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COMPARISON BETWEEN THE BRITISH AND BRAZILIAN STANDARD APPROACHES IN THE DESIGN OF MASONRY WALLS AND COLUMNS CONSIDERING LOCAL SECOND ORDER EFFECTS

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ABSTRACT

Slender walls and columns have lower capacity to carry loads due to lateral deflections. In order to cover slenderness effects in masonry structures, an additional eccentricity is added to the vertical load eccentricity as presented in BS 5628-1:2005. Capacity reduction factor (CRF) is used to determine the compressive strength of masonry walls and columns in terms of slenderness ratio and eccentricity of load. It can be derived by assuming that the stress is uniformly distributed along the compressive zone. However, the equations to calculate the CRF given in the British Standard (BS 5628-1) for single wythe walls only deals with solid section walls. Then, the CRF of four different hollow cross sections were determined. The local second order effects were considered in the design of the masonry walls following the recommendations given in the British (BS 5628-1:2005) and Brazilian (NBR 15961-1:2011) standards. It was noticed that the required strength increased significantly, meaning that it is important to consider these effects in the design. Large masonry shear walls of the ground floor of an actual multi-story building were divided in equal parts and analyzed as individual columns, similarly to the Brazilian Concrete Code (NBR 6118:2014) procedure. The geometry and loading data of the shear walls have been taken from the structural design of the building, which was calculated using the Brazilian software TQS for masonry building design. No modelling was performed, only simplified analysis on mid span cross sections of the wall associated to the largest transverse displacement. It was evaluated the increase in the required compressive strength of the walls when the second order bending moments due to slenderness are taken in account.

KEYWORDS: *additional eccentricity, capacity reduction factors, slender masonry walls, second order effects*

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INTRODUCTION

The use of structural masonry system in buildings apartments has been widely increased in Brazil. Also, as the construction techniques and design procedures are gradually improving, buildings up to 20 floors have become usual. However, it leads to walls under greater axial loading and also to slender structural elements, in which additional loadings may arise. On masonry buildings of several storeys, the panels are under compression and bending. Normal stresses come from the vertical loading having a uniform distribution and also from the bending moment caused by the lateral loads and eccentricity of the vertical load.

The Brazilian and British standards approach differently the local second order effects. In the Brazilian standard [2], the local second order effects are considered by applying a bending moment about the minor axis, as shown in figure 3 and it is calculated using equation 11. In the British standard [3], it is considered by an additional eccentricity presented in annex B of the referred standard.

According to [1], it is expected that using different international standards, the capacity strength of masonry walls would be reasonably distinct, due to the different design recommendations of each code.

Currently, the local second order effects in the design of masonry shear walls are not considered. For this reason, the scope of this paper is to compare the required compressive characteristic strength of shear walls with different geometry using the approaches of both the Brazilian and British Standards, considering the local second order effects in the design.

BRITISH STANDARD (BS 5628:2005)

The British standard for the design of masonry walls deals only with single wythe walls and columns of solid section. According to [4], the method used on the design of unreinforced walls under vertical loading, is based on a plastic compressive zone with uniform tension distribution. Moreover, it is assumed that the wall is not capable of bearing tensile stresses after cracking occurs. Once unreinforced masonry was assumed to the analysis, the tensile stresses were neglected on the CRF calculation. These assumptions lead to the stress distribution illustrated by Figure 1, in which the tensile stresses are low and can be neglected.

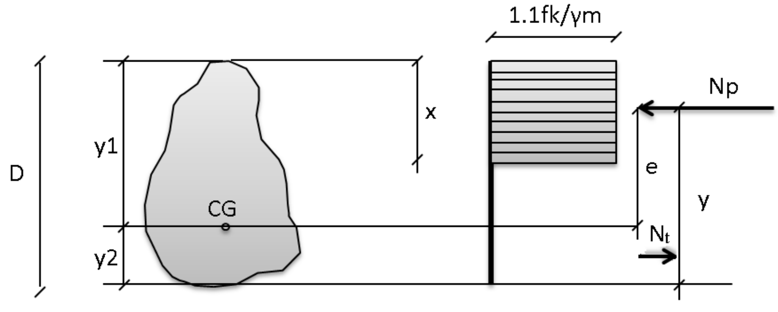

Figure 1: Compressive stress distribution in the cross section. Adapted from [4]

When the vertical load is not applied to the centroid, the neutral axis must be determined and therefore the compressive block area (shown in Figure 01) can be calculated. The equations to determine the neutral axis are shown on the next section.

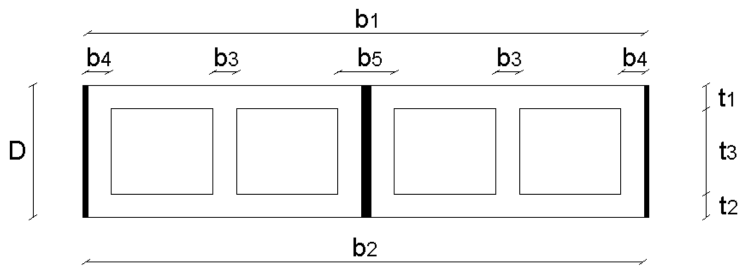

Figure 2: Cross section 01

Neutral axis equations

Depending on the position where the load is applied, the neutral axis will be either on the flange or the web of the section. The equations to determine the neutral axis of the cross section 01 were derived and are shown below.

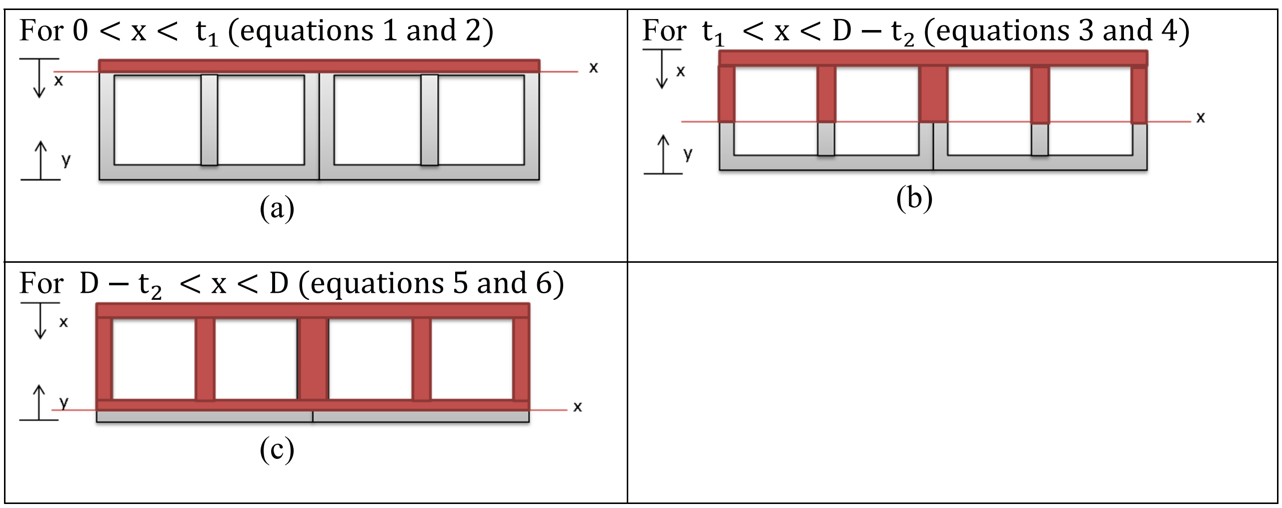

Figure 3: Neutral axis: (a) Top flange; (b) Web; (c) Bottom flange

$$A_c = b_1 * x \tag{1}$$

$$y = D - \frac{x}{2} \tag{2}$$

$$A_c = b_1 * t_1 + (2b_3 + 2b_4 + b_5) * (x - t_1) \tag{3}$$

$$y = \frac{b_1 * t_1 * \left(D - \frac{t_1}{2}\right) + [(2b_3 + 2b_4 + b_5) * (x - t_1)] * (D - \frac{t_1}{2} - \frac{x}{2})}{A_c} \tag{4}$$

$$A_c = b_1 * t_1 + (2b_3 + 2b_4 + b_5) * (D - t_1 - t_2) + b_2 * (t_2 - D + x) \tag{5}$$

$$y = \frac{(b_1 t_1)\left(D - \frac{t_1}{2}\right) + [(2b_3 + 2b_4 + b_5)(D - t_1 - t_2)] * \left(\frac{D}{2}\right) + [b_2(t_2 - D + x)] * (D - x + \frac{t_2}{2})}{A_c} \tag{6}$$

Capacity reduction factor

Once the neutral axis was determined, it is possible to calculate the compressive plastic block (A_c) and also the capacity reduction factor (β), which is given by:

$$\beta = \frac{1{,}1.A_c}{A_t} \tag{7}$$

where: A_c is the compressive plastic block shown in Figure 1 and A_t is the area of the cross section. According to [4], "β" is increased by 10% due to the eccentricity of the load. The capacity reduction factors are presented in tables, in terms of slenderness ratio and eccentricity of load as shown in table 07 of the British Standard [3] for solid section of single wythe walls. Capacity reduction factor of a slender masonry wall of hollow cross section blocks, as shown in figure 2, were derived and are presented in the Results section on tables 2 and 3.

Design resistance of columns

The design resistance of columns is calculated as follows:

$$N_d = \frac{\beta.A.f_k}{\Upsilon_m} \tag{8}$$

where: β is the capacity reduction factor; A is the net area of the column's cross section; f_k is the characteristic strength of the wall and Υ_m is the partial safety factor for material

Local second order analysis

Slender walls and columns have lower capacity support, due to the lateral deflection. In the British Standard [3], local second order is considered by an additional eccentricity (e_a), given by:

$$e_a = D.(\frac{1}{2400}\lambda^2 - 0.015) \tag{9}$$

where: D is the depth of the section and λ is the slenderness ratio of the wall or column

The British standard permits to reduce the effective length when lateral supports provide resistance to lateral movement. However, on the calculations the effective length considered was the height of the wall itself, this is 280 centimeters.

BRAZILIAN STANDARD (NBR 15961:2011)

Design Resistance of columns

According to section 11.5.2 Brazilian standard [2], unreinforced walls under vertical and wind load are calculated by the superposition of equivalent normal stresses. Hence, the following equation must be satisfied:

$$\frac{N_d}{A.R} + \frac{M_d}{W.K} \leq \frac{f_k}{\Upsilon_m} \tag{10}$$

where: N_d, M_d is the design axial resistance and design bending moment, respectively;

A is the gross area of the column's cross section; $R = 1 - \left(\frac{\lambda}{40}\right)^3$ is the reduction coefficient due to slenderness of the wall; W is the elastic section modulus; K is the factor to adjust the compression resistance in bending and f_k, Υ_m were previously defined.

Local second order analysis

According to section 11.5.3.2 of the Brazilian Standard [2], local second order must be taken in account in walls under vertical loading with slenderness ratio greater than 12. It is considered by applying a bending moment about the minor axis as illustrated in figure 4.

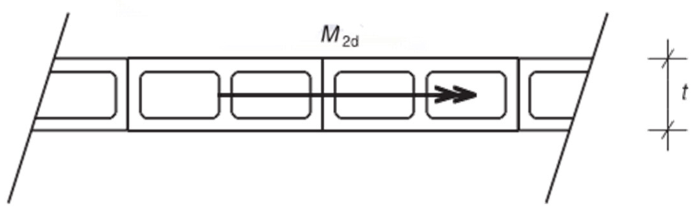

Figure 4: Second order bending moment out of plane

The second order bending moment is calculated as follows:

$$M_{2d} = \frac{N_d . l_e^2}{2000. t} \tag{11}$$

Where: N_d is the design axial loading; l_e is the length of the wall and t is the thickness. Equation 11 is very similar to the one presented in Eurocode 6, which prescribes its use for reinforced masonry walls with slenderness ratio greater than 12, adopting the principles and application rules for unreinforced masonry.

METHODS

A case study of a 12 story building is presented, in which four masonry shear walls of the ground floor were chosen to be evaluated. The results obtained using the design procedures of each standard were compared and the local second order effects were considered. In order to perform the calculations, the shear walls were divided in equal parts and analyzed as smaller individual columns, similarly to the method presented in [5] for the local analysis of large concrete columns (where the ratio of the cross section dimensions' is greater than 5) illustrated in Figure 5. n_d is the estimated normal stress distribution due to the vertical and wind load (bending moment about the major axis) and m_d is the second order bending moment about the minor axis.

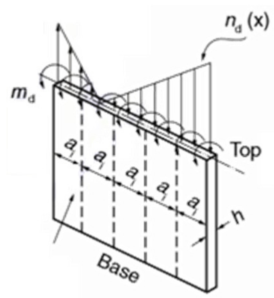

Figure 5: Shear wall division and approximate normal stress distribution (NBR 6118:2014)

The effective length of the columns resulting of the division of the shear wall was also calculated using [5]. When the columns are laterally restrained by a transversal wall, it can be considered a vertical support along the corresponding vertical edge of the column (right-hand side), otherwise it is considered free on both edges (left-hand side), as illustrated in Figure 6.

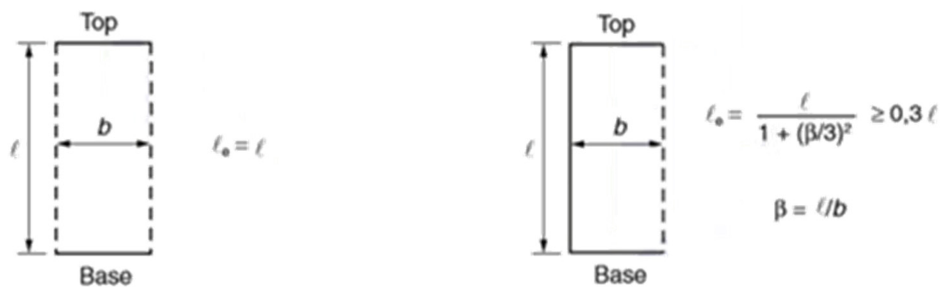

Figure 6: Effective length of the columns (NBR 6118:2014)

The building floor plans as well as the masonry shear walls selected for the analysis are illustrated by Figure 7.

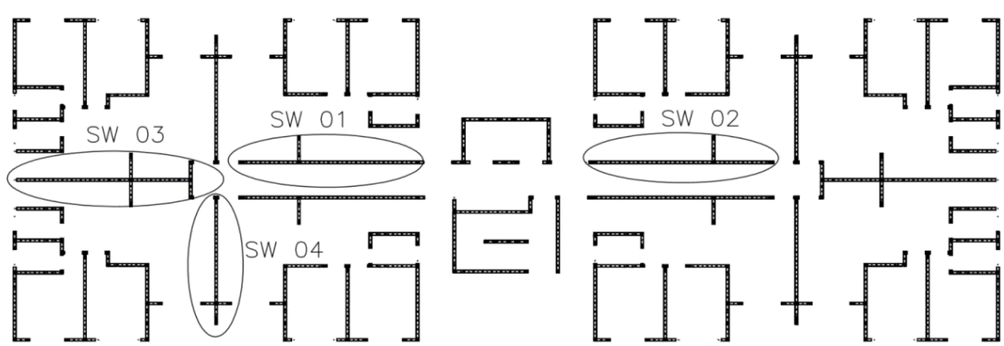

Figure 7: Floor plan of the building and shear walls evaluated

Figures 8(a) to 8(d) show all four column's cross sections (F1 to F4) selected to perform the calculations. F1 to F4 are the columns resulting from the partition of the masonry shear walls utilizing the adapted method given in [5], whose geometry and dimensions are shown in the Figures 8(a) to 8(d) and Table 1.

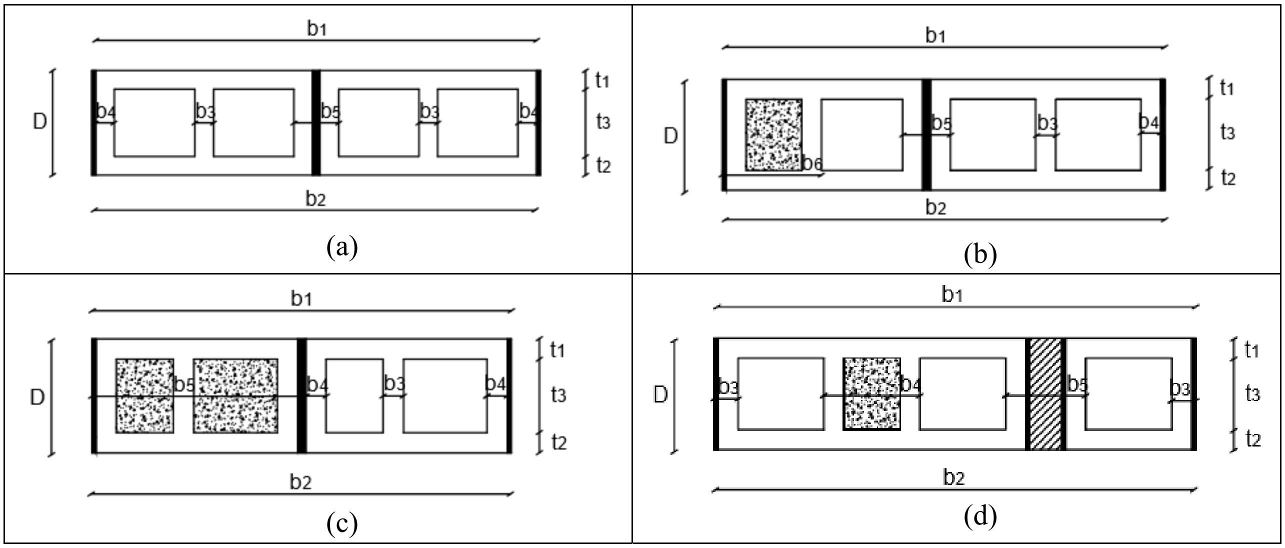

Figure 8: Column's cross sections: (a) Type 01; (b) Type 02; (c) Type 03 and (d) Type 04.

Table 1: Dimensions of the columns' cross sections in centimeters.

Column's Cross sections	Dimensions (centimetres)										
	b1	b2	b3	b4	b5	b6	t1	t2	t3	D	Net Area	Gross Area
Cross section 01	80.0	80.0	2.5	3.0	6.0	-	2.5	2.5	9.0	14.0	553.0	1120.0
Cross section 02	75.0	75.0	2.5	3.0	6.0	17.5	2.5	2.5	9.0	14.0	636.0	1272.0
Cross section 03	70.0	70.0	2.5	3.0	35.0	-	2.5	2.5	9.0	14.0	741.5	1483.0
Cross section 04	80.0	80.0	3.0	17.5	11.0	-	2.5	2.5	9.0	14.0	710.5	1421.0

Shear wall 04 is illustrated in Figure 9 showing the columns chosen for the analysis.

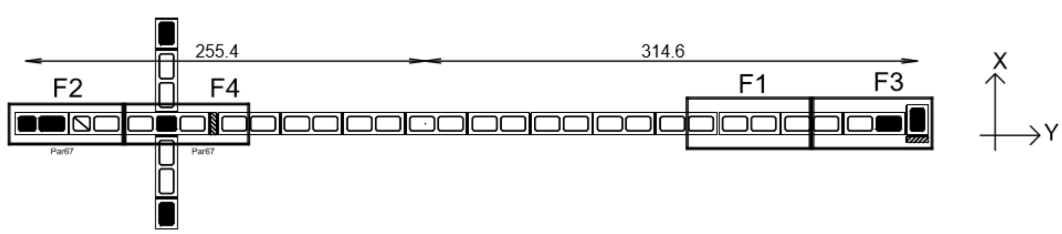

Figure 9: Cross sections F1 to F4 of the Shear Wall 04

RESULTS

Capacity reduction factors in terms of slenderness ratio and eccentricity for all four column's cross sections are shown in tables 2 and 3.

Table 2: Capacity reduction factor of cross sections types 01 and 02 according to BS 5628.

Slenderness Ratio	Capacity Reduction Factor											
	Cross Section 01						Cross Section 02					
	Eccentricity (e/D)						Eccentricity (e/D)					
	0.05	0.10	0.15	0.20	0.25	0.30	0.05	0.10	0.15	0.20	0.25	0.30
0	1.0	0.90	0.82	0.75	0.68	0.61	1.0	0.90	0.81	0.73	0.65	0.56
5	1.0	0.90	0.82	0.75	0.68	0.61	1.0	0.90	0.81	0.73	0.65	0.56
6	1.0	0.90	0.82	0.75	0.68	0.61	1.0	0.90	0.81	0.73	0.65	0.56
8	1.0	0.90	0.82	0.75	0.68	0.61	1.0	0.90	0.81	0.73	0.65	0.56
10	0.98	0.90	0.82	0.75	0.68	0.61	0.98	0.90	0.81	0.73	0.65	0.56
12	0.95	0.89	0.82	0.75	0.68	0.61	0.95	0.89	0.81	0.73	0.65	0.56
14	0.91	0.86	0.81	0.75	0.68	0.61	0.90	0.85	0.80	0.73	0.65	0.56
16	0.86	0.82	0.77	0.73	0.68	0.61	0.86	0.81	0.76	0.71	0.65	0.56
18	0.82	0.77	0.73	0.69	0.65	0.61	0.81	0.76	0.71	0.67	0.62	0.56
20	0.77	0.73	0.69	0.65	0.61	0.57	0.76	0.71	0.66	0.61	0.56	0.51
22	0.72	0.68	0.64	0.60	0.56	0.51	0.70	0.66	0.60	0.55	0.50	0.44
24	0.67	0.63	0.59	0.54	0.49	0.42	0.64	0.59	0.54	0.48	0.42	0.34
26	0.62	0.57	0.52	0.47	0.37	0.24	0.57	0.52	0.46	0.39	0.30	0.19
27	0.59	0.54	0.48	0.41	0.27	0.14	0.53	0.47	0.41	0.33	0.22	0.11

Table 3: Capacity reduction factors of cross sections types 03 and 04 according to BS 5628.

Slenderness Ratio	Capacity Reduction Factor											
	Cross Section 03						Cross Section 04					
	Eccentricity (e/D)						Eccentricity (e/D)					
	0.05	0.10	0.15	0.20	0.25	0.30	0.05	0.10	0.15	0.20	0.25	0.30
0	1.0	0.89	0.80	0.70	0.61	0.51	1.0	0.90	0.81	0.73	0.64	0.55
5	1.0	0.89	0.80	0.70	0.61	0.51	1.0	0.90	0.81	0.73	0.64	0.55
6	1.0	0.89	0.80	0.70	0.61	0.51	1.0	0.90	0.81	0.73	0.64	0.55
8	1.0	0.89	0.80	0.70	0.61	0.51	1.0	0.90	0.81	0.73	0.64	0.55
10	0.98	0.89	0.80	0.70	0.61	0.51	0.98	0.90	0.81	0.73	0.64	0.55
12	0.94	0.88	0.80	0.70	0.61	0.51	0.94	0.89	0.81	0.73	0.64	0.55
14	0.90	0.84	0.78	0.70	0.61	0.51	0.90	0.85	0.80	0.73	0.64	0.55
16	0.85	0.79	0.74	0.68	0.61	0.51	0.86	0.81	0.76	0.71	0.64	0.55
18	0.80	0.74	0.68	0.63	0.57	0.51	0.81	0.76	0.71	0.66	0.61	0.55
20	0.74	0.68	0.62	0.56	0.50	0.44	0.76	0.71	0.66	0.60	0.55	0.49
22	0.67	0.61	0.55	0.49	0.43	0.37	0.70	0.65	0.59	0.54	0.48	0.42
24	0.60	0.54	0.48	0.42	0.35	0.28	0.63	0.58	0.53	0.47	0.40	0.33
26	0.51	0.45	0.39	0.32	0.24	0.16	0.56	0.50	0.44	0.38	0.29	0.18
27	0.47	0.41	0.34	0.27	0.18	0.09	0.52	0.46	0.40	0.32	0.21	0.11

Values shown in table 2 for the cross section 01 are presented below in graphical form. The graphs associated to the other cross sections presented similar curves. The results show that for low eccentricity ratios (up to 0.10D), the second order effects are more significant for slenderness ratio greater than 12 (lower limit is specified by the Brazilian standard to consider the second order effects).

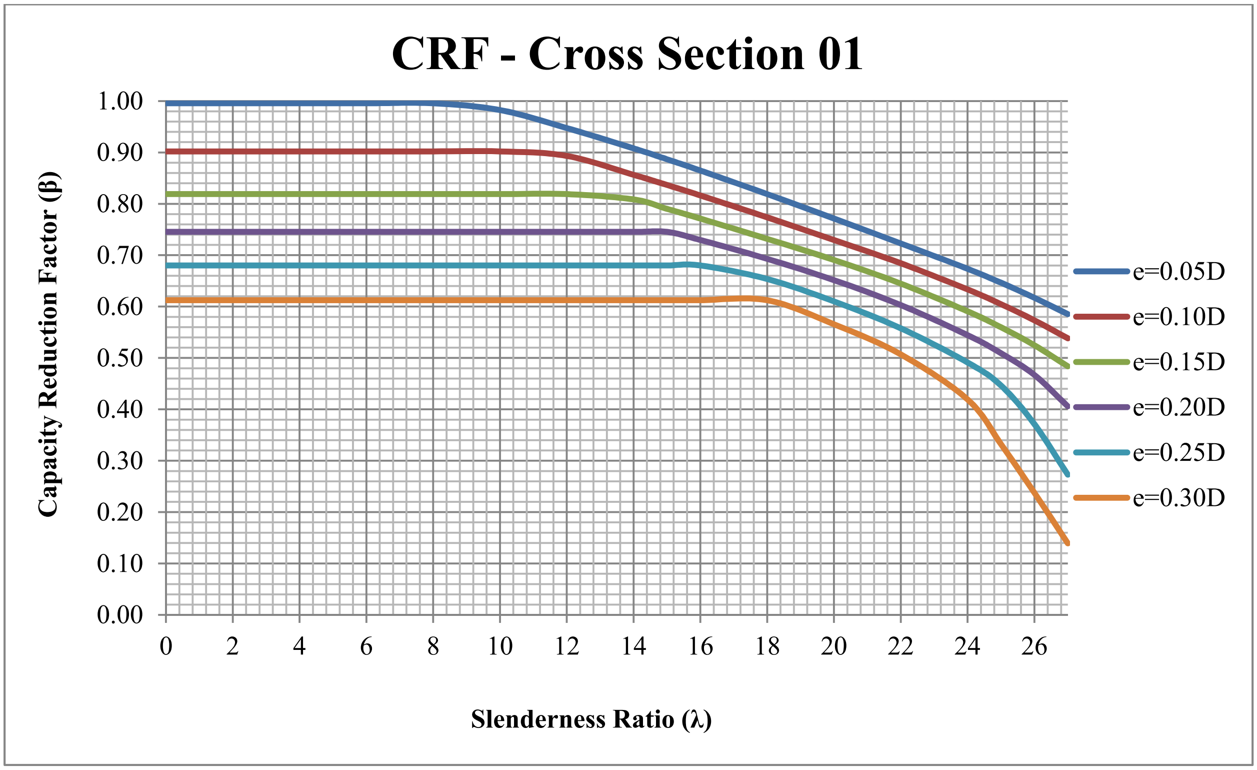

Figure 10: Capacity reduction factors for cross section 01.

Table 4 shows the results for the required characteristic compressive strength. The Brazilian standard considers the gross area of the cross section on the calculations, while the British standard the net area. For this reason, in order to effectively compare the required characteristic strength of the masonry shear walls (f_k) obtained, both were calculated considering the net area of the section. The British Standard's results were found using the equations 7 to 9, whilst the results by means of the Brazilian Standard were obtained from equations 10 and 11, by separating f_k.

Table 4: Required Characteristic Compressive Strength of individual columns.

Shear Wall	Column	Required Characteristic strength (MPa)			Difference (Net Area)
		Brazilian Standard		British Standard	
		Gross Area	Net Area	Net Area	(%)
SW 01	F1	8.89	14.83	14.79	0.30
	F2	8.63	14.30	14.09	1.49
	F3	9.26	15.88	16.44	3.53
	F4	5.17	9.87	10.79	9.32
SW 02	F1	8.74	14.58	14.58	0.00
	F2	8.48	14.05	13.89	1.19
	F3	9.14	15.67	16.29	3.97
	F4	5.09	9.69	10.59	9.32
SW 03	F1	8.74	14.53	14.65	0.83
	F2	8.02	13.07	12.73	2.69
	F3	10.11	17.55	19.03	8.46
	F4	4.85	9.16	10.01	9.32
SW 04	F1	10.61	17.14	17.60	2.66
	F2	7.95	15.46	15.78	2.07
	F3	11.67	19.99	21.87	9.41
	F4	6.86	12.20	13.33	9.32

It is noticeable that the percentage difference of column F4 of all panels presented the same value (9.32%). Since the column F4 is restrained by a small transversal wall, a vertical support was considered. For this reason, the effective length was reduced and thus the slenderness ratio, resulting in a capacity reduction factor of 1.0 (table 3, cross section 4, slenderness ratio = 6.4 and e/D = 0.05). Therefore, the required characteristic compressive strength of the columns was only in terms of the constants, which explains this result. The results also show the increase in the required characteristic compressive strength when the second order effects are taken into account by means of the Brazilian Standard [2], which emphasizes the importance of these effects in the design of shear walls.

Columns F4 of all four shear walls presented lower values of required characteristic compressive strength when compared to the other columns, F1 to F3. This is due to the vertical support provided by the flange, according to the method of local second order analysis given in [5], illustrated in figures 4 and 5. It resulted in a reduction of effective length, and consequently, of the slenderness ratio. On the other hand, columns F1 to F3 of all shear walls were not restrained, meaning that there is no flange providing vertical support. As consequence, they presented greater values of required characteristic compressive strength.

CONCLUSIONS

From the analysis evaluated to the particular building presented in this paper, the following conclusions can be made:

The increase in the required characteristic compressive strength of shear walls due to second order effects was very significantly, emphasizing the importance of considering the out-of-plane bending on shear wall's individual columns design described in this study.

Individual columns restrained vertically by transverse walls (flanges), (such as column F4 shown in figure 9), demand the lowest compressive strength due to the reduction of the effective length as a consequence reduction of slenderness ratio (calculated as shown in figure 6), although they have the greatest combined vertical compressive stresses from vertical and wind loads. Moreover, these particular individual columns presented similar capacity reduction factor (CRF) for hollow and half-grouted cross sections. On the other hand, individual columns at the far ends of the shear wall and without vertical restraint presented the greatest values for required characteristic compressive strength.

It is important to be careful when comparing different standards. In particular case of this study was necessary to adjust the results from Brazilian Standard to the net area for an appropriate comparison with British Standard. After that, the results obtained from both standards were very similar, being 9.41% the greatest percentage difference observed for the required compressive strength, which is reasonable. It was also observed that the value 12 prescribed in the Brazilian Standard it seems to be adequate for the lower limit of slenderness ratio to take into account the second order effects for individual columns in shear walls design.

Also, the Brazilian standard presented larger second order values compared to the British standard. For instance, a column having slenderness ratio of 16 presented a second order eccentricity of 2.21 cm by the Brazilian standard, while for the British standard 1.28 cm was obtained.

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