing on the back façade the only visible change.

An alternative proposal would be to consider a mixed approach, with the introduction of new elements and the retrofitting of some of the existing columns. The retrofit of the columns of the stairs would be added to the proposal presented before, with the same retrofitting strategy as the walls on the main façade.

Regarding the presented proposals, for the validation of the retrofit solutions a new model should be developed, including the retrofit measures and the seismic assessment performed.

6. Conclusions

A six-storey framed RC building, located in Lisbon and built in 1967, was studied in this research. With a survey and a preliminary assessment of the structure some inadequate reinforcement detailing conditions were noticed. Moreover, some system and layout mechanisms and deficiencies were identified, namely: (i) vertical irregularity, as a result of changes in both the cross sections dimensions and the amount of reinforcement and (ii) the concrete-steel “bond-slip” mechanism due to the use of smooth rebars.

For the seismic assessment of the building, a 3D model was developed using SAP2000, where the infill walls were also numerical modelled.

The seismic evaluation was developed by performing nonlinear static analyses. These analyses showed the building does not verify the significant damage requirement of the code and needs to be retrofitted. The results achieved pointed out that most the vertical elements collapse in shear (due to insufficient shear capacity) and that the slabs of the stairs affect the seismic performance of the columns which support, at their half height, the stairs (collapse in flexure).

After analyzing the distribution of damage at the ultimate displacement and the progressive development of plastic hinges, it was possible to identify the structural members requiring an upgrade of their capacities. It is clear that the main causes for the collapse of this structure are the variation of the size and reinforcement area of the columns between storeys, the lack of confinement (transversal reinforcement) in columns and the lack of longitudinal reinforcement in columns located close to the stairs.

Based on the vulnerabilities identified on the structure’s seismic behavior, retrofitting strategies and solutions were proposed. Nevertheless, tt seems that the best-retrofit approach to the building should include a steel bracing structure on the back façade and new reinforced concrete structural elements on the main façade and, eventually, the retrofitting of the columns that support the stairs. This approach have a minimum impact in the architecture of the building.

With this retrofit proposal, it is expected to overcome the seismic vulnerabilities, however, it needs to be evaluated with new numeric model as well as new seismic evaluation analyses.

7. References

[1] Ministério das Obras Públicas, “Regulamento de Estruturas de Betão Armado,” 1967.

[2] V. Ferreira e B. Farinha, “Tabelas Técnicas para Engenharia Civil.” Associação dos Estudantes do I.S.T., 1977.

[3] CSI, “CSI Analysis Reference Manual,” 2013.

[4] CM Lisboa, “Memória Descritiva da obra nº57332.” Lisboa, Portugal, 1967.

[5] EC 2 Parte 1, “NP EN 1992-1-1 Eurocódigo 2 -Projecto de estruturas de betão Parte 1-1: Regras gerais e regras para edifícios,” 2010.

[6] C. Caruso, R. Bento, R. Sousa e A.A. Correia, "Modelling strain penetration effects in RC walls with smooth steel bars". Magazine of Concrete Research 71(17): 894–906, https://doi.org/10.1680/jmacr.18.0005, 2019.

[7] ATC, “ATC-40: Seismic Evaluation and Retrofit of Concrete Buildings, Volume 1 and 2,” 1996.

[8] M. N. Fardis, *“Seismic design, assessment and retrofitting of concrete buildings: based on EN-Eurocode 8.”* 2009.

[9] T. Paulay and J. N. Priestley, *Seismic Design of Reinforced Concrete and Masonry Buildings*. 1992.

[10] H. Varum, “Seismic assessment, strengthening and repair of existing buildings,” PhD thesis, *Univ. Aveiro Secção Autónoma Eng. Civ.*, 2003.

[11] EC 8 Parte 3, “EN 1998-3 - Eurocode 8: Design of structures for earthquake resistance - Part 3: Assessment and retrofitting of buildings,” 2017.

[12] M. Lopes, *Sismos e edifícios*. Edição Orion, 2009.

[13] M. N. Fardis, “Seismic Design, Assessment and Retrofitting of Concrete Buildings”, Springer, 2009.

[14] H. Varum, “Reabilitação e reforço sísmico de estruturas de betão armado”, Comunicação, 2008.