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STUDY OF THE SEISMIC VULNERABILITY OF A REMODELLED REINFORCED CONCRETE BUILDING IN LISBON

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Abstract

The main objective of this work is to assess the seismic vulnerability of a remodelled building in reinforced concrete, situated in Lisbon area. This evaluation is performed through the elaboration of a seismic vulnerability report through the analysis of the building seismic behaviour and the verification of the need to elaborate a seismic reinforcement project, if such report concludes that the building does not comply with structural safety requirements. In order to fulfil the objective, this work is divided into two parts: the first part concerns the application of the expedite methodologies I and II proposed by LNEC (2019); the second part, the seismic safety is verified for the reference method described in NP EN 1998-3 (2017), which also coincides with methodology III proposed by LNEC (2019), through a Pushover analysis.

Keywords: Seismic Vulnerability, Expedite Methods, Eurocode 8 part 3 Portuguese version NP EN 1998-3, Pushover Analysis, SeismoStruct.

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1 INTRODUCTION

The building that will be analysed under this seismic vulnerability assessment is a hotel located in Lisbon; but for data protection reasons, its location and ownership will not be identified. This building was designed in 1999 as an industrial building, consisting of 3 floors with high ceilings. However, in 2019 the building was subject to a remodelling, and two new floors were added between the existing floors, as well as new pillars, and some of the pre-existing ones were also reinforced (Figure 1). Another relevant aspect was the implementation of a lift shaft at each floor.

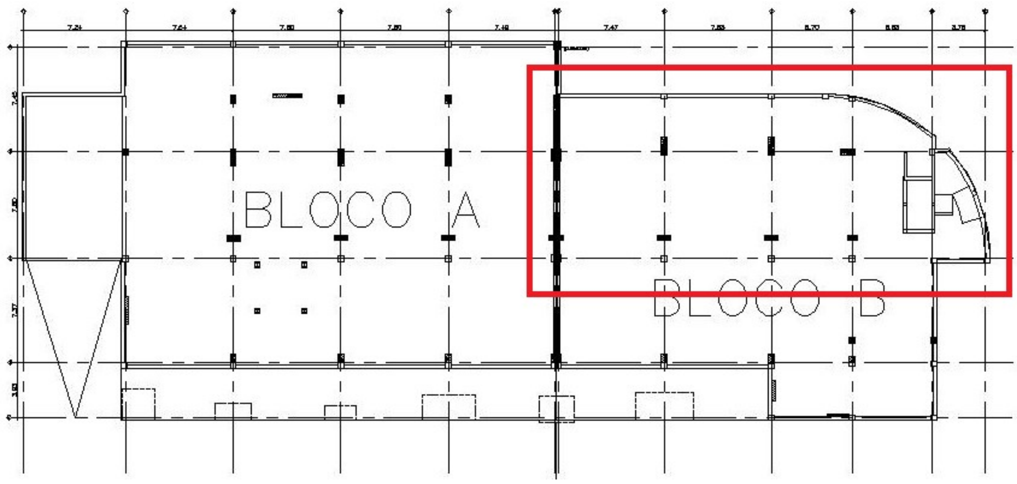

Fig. 1 – Part of the building to be studied - Floor plan.

2 EVALUATION OF THE APPLICABILITY OF THE EXPEDITE METHODOLOGIES I AND II IN THE BUILDING UNDER STUDY

Through the expedite methodologies I and II proposed by Report 81/2019 of LNEC [1], it is possible to evaluate the resistant capacity of the building subjected to seismic actions. However, it is necessary to verify certain criteria for their application, given in Tables 1-2-3.

2.1 Plant regularity and Structural regularity

Plant regularity	**Direction x**				**Direction y**			
Verification	**Floor**	**$0{,}3r_x$**	**$e_{o,x}$**	Verification	**Floor**	**$0{,}3r_y$**	**$e_{o,y}$**	Verification
$e_{0,x} \leq 0{,}30r_x$	Floor 1	7,86	7,00	**OK**	Floor 1	3,25	2,49	**OK**
	Floor 1	8,17	6,00	**OK**	Floor 1	16,07	15,65	**OK**
	Floor 1	8,17	6,00	**OK**	Floor 1	16,07	15,65	**OK**
	Roof	8,33	6,00	**OK**	Roof	17,29	16,49	**OK**
Verification	Floor	**r_x**	**l_{sx}**	Verification	**Floor**	**r_y**	**l_{sy}**	Verification
$r_x \geq l_s$	Floor 1	26,20	5,00	**OK**	Floor 1	10,82	8,87	**OK**
	Floor 2	27,22	5,00	**OK**	Floor 1	53,58	9,40	**OK**
	Floor 3	27,22	5,00	**OK**	Floor 1	53,58	9,40	**OK**
	Roof	27,76	4,00	**OK**	Roof	57,63	7,85	**OK**

Table 1- Verification of e_o, r_x and l_s, along x and y directions on each floor.

The boundary conditions considered for the columns correspond to built-in conditions. In this sense, Table 1 presents the verification of plant regularity; while Table 2 presents the verification of the non-existence of short columns.

Floor	Unfavourable section **(b_i x h_i)**	**h_i** of the unfavourable column (m)	Column heights - **H** (m)	**H/h_i**	**Verification** If $H/h_i \geq 2.5 \rightarrow$ Slender column
Floor 1	0,60x1,00	1,00	5,00	5,00	**OK**
Floor 2	0,75x0,25	0,75	4,00	5,33	**OK**
Floor 3	0,40x0,70	0,70	3,50	5,00	**OK**
Roof	0,40x0,80	0,80	3,50	4,38	**OK**

Table 2 - Verification of the inexistence of short columns.

2.2 General criteria for application of expedite methods

Criteria required for applying method I and II		**Criteria observed in the building**
Class of importance	I or II (NP EN 1998-1 [2])	II
Nº. of floors	Up to 4 Note: *In LNEC Report 81/2019 [1], buried basements are not considered as floors.*	4
Implantation area	Below 400 m^2	≈300 m^2
Structural regularity	No short pillars	*(Subchapter 2.1)*
Plant regularity	$e_{ox} \leq 0{,}3r_x$ (1) $r_x \geq l_s$ (2) - e_{ox} - distance between the centre of rigidity and the centre of gravity, measured in the x direction; - rx- radius of torsion; - ls- radius of gyration of the floor mass in plan.	*(Subchapter 2.2)*
	- The floor setback does not affect its rigidity; - The setback area is less than 5% of the floor area.	- No floor setbacks.
	- Floor stiffness sufficiently large in relation to the lateral stiffness of the vertical elements.	- Satisfies the rigid diaphragm condition.
	$\lambda = L_{máx}/L_{min} < 4$ (3) - $L_{máx}$ – largest dimension in the plan of the building; - Lmin - smallest dimension in the plan of the building	λ=31/12<2,6
Regularity in height	- Continuous structural systems in height; - Gradual reduction of lateral stiffness on each storey (kl); - Gradual reduction of the mass on each floor (mi).	- No discontinuity between floors; - Difference of kl of approximately 18% between floors; - Difference of mi of approximately 12% between floors.
No interaction between adjacent buildings	- Alignment between adjacent slabs lower than the thickness of the respective slab (LNEC Report 81/2019 [1]); - Height between adjacent buildings equal or less than 50% of the assessed one (LNEC Report 81/2019 [1]).	- No disparity between adjacent slabs; - Equal height between adjacent buildings.
Local geotechnical conditions	- Soil type A, B or C (NP EN 1998-1 [2]).	- Type C

Table 3 – Criteria of the expedite methods applicability for the building under study.

3 SEISMIC SAFETY EVALUATION ACCORDING TO METHOD II

3.1 Shear and flexural resistances

To determine the flexural and shear strengths of the vertical elements of the building, the expressions suggested in Report 81/2019 of LNEC [1] were applied, which are simplifications to those exposed in NP EN 1998-3 (2017) [3]. The first phase in the application of this method consisted in collecting data of the cross sections of all the vertical elements of the floors, only given in Table 4 for the most critical results obtained for floor 2.

Vertical elements	New				Existing				b_{wx} (m)	b_{wy} (m)	f_{yd} (Kpa)
	φw (mm)	branch-esSx	branch-esSy	S_h (m)	φw (mm)	branch-esSx	branch-esSy	S_h (m)			
PB' 6	8	2	4	0,15	-	-	-	-	0,25	0,75	434782
PB'7	8	2	4	0,15	-	-	-	-	0,25	0,75	
PB'5	8	2	4	0,15	-	-	-	-	0,25	0,50	
PB'8	8	2	4	0,15	-	-	-	-	0,25	0,75	
P31	8	2	2	0,15	0,28	4	4	0,15	0,60	0,60	289855
P37	8	2	2	0,15	0,28	2	2	0,15	0,60	0,60	
P47	8	2	2	0,15	0,28	2	2	0,15	0,60	0,60	
P33	8	2	2	0,15	0,28	4	4	0,20	0,60	0,60	
P29	8	4	2	0,15	0,50	3	2	0,25	0,80	0,40	
P38	-	-	-	-	0,28	2	2	0,15	0,40	0,40	
P39	8	2	2	0,15	0,28	2	2	0,15	0,60	0,60	
P40	-	-	-	-	0,50	2	2	0,25	0,40	0,40	
P41	-	-	-	-	0,50	2	2	0,25	0,40	0,40	
P42	8	2	2	0,15	0,50	2	2	0,25	0,60	0,60	
P46	8	2	2	0,15	0,50	3	2	0,25	0,60	0,40	
P68	-	-	-	-	0,28	2	2	0,20	0,30	0,30	
P53	-	-	-	-	0,28	2	2	0,20	0,30	0,40	
Lift Wall 1	10	6	2	0,15	-	-	-	-	0,20	1,95	434782
Lift Wall 2	10	2	4	0,15	-	-	-	-	1,75	0,20	
Lift Wall 3	10	18	2	0,15	-	-	-	-	0,20	5,25	
Lift Wall 4	10	2	6	0,15	-	-	-	-	2,15	0,20	

Table 4 - Cross section data for each vertical element on storey 2.

Equation (4) was applied to obtain the shear strength values for each vertical element, as shown in Table 5. Equally, Table 6 synthesizes calculation of flexural resistance also for each vertical element on floor 2, taken from the thesis of Pedreiras [4].

$$\rho_{sx} = \frac{A_{sx}}{b_w s_h} \tag{4}$$

Where: ρ_{sx} - percentage of transverse reinforcement parallel to the x direction of loading; A_{sx} - area of existing transverse reinforcement, parallel to the x direction; b_w - base of the cross section, perpendicular to x-direction of loading; s_h - stirrups spacing, in x-direction of loading. Thus, after analysing the shear and flexural strengths of all the vertical elements existing on each floor, it was possible to verify that the most unfavourable resistance corresponds to the flexural strength for each vertical element, as shown in Table 7.

Vertical elements	New	Existing	ρ_{swx} (%)	ρ_{swy} (%)	$V_{C,x}$ (kN)	$V_{C,y}$ (kN)	$V_{C,i}$ (kN)
	A_{sw} (cm^2)	A_{sw} (cm^2)					
PB' 6	0,50	-	0,27	0,18	209	135	209
PB'7	0,50	-	0,27	0,18	209	135	209
PB'5	0,50	-	0,27	0,27	132	132	132
PB'8	0,50	-	0,27	0,18	209	135	209
P31	0,50	0,28	0,23	0,23	338	338	338
P37	0,50	0,28	0,17	0,17	256	256	256
P47	0,50	0,28	0,17	0,17	256	256	256
P33	0,50	0,28	0,20	0,20	297	297	297
P29	0,50	0,50	0,25	0,27	315	360	315

P38	-	0,28	0,09	0,090	61	61	61
P39	0,50	0,28	0,17	0,17	256	256	256
P40	-	0,50	0,10	0,10	67	67	67
P41	-	0,50	0,10	0,10	67	67	67
P42	0,50	0,50	0,18	0,18	270	270	270
P46	0,50	0,50	0,21	0,27	200	261	200
P68	-	0,28	0,09	0,090	34	34	34
P53	-	0,28	0,09	0,070	45	36	45
Lift wall 1	0,79	-	0,16	0,53	252	847	252
Lift wall 2	0,79	-	0,53	0,12	758	174	758
Lift wall 3	0,79	-	0,18	0,53	759	2308	759
Lift wall 4	0,79	-	0,53	0,15	935	262	935

Table 5 -$V_{C,i}$ - Calculation of the shear strength of each vertical element i existing on floor 2.

Vertical elements	**Area of reinforcements**			**L_x** (m)	**L_y** (m)	**ρ** (%)	**f_{yl}** (kPa)	**$V_{F,x}$** (kN)	**$V_{F,y}$** (kN)	**$V_{F,i}$** (kN)
	Reinforced	Existing	Total							
PB'6	39,27	-	39,27	0,75	0,25	2,09	434782	117	52	117
PB'7	39,27	-	39,27	0,75	0,25	2,09		117	52	117
PB'5	31,69	-	31,69	0,50	0,25	2,54		74	45	74
PB'8	49,56	-	49,56	0,75	0,25	2,64		139	62	139
P31	16,08	38,18	54,29	0,60	0,60	1,51	289855	94	94	94
P37	16,08	29,64	45,73	0,60	0,60	1,27		83	83	83
P47	16,08	25,12	45,73	0,60	0,60	1,27		83	83	83
P33	16,08	25,12	33,17	0,60	0,60	0,92		65	65	65
P29	57,14	24,62	78,75	0,40	0,80	2,46		92	152	92
P38	-	20,60	20,60	0,40	0,40	1,29		34	34	34
P39	-	14,32	14,32	0,60	0,60	0,40		35	35	35
P40	-	42,02	42,02	0,40	0,40	2,63		58	58	58
P41	-	14,32	14,32	0,40	0,40	0,90		26	26	26
P42	-	27,68	27,67	0,60	0,60	0,77		57	57	57
P46	-	10,30	10,30	0,40	0,60	0,43		20	28	20
P68	-	12,56	12,56	0,30	0,30	1,40		19	19	19
P53	-	12,56	12,56	0,40	0,30	1,05		24	19	24
Lift wall 1	41,70	-	41,70	0,20	1,95	1,07	434782	47	247	47
Lift wall 2	34,93	-	34,93	1,75	0,20	1,00		201	41	201
Lift wall 3	184,86	-	184,86	0,20	5,25	1,76		139	1511	139
Lift wall 4	69,98	-	69,98	2,15	0,20	1,63		388	68	388
$\sum V_{F,\,total}$								1921	2841	1921

Table 6 - $V_{F,i}$- Calculation of the flexural resistance of each vertical element i existing on floor 2.

Vertical elements	**Floor 1**			**Floor 2**			**Floor 3**			**Roof**		
	$V_{F,i}$ (kN)	$V_{c,i}$ (kN)	$V_{H,i}$ (kN)	$V_{F,i}$ (kN)	$V_{c,i}$ (kN)	$V_{H,i}$ (kN)	$V_{F,i}$ (kN)	$V_{c,i}$ (kN)	$V_{H,i}$ (kN)	$V_{F,i}$ (kN)	$V_{c,i}$ (kN)	$V_{H,i}$ (kN)
PB' 6	74	135	74	117	209	117	67	132	67	67	132	67
PB'7	74	135	74	117	209	117	67	132	67	67	132	67
PB'5	44	132	44	74	132	74	74	132	74	51	68	51
PB'8	62	135	62	139	209	139	78	132	78	78	132	78
P31	267	846	267	94	338	94	94	359	94	40	347	40
P37	98	256	98	83	256	83	98	256	98	75	256	75
P47	38	256	38	83	256	83	77	256	77	32	256	32
P33	246	778	246	65	297	65	65	200	65	40	347	40
P29	139	360	139	92	316	92	94	297	94	87	279	87
P38	38	61	38	34	61	34	34	61	34	32	147	32
P39	46	256	46	35	256	35	35	256	35	32	256	32
P40	58	67	58	58	67	58	58	61	58	24	61	24
P41	47	67	47	26	67	26	26	61	26	24	61	24
P42	64	270	64	57	270	57	57	256	57	32	256	32
P46	61	360	61	20	200	20	20	264	20	15	252	15
P68	28	34	28	19	34	19	19	44	19	12	44	12
P53	28	36	28	24	45	24	24	59	24	18	45	18
Existing wall	262	1883	262	-	-	-	-	-	-	-	-	-
Lift wall 1	2097	3591	2097	775	2704	775	775	2704	775	569	2350	413
Lift wall 2	67	174	67	201	758	201	201	758	201	156	121	121
Lifet wall 3	1 534	2308	1 534	139	759	139	139	759	139	79	1 513	79
Lift wall 4	100	262	100	388	935	388	388	935	388	285	164	164
$\sum V_{H\,total}$			3783			1921			1771			1147

Table 7 - $V_{H,\,i}$ - Calculation of the most unfavourable resistance for each vertical element i existing on each floor.

3.2 Obtaining the total weight of the studied building

The total weight of the building was determined considering the masses related to the gravitational forces on each floor, obtaining the values presented in Table 8.

Floor	$G_{K,j}$ (kN/m^2)	$\Psi_{E,i}$	$Q_{k,i}$ (kN/m^2)	Area of the floor, j - $A_{p,j}$ (m^2)	$w_{E,j}$ (kN/m^2)	W_E (kN)
Floor 1	9,45	0,24	2	291,483	9,93	2894,43
Floor 2	8,70	0,24	2	291,483	9,18	2675,81
Floor 3	9,45	0,24	2	291,483	9,93	2894,43
Roof	9,25	0,30	2	300,483	9,85	2959,76

Table 8-W_E - Total weight per Floor.

3.3 Evaluation of the seismic vulnerability

In this phase, it was possible to obtain the seismic capacity coefficients that describe the resistance of each floor of the building, which are presented in Table 9.

Floor	$V_{H,j}$ (kN)	W_E (kN)	$CS_{c,j}$
Floor 1	3782,58	2894,43	1,31
Floor 2	1920,63	2675,81	0,72
Floor 3	1771,24	2894,43	0,61
Roof	1146,66	2959,76	0,39

Table 9 - $CS_{c,j}$- Seismic capacity coefficient for each floor of the building

It is now necessary to know the seismic coefficient required at each floor. In a first phase, the global seismic coefficient is obtained for the building composed of 4 floors founded on type C ground, for the seismic actions of types 1 and 2, from the data obtained by the LNEC Report 81/2019 [1] (and resulting from reliability analyses), that assumes the values of 0.19 and 0.07, respectively.

In this sense, the seismic action type 1 is the most demanding, which is why the comparison of the seismic coefficients (capacity and required) will be analyzed only for the seismic action type 1. Thus, it is necessary to proceed to the definition of the demand for each floor, based on the coefficient ηj, as shown in Table 10.

Floor	CS_E Type 1	CS_E Type 2	η_j	$CS_{E,j}$ Type 1	Verification $CS_{c,j}$	≥	$CS_{E,j}$	
Floor 1	0,19	0,07	1	0,19	1,31	≥	0,19	**OK**
Floor 2			0,9	0,17	0,72	≥	0,17	**OK**
Floor 3			0,7	0,13	0,61	≥	0,13	**OK**
Roof			0,4	0,08	0,39	≥	0,08	**OK**

Table 10-$CS_{E,j}$ – Seismic coefficient required on each floor, and verification of methodology II.

Thus, comparing the two coefficients obtained, it is possible to verify that the requirement or demand in each floor of the building does not override its capacity.

However, it is necessary to verify the same for methodology I, since it is characterized by being more conservative than the previous one, and for taking into account the values that a certain building should present in terms of shear capacity.

4 SEISMIC SAFETY ASSESSMENT ACCORDING TO METHOD I

In this method, the percentage of column area required in relation to the area of floor j ($A_{PE,j}$), will be compared with the percentage of column area existing on same floor ($A_{PC,j}$).

In this way, the building under study does not present seismic vulnerability, according to method I, if each floor obeys the following expression:

$$A_{PC,j} \geq A_{PE,j} \tag{5}$$

To determine the percentage of $A_{PC,j}$, it was necessary to calculate the ratio of the area of the vertical elements of each floor, $\sum A_{pi}$, by the area of the respective floor, $A_{p,j}$, which are represented in Table 11.

Floor	$\sum A_{pi}$ (m^2)	A_{pj} (m^2)	$A_{PC,j}$ (%)
Floor 1	14,81	291,483	5,082
Floor 2	6,32	291,483	2,167
Floor 3	6,57	291,483	2,254
Roof	6,75	300,483	2,245

Table 11 -$A_{PC,j}$- Percentage of the area of all the columns on floor j in relation to the area of the respective floor.

Subsequently, from Report 81/2019 of LNEC (2019) [1] it was possible to determine the required column area, expressed as a percentage of the floor area. However, this area refers to floor 1 of the building to be analysed. To obtain the required column areas in relation to the area of the respective floor it was necessary to apply the coefficient, η_j, also used in method II, obtaining the values presented in Table 12 below.

Floor	$A_{PE,1}$	η_j	**100% x $A_{PE,j}$** (%)	**Verification** $A_{PC,j}$	$\geq$	$A_{PE,j}$	
Floor 1	2,5	1	2,50	5,082	≥	2,50	**OK**
Floor 2		0,9	2,25	2,167	≤	2,25	**KO**
Floor 3		0,7	1,75	2,254	≥	1,75	**OK**
Roof		0,4	1,00	2,245	≥	1,00	**OK**

Table 12 - $A_{PE,j}$- Percentage of column area required on the floor in relation to the area of the respective floor, and verification of expression (5).

Through the results obtained, it is possible to verify safety, analyzing on each floor, if the percentage of the area of the existing elements, satisfies the percentage of area of elements required. This comparison is demonstrated in the last column of Table 12, where it is possible to conclude that in method I, the requirements for 100% of the seismic action on floor 2 are not verified.

However it is mentioned in Ordinance nº 302/2019 [5], in its paragraph 3 of article 1, that in case the seismic vulnerability report does not satisfy the safety requirements related to 90% of the seismic action, defined by the masses associated with the gravitational force of the building, it is mandatory to elaborate the seismic reinforcement project. Therefore, in this sense, it will be necessary to apply methodology I for 90% of the seismic action. Thus, from expression (6) it was possible to determine the required column area for 90% of the seismic action:

$$A_{PE} = \frac{C_{SE}\, n\, w_E}{\tau_{Ml}} \tag{6}$$

Where: n – number of floors existing in the building; τ_{Ml} - equivalent transverse stress (MPa); w_E - weight per unit area resulting from the seismic combination of actions (kN/m^2).

In turn, the equivalent transverse stress was calculated by equation (7), based on the values of resistance to horizontal forces of the vertical elements, obtained by method II and the parameter A_{pi}, corresponding to the area of each column i existing on floor 2:

$$\tau_{MI} = \frac{min(V_{F,i}; V_{C,i})}{A_{pi}} \qquad (7)$$

Thus applying equation (7) it was possible to obtain the equivalent stress (334.79 kPa) represented in the last column of Table 13, as the arithmetic mean of the stress obtained for each column i (of this floor 2).

Vertical elements	$V_{F,i}$ (kPa)	$V_{C,i}$ (kPa)	$min(V_{F,i}V_{C,i})$ (kPa)	L_x (m)	L_y (m)	$A_{p,i}$ (m^2)	$\tau_{MI,i}$ (kPa)
PB'6	117	209	117	0,75	0,25	0,19	628
PB'7	117	209	117	0,75	0,25	0,19	628
PB'5	74	132	74	0,5	0,25	0,13	599
PB'8	139	209	139	0,75	0,25	0,19	744
P31	94	338	94	0,6	0,60	0,36	262
P37	83	256	83	0,6	0,60	0,36	231
P47	83	256	83	0,6	0,60	0,36	231
P33	65	297	65	0,6	0,60	0,36	182
P29	92	316	92	0,4	0,80	0,32	287
P38	34	61	34	0,4	0,40	0,16	216
P39	35	256	35	0,6	0,60	0,36	99
P40	58	67	58	0,4	0,40	0,16	363
P41	26	67	26	0,4	0,40	0,16	165
P42	57	270	57	0,6	0,60	0,36	160
P46	20	200	20	0,4	0,60	0,24	86
P68	19	34	19	0,3	0,30	0,09	217
P53	24	45	24	0,4	0,30	0,12	200
Lift wall 1	46	252	46	0,2	1,95	0,39	120
Lift wall 2	200	758	200	1,75	0,20	0,35	573
Lift wall 3	117	209	117	0,75	0,25	0,19	132
Lift wall 4	117	209	117	0,75	0,25	0,19	901
						$\sum A_{p,i}$=6,32	τ_{MI}= 334,79

Table 13 - τ_{MI} - Determination of equivalent transverse stress - Floor 2.

In this sense, considering 90% of the masses related to the gravitational forces existing on each floor, it was possible to obtain the values presented in Table 14.

Floor	**90% x$A_{PE,j}$**	**$A_{PC,j}$**	**Verification** $A_{PC,j} \geq 90\% x A_{PE,j}$
Floor 1	2,380	5,082	**OK**
Floor 2	1,687	2,167	**OK**
Floor 3	1,502	2,254	**OK**
Roof	1,157	2,245	**OK**

Table 14 - Verification of seismic safety by method I on each floor.

It can be observed, that seismic safety is verified on each floor by this method I.

5 NONLINEAR PUSHOVER ANALYSIS

5.1 The N2 method in SeismoStruct software

The N2 method proposed by Fajfar [6] and universally divulgued by Fajfar *et al* [7-9], used at FEUP by Barros *et al* [10-15], was implemented in successive versions of the SeismoStruct software [16] which allows using uniform static Pushover analysis and adaptive static Pushover analysis. For each analysis seismic actions of types 1 and 2 were considered, as well as in the X and Y directions, applying the accidental eccentricity at the centre of mass of the floor, for the positive and negative direction of each seismic action. However, only the results obtained for seismic action type 1, according to direction X, will be demonstrated, because it is the most unfavorable case.

In this sense, after the calculation of the program, the following deformation was obtained (Fig. 2), as well as the capacity curve and idealized curve necessary for the application of the N2 (Fig.3).

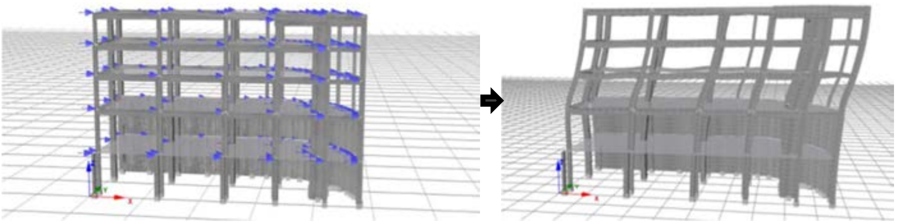

Fig. 2 - Deflection referring to seismic action type 1, according to X direction.

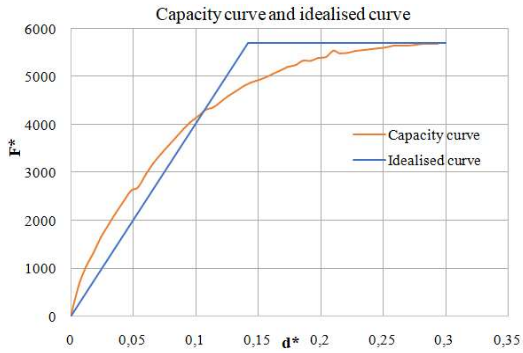

Fig. 3- Capacity curve and idealised curve, according to the X direction.

5.3 Evaluation of the resistant capacity of the building

In order to evaluate the resistant capacity of the structure, the seismic action was applied in the X direction with a target displacement of 49.19 mm; and it was verified that in none of the analysed elements the required requirement exceeded its capacity. However, the shear capacity was exceeded in columns P41, P40 and P53, after the structure assumed a displacement about 50% higher than the target displacement. In P41, the formation of two plastic hinges was observed in the two ends between the 2nd and 3rd floors of the building; Regarding P40, as well as P53, a plastic hinge occurred in one of its ends at floor 2, (assuming the same definition of floor used in the expedite methods). It should be noted that the columns referred, represented in brown in Fig. 4, are pillars existing before the remodelling and were not subjected to structural strengthening.

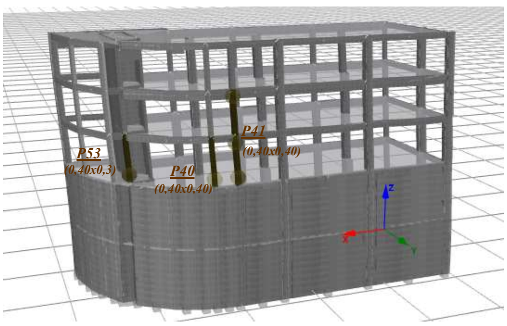

Fig. 4 - Formation of plastic hinges at the ends of columns P41-P40-P53 for a displacement of 78 mm.

CONCLUSIONS

From the exposed analyses, it is concluded that the seismic safety of the building considered by the methods I and II, as well as by the reference method, is verified, not being necessary its seismic reinforcement according to current legislations and ordinances translating the current level of technical knowledge.

Besides, the building has a good performance to the seismic action. In this sense, the results obtained from the adaptive Pushover analysis allowed for the conclusion that the building had a good flexural deformation capacity, since the capacities associated to the shear of the elements were reached first than the ultimate rotation of the chord (associated to flexural capacities); since the post-elastic resistance of the vertical elements existing in the building was considered. Moreover, the cyclic shear strength decreases according to the ductility requirements of the plastic part, according to NP EN 1998-3 (2017) [3], so it is natural, its increase in the expedite method II. However, it is important to mention that through the results obtained in the adaptive nonlinear Pushover analysis, it was possible to verify an adequate cyclic shear capacity in the vertical elements of the building, since only the formation of the first hinge in the structure occurred, after exceeding the target displacement of the real structure of n degrees of freedom.

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