IBRACON Structures and Materials Journal
Revista IBRACON de Estruturas e Materiais
ISSN 1983-4195 ismj.org

ORIGINAL ARTICLE

Influence of shear walls on the structural behavior of a multi-storey concrete building according to Brazilian Technical Code ABNT NBR 15421:2006

Influência de pilares-parede no comportamento estrutural de um edifício de múltiplos pavimentos de concreto de acordo com a norma técnica brasileira ABNT NBR 15421:2006

Carlos Alexandre Seruti[a]
Eduardo Fernandes Lima[a]
Silvio de Souza Lima[a]
Sergio Hampshire de Carvalho Santos[a]

[a]Universidade Federal do Rio de Janeiro – UFRJ, Programa de Projeto de Estruturas da Escola Politécnica, Rio de Janeiro, RJ, Brasil

Received 11 March 2022
Accepted 06 December 2022

Abstract: This paper presents a comparative study of the influence of shear walls on the behavior of a multi-storey concrete building when subjected to seismic actions. Two structures with different earthquake-resistant systems were evaluated. One is composed of concrete frames with usual detailing in both directions (x and y), and the other one is formed by a dual system, composed of frames and shear walls with usual detailing. The Equivalent Lateral Force procedure and the Modal Response Spectrum analysis presented by the Brazilian Technical Code ABNT NBR 15421:2006 were considered. The dual system building presented a greater stiffness with the shear walls arranged parallel to direction y. In the x direction, the responses are equivalent in both buildings. The horizontal forces obtained by the Equivalent Lateral Force method at the base of the buildings were greater than the Modal Response Spectrum method.

Keywords: seismic analysis, equivalent lateral force procedure, modal response spectrum analysis.

Resumo: Este trabalho apresenta um estudo comparativo da influência de pilares-parede no comportamento de um edifício de concreto de vários andares quando submetido a ações sísmicas. Foram avaliadas duas estruturas com diferentes sistemas sísmicos. Uma é composta por pórticos de concreto com detalhamento usual em ambas as direções (x e y), e a outra é formada por um sistema dual, composto por pórticos e pilares-parede com detalhamento usual. Os Métodos das Forças Horizontais Equivalentes e Espectral apresentados pela norma técnica brasileira ABNT NBR 15421:2006 foram considerados. Verificou-se que a edificação de sistema dual apresentou maior rigidez com os pilares-parede dispostos paralelamente à direção y. Na direção x, as respostas são equivalentes em ambos os edifícios. Os esforços horizontais obtidos pelo método das forças equivalentes na base dos edifícios foram maiores que os espectrais.

Palavras-chave: análise sísmica, método das forças horizontais equivalentes, método espectral.

How to cite: C. A. Seruti, E. F. Lima, S. S. Lima, and S. H. C. Santos, "Influence of shear walls on the structural behavior of a multi-storey concrete building according to Brazilian Technical Code ABNT NBR 15421:2006," *Rev. IBRACON Estrut. Mater.*, vol. 16, no. 5, e16504, 2023, https://doi.org/10.1590/S1983-41952023000500004

Corresponding author: Carlos Alexandre Seruti. E-mail: carlosseruti@gmail.com
Financial support: None.
Conflict of interest: Nothing to declare.
Data Availability: the data that support the findings of this study are available from the corresponding author, [CAS], upon reasonable request.

1 INTRODUCTION

Brazil seismic occurrence is low compared to other South American countries, such as Argentina, Chile, Colombia, Ecuador, and Peru. These countries are located on the edge of the seismic plate which gives rise to the Andes, where earthquakes are frequent. Brazil, despite most of its territory is in a region of low seismicity, located internally in a tectonic plate, there are areas that show a potential for the occurrence of earthquakes. The greatest registered accelerations in the country occurred in the Northeast region (Ceará and Rio Grande do Norte) and in west of the North region, in Amazonas and Acre, as shown by Santos et al. [1] and Paiva et al. [2]. Therefore, such activity is not totally null, and even being of low intensity, it should not be disregarded in Brazilian civil engineering projects.

The Brazilian Association of Technical Codes (ABNT) published in 2006 the first version of a technical code in this subject, ABNT NBR 15421 [3]. This code was based on the Seismic Hazard presented by Giardini et al. [4] on the Global Seismic Hazard Assessment Program (GSHAP). However, seismic zones, such as northern Mato Grosso do Sul State, were not properly considered in the GSHAP and ended up not being represented in the technical code map. Thus, there is a need to update the Brazilian seismic map, as mentioned by Miranda et al. [5].

In the international technical literature, it is possible to find numerous references involving seismic studies, see for instance [6]-[15]. In Brazil, however, there are few studies on this subject. This is due to the low occurrence of earthquakes of large magnitudes in the Brazilian territory. Among the Brazilian general studies on earthquakes, some can be cited: Pereira et al. [16] evaluates the seismic reliability assessment of a reinforced concrete (RC) frame designed in accordance with ABNT NBR 6118 [17] without considering the seismic design prescribed by ABNT NBR 15421 [3]; Aguero et al. [18] proposed a structural retrofit method for evaluation of a RC casing in columns with increase in bending moments under seismic loads; Matos [19] and Alva et al. [20] studied the influence of masonry on the strength of buildings subjected to seismic excitations; Gidrão et al. [21] simulated seismic effects in a 12-story reinforced concrete building; Peña and Carvalho [22] studied the influence of the structural configuration on the seismic response of a reinforced concrete structure.

This paper aims to compare the influence of shear walls on the stiffness of the reinforced concrete structure of a building in terms of resulting forces in its base and displacements in the top of the structure. For the calculations, two methods for seismic analysis described in ABNT NBR 15421 [3], were considered: (*i*) Equivalent Lateral Forces and (*ii*) Modal Response Spectrum.

2 CASE STUDY

Two hypothetical building models with distinct earthquakes-resistant basic systems are evaluated. One is composed only by concrete frames with the usual detailing in both directions (x and y), and the other one presents a dual system, composed of frames and concrete shear walls, both with usual detailing. Thus, the difference lies in the presence of shear walls in one of them since the dimensions of the structural elements are the same in both models. The main characteristics of the building can be seen in Table 1.

Table 1. Building characteristics.

Elements and properties	Values
Columns section	(60 x 60) cm^2
Beams section	(30 x 60) cm^2
Slabs thickness	20 cm
Walls thickness	30 cm
Floor height	3.0 m
Concrete compression strength (f_{ck})	30 MPa
Concrete specific weight	25 kN/m^3
Concrete modulus of elasticity	30000 MPa
Poisson coefficient	0.20

It is important to note that in this research it is not being considered whether the structure, from de design point of view, is acceptable. Loads were verified only at the base of the structure.

The studied buildings have 12 storeys and height of 36 m, with dimensions in plan of 12 m x 16 m, as shown in Figure 1. The modeling was performed in the Robot Structural Analysis Professional 2021 computer program [23] and it is depicted in Figure 2. In the dual system, each shear wall was discretized into 12 shell finite elements of dimensions 1 m x 1 m. The properties of these shear walls are the same as the structure and are defined in Table 1.

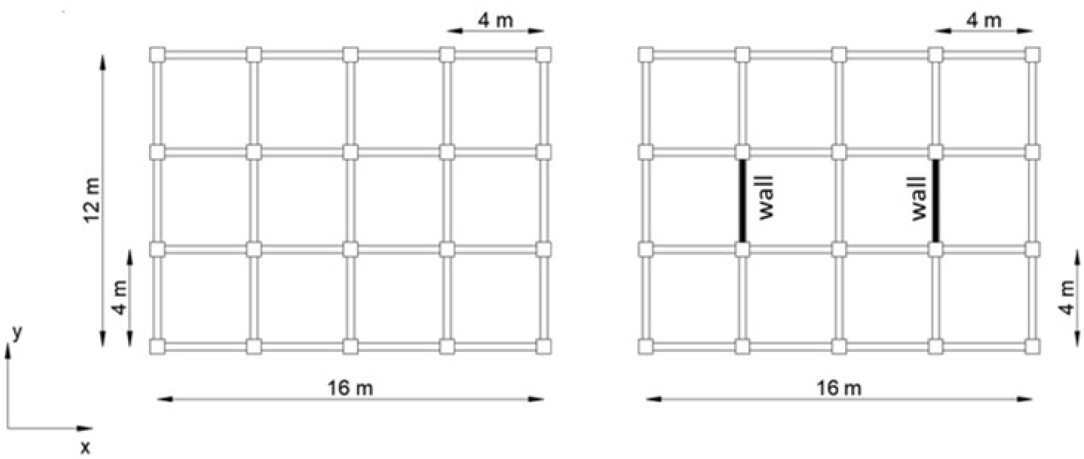


Figure 1. Plan dimensions of the earthquake-resistant basic systems.

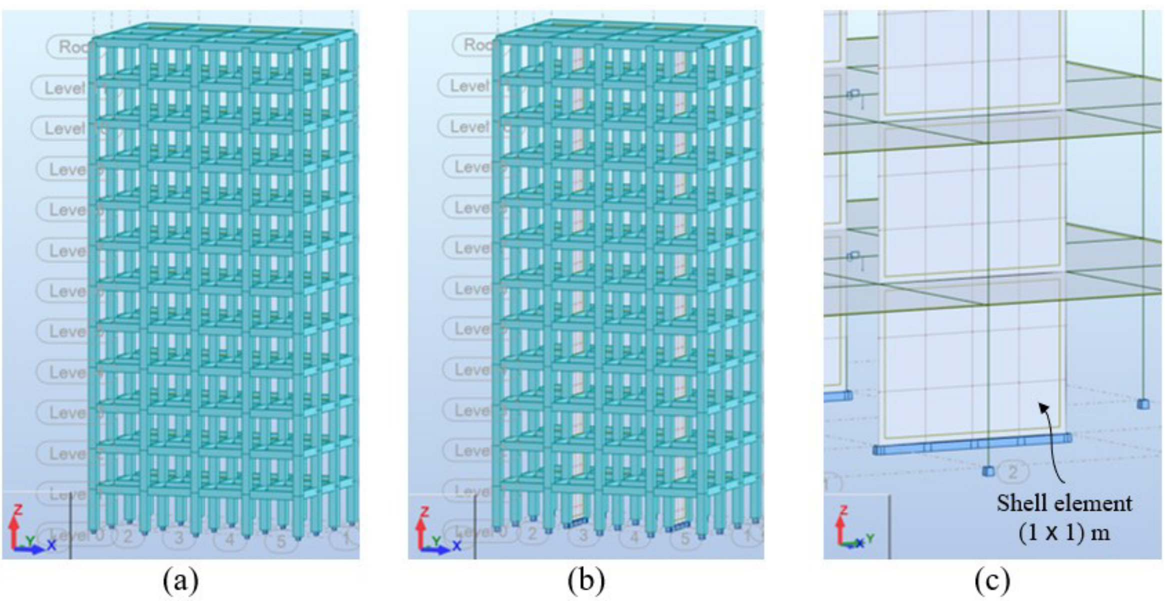


Figure 2. Structural systems: (a) framed with usual detailing in both directions; (b) dual system, composed of frames and shear walls with usual detailing; (c) shear walls in shell finite elements.

It was considered that the location of the model buildings is in the Acre State, in Brazilian Seismic Zone 4, in a site where the basic horizontal seismic acceleration (a_g) is 0.15g, being g the gravity acceleration, as defined by ABNT NBR 15421 [3] and shown in Figure 3. The considered site class is *D*, corresponding to "stiff soil" and the seismic category is *C*. The average properties for the 30 m below the ground top, i.e., the average velocity of propagation of shear waves, $\overline{V_S}$, and the average blows SPT test number, $\overline{N}$, are in the ranges, respectively, 370 m/s $\geq \overline{V_S} \geq 180$ m/s and $50 \geq \overline{N} \geq 15$. The values defined as nominal for seismic actions are those that have a 10% probability of being exceeded during a period of 50 years, which corresponds to a recurrence period of 475 years.

Elastic response spectrum of accelerations can be obtained considering the soil characteristics and the standard code definitions (see Figure 4). The figure expresses the relationship between the structural periods T(s) and horizontal accelerations S_a(m/s^2), corresponding to the elastic response of a one-degree-of-freedom system with a critical damping ratio equal to 0.05.

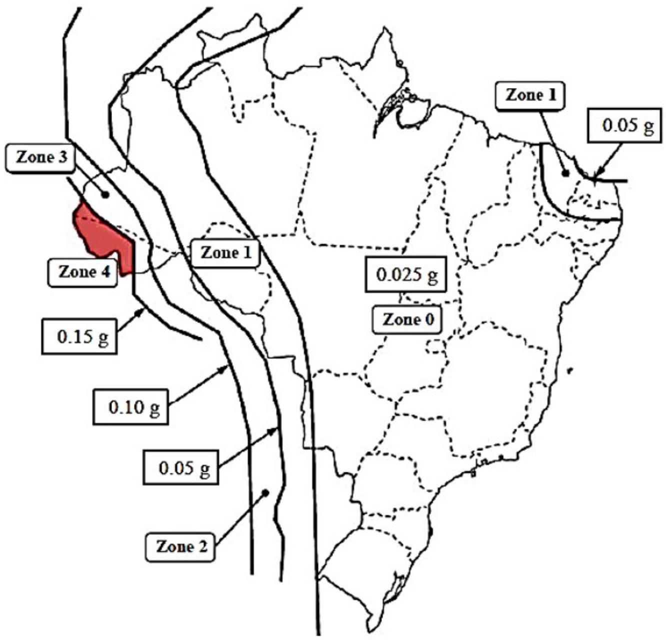


Figure 3. Horizontal nominal seismic accelerations in Brazil and region of location of building [3].

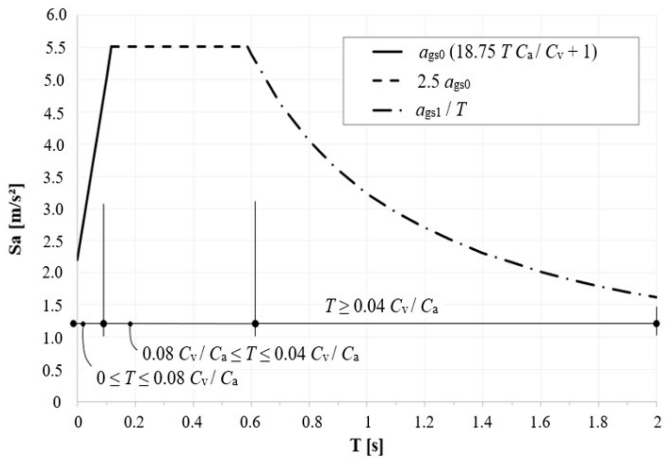


Figure 4. Elastic response spectrum of accelerations.

The spectrum does not depend on the basic earthquake-resistant systems defined in the two resistant models studied, but on the site class (*D*) and the characteristic horizontal seismic acceleration (a_g). The seismic amplification factors, C_a and C_v, are in this case, respectively, 1.5 and 2.2 (Table 3 of ABNT NBR 15421 [3]). The spectral acceleration for the periods of 0 s (a_{gs0}) and 1 s (a_{gs1}) were obtained by multiplying C_a e o C_v by a_g.

3 METHODOLOGY

In this study, analyses considering two NBR 15421 calculation methods are performed. The methods are:

(i) Equivalent Lateral Force;

(ii) Modal Response Spectrum.

Two earthquake-resistant systems were evaluated: (a) Only concrete frames with usual detailing and (b) dual system, composed of frames and concrete shear walls, both with usual detailing. The results are forces and displacements obtained in the two analyses. The absolute displacements δ and relative displacements Δ were compared, verifying the influence of the shear walls on the response of the buildings.

3.1 Equivalent lateral force (ELF) procedure

3.1.1 Total horizontal force

The ELF procedure leads to a static linear analysis. The total horizontal force at the base of the structure H, in each direction, can be determined by Equation 1.

$$H = C_s\, W \tag{1}$$

where C_s = is the seismic response coefficient; W = total weight of the structure, considering only permanent loads.

The seismic response coefficient, C_s, is defined according to Equation 2:

$$C_s = \frac{2.5\, I\ a_{gs0}}{g\, R} \tag{2}$$

where I = use importance factor, adopted in this case equal to 1.0 (see Table 4 of the ABNT NBR 15421 [3]); a_{gs0} = spectral acceleration for the period of 0.0 s; g = gravity acceleration, considered equal to 9.81 m/s^2; R = response modification coefficient.

According to ABNT NBR 15421 [3], the minimum value of the C_s is 0.01, and it need not exceed that given by Equation 3:

$$C_s = \frac{I\ a_{gs1}}{g\, R\, T} \tag{3}$$

where a_{gs1} = spectral acceleration for the period of 1.0 s; T = natural period of the structure.

3.1.2 Period determination

The natural period of the structure, T, can be obtained by a modal extraction process, provided that it does not exceed the product of the modal limiting coefficient C_{up} times the approximate natural period of the structure T_a. The value of the C_{up}, which is defined considering the seismic zone 4, is given as 1.5 (Table 10 of ABNT NBR 15421 [3]). The approximate natural period of the structure, T_a, can be obtained by the Equation 4:

$$T_a = C_T\, h_n^{\,x} \tag{4}$$

where C_T = coefficient function of the earthquake-resistant system; x = parameter that varies also according to the earthquake-resistant system; h_n = height of the structure above the base.

The building does not have double symmetry since the dimensions in directions x and y are different and therefore the fundamental periods are also different. As a model has shear walls, in order to make a comparison with the composed only of concrete frames, it is necessary to obtain the period in the direction in which it provides the greatest influence on stiffness, which is in the direction y, and thus obtain the values of displacements and loads at the base of the building proposed in the research.

3.1.3 Vertical distribution of seismic forces

The total static horizontal force on the base H (Equation 1) is distributed vertically between the several elevations of the structure, so that at each elevation x, a force F_x is applied as defined by Equation 5:

$$F_x = C_{vx}\, H \tag{5}$$

The vertical distribution coefficient, C_{vx}, is defined according to Equation 6:

$$C_{vx} = \frac{w_x\, h_x^k}{\sum_{i=1}^{n} w_i\, h_i^k} \tag{6}$$

where w_i and w_x = total effective weight parts corresponding to elevations i or x, respectively; h_i and h_x = heights between the base and the elevations i or x, respectively; k = distribution exponent, related to the natural period of structure T, with the following values:

- for structures with period less than 0.5 s: $k = 1$;
- for structures with periods between 0.5 s and 2.5 s, $k = (T + 1.5)\,/\,2$;
- for structures with period greater than 2.5 s: $k = 2$.

3.1.4 Torsional moment consideration

In addition to the distribution of horizontal seismic loads, the design must include an inherent torsional moment (M_t) in the floors, caused by the eccentricity of the centers of mass relative to the centers of rigidity, plus an accidental torsional moment, M_{ta}. This moment is determined considering a displacement of the center of mass in each direction equal to 5% of the dimension of the structure, in a direction parallel to the axis perpendicular to the direction of application of the horizontal forces. As the building has double symmetry, the centers of gravity (CG) of the slabs are located at the intersections of the main axes. Such slabs behave like rigid diaphragms. In seismic category *C* structures, where the models of this paper are included, if there are structural irregularities of type 1 (torsional), M_{ta} must be multiplied by an amplification factor of the torsional moment A_x, obtained by Equation 7.

$$A_x = \left(\frac{\delta_{máx}}{1.2\ \delta_{avg}}\right)^2 \tag{7}$$

where $\delta_{máx}$ = maximum horizontal displacement in one direction, in the dimension *x*; δ_{avg} = average of displacements in the same direction, at the extreme points of the structure on an axis transverse to this direction.

3.1.5 Absolute and relative displacements

The displacements resulting from the static analysis, δ_e, are obtained directly from the computer program after the application of the referred horizontal forces F_x and moments M_{ta}. The absolute displacements, δ, evaluated at the centers of mass, are determined by multiplying δ_e by the ratio between the amplification coefficient of the displacements C_d (variable according to the earthquake-resistant system) and by the importance factor *I*. The relative displacements, Δ, are determined as the difference between the absolute displacements in the centers of mass in the elevations above and below the storey in consideration. The latter are limited to 2% of the floor height (0.020 h_s).

3.1.6 Second-order effects

As for second-order effects, ABNT NBR 15421 [3] disregards their consideration on a given storey if the stability coefficient θ (Equation 8) is less than 0.10.

$$\theta = \frac{P_i\ \Delta_i}{H_i\ h_{si}\ C_d} \tag{8}$$

where P_i = vertical force acting on the floor *i*, obtained with multiplying factors of load taken equal to 1.0; Δ_i = relative displacement of the floor; H_i = seismic shear force acting on floor *i*; h_{si} = distance between the two elevations corresponding to the storey in analysis; C_d = amplification coefficient of the displacements.

The value of the stability coefficient θ cannot exceed the maximum value $\theta_{máx}$, defined according to the Equation 9:

$$\theta_{máx} = \frac{0.5}{C_d} \tag{9}$$

The ABNT NBR 15421 [3] limits $\theta_{máx} \leq 0.25$. If the value of θ is between 0.1 and $\theta_{máx}$, the forces on the elements and the displacements shall be multiplied by the factor 1.0 / (1 – θ).

3.2 Modal Response Spectrum analysis

The Complete Quadratic Combination (CQC) modal responses are used to obtain the maximum value of any elastic effect, d_{ij}. In this case, it was used to obtain the forces and displacements in the building. The CQC formulation, proposed by Wilson et al. [24], which is based on the theory of random vibrations, is given in Equation 10:

$$d_{ij} = \sqrt{\sum_{i=1}^{N} \sum_{j=1}^{N} d_i\ \rho_{ij}\ d_j} \tag{10}$$

The correlation coefficient, ρ_{ij}, can be approximated by Equation 11:

$$\rho_{ij} = \frac{8\,(\xi_i\,\xi_j)^{1/2}\,(\xi_i + r_{ij}\,\xi_j)\,r_{ij}^{3/2}}{(1 - r_{ij}^2)^2 + 4\,\xi_i\,\xi_j\,r_{ij}\,(1 + r_{ij}^2) + 4\,(\xi_i^2 + \xi_j^2)\,r_{ij}^2} \tag{11}$$

where ξ_i and ξ_i = damping ratios for modes *i* and *j*, respectively; r_{ij} = ratio of the natural frequencies, given by Equation 12:

$$r_{ij} = \frac{\omega_i}{\omega_j} \tag{12}$$

The modal responses obtained in terms of forces are multiplied by a I / R factor, and the displacements are multiplied by a C_d / R factor.

3.3 Model building with concrete frame structural system

The first analyses are the ones conducted in the building with an earthquake-resistant system composed only of frames and concrete slabs, in both directions. The analyses by the two methods are presented next.

3.3.1 Seismic analysis by Equivalent Lateral Forces procedure

The total permanent load of the building is composed of the dead load (G) and the live load (Q). In the system composed of concrete frames with usual detailing, the dead load is formed by the weight of the columns, walls, and concrete slabs. In the dual system, the weight of the shear walls is included. It was established, according to ABNT NBR 6120 [25], for office buildings, a live load of 2.5 kN/m^2, considered on all floors; on the roof, the live load is 1.5 kN/m^2.

Linear analyses were performed considering the Exceptional Combination of the Ultimate Limit State (ULS) of ABNT NBR 6118 [17]. These analyses involve the total permanent loads and the seismic forces obtained by the Equivalent Lateral Forces method. Equation 13 presents the combination used:

$$F_d = 1.2\ G + 1.0\ Q + 1.0\ Q_{exc} \tag{13}$$

where F_d = design load for the combination of the Ultimate Limit State; G = dead load; Q = live load; Q_{exc} = seismic horizontal force.

The total permanent load of building with concrete frame structural system is 24697 kN. The response modification coefficient R is equal to 3.0 (Table 6 of ABNT NBR 15421 [3]) and the utilization importance factor is $I = 1.0$ (Table 4 of ABNT NBR 15421 [3]). As in this system, horizontal seismic forces are 100% resisted by frames of concrete in both directions, not being these connected to more rigid systems that prevent their free deformation when subjected to seismic action, C_T and x coefficients for determining the approximate natural period of the structure T_a are 0.0466 and 0.9, respectively, with $h_n = 36$ m.

For the analyses referring to the ELF method, for the T periods in the x and y directions, those obtained in the first frequency mode with the modal analysis of the structure are considered. The values in each direction are show in Table 2:

Table 2. Building with concrete frame structural system in both directions: Parameters.

Parameters	Direction x	Direction y
1st period of structure (T) – obtained computationally:	0.88 s	0.92 s
Period limitation coefficient (C_{up}) - Table 10*:	1.5	1.5
Period coefficients: C_T:	0.0466	0.0466
x:	0.9	0.9
Height of the structure above the base (h_n):	36 m	36 m
Approximate natural period of the structure (T_a): $C_T \cdot h_n^x$	1.17 s	1.17 s
$C_T \cdot h_n^x \cdot C_{up} > T$	1.78 s	1.78 s
Use importance factor (I) - Table 4*:	1.0	1.0
Response modification coefficient (R) – Table 6*:	3.0	3.0
I / R ratio:	0.3333	0.3333
Design characteristic acceleration (a_g): 0.15 g (Zone 4) – Table 1*:	1.47 m/s^2	1.47 m/s^2
Seismic amplification factor for the period of 0.0 s (C_a) – Table 3*:	1.5	1.5
Spectral acceleration for the period of 0.0 s (a_{gs0}): $C_a \cdot a_g$	2.21 m/s^2	2.21 m/s^2
Seismic amplification factor for the period of 1.0 s (C_v) – Table 3*:	2.2	2.2
Spectral acceleration for the period of 1.0 s (a_{gs1}): $C_v \cdot a_g$	3.23 m/s^2	3.23 m/s^2
Seismic response coefficient (C_s): $2.5\ I\ a_{gs0} / (g\ R)$	0.19	0.19
Maximum seismic response coefficient (C_s): $I\ a_{gs1} / (g\ R\ T)$	0.13	0.12
Distribution exponent (k) for 0.5 s < T < 2.5 s: $(T + 1.5) / 2$	1.19	1.21
Amplification coefficient of the displacements (C_d) – Table 6*:	2.5	2.5
C_d / R ratio:	0.8333	0.8333

*See ABNT NBR 15421 [3].

The values of the equivalent lateral forces F_x, F_y, $M_{ta,x}$ and $M_{ta,y}$ are presented in Table 3. The displacements at the nodes of the centers of mass obtained by the static analysis, δ_e, the absolute displacements δ, the relative displacements Δ and the stability coefficients θ are shown in Table 4.

According to Table 4, the values of the stability coefficients θ_x and θ_y did not exceed the value 0.1 (see Equation 8), so the second-order effects on the floors may be disregarded. Then, it was not necessary to increase forces and displacements. The relative displacements were also not greater than the limit of 0.020 h_{s}= 0.020 × 300 = 6 cm.

It is worth mentioning that the structure did not present any irregularity in the plane, Therefore, the torsional moments were not amplified by A_x.

Table 3. Building with concrete frame structural system: horizontal forces F per floor with the ELF method.

Storey	w_x [kN]	Direction x			Direction y		
		C_{vx}	F_x [kN]	$M_{ta,x}$ [kN·m]	C_{vy}	F_y [kN]	$M_{ta,y}$ [kN·m]
1st	2058.09	0.0086	26.08	15.65	0.0083	24.10	19.28
2nd	2058.09	0.0198	59.58	35.75	0.0193	55.76	44.61
3rd	2058.09	0.0320	96.61	57.96	0.0315	91.09	72.87
4th	2058.09	0.0451	136.12	81.67	0.0446	129.03	103.22
5th	2058.09	0.0589	177.60	106.56	0.0584	169.03	135.23
6th	2058.09	0.0732	220.71	132.42	0.0728	210.77	168.62
7th	2058.09	0.0880	265.22	159.13	0.0878	254.01	203.20
8th	2058.09	0.1031	310.98	186.59	0.1031	298.56	238.85
9th	2058.09	0.1187	357.85	214.71	0.1190	344.31	275.45
10th	2058.09	0.1346	405.74	243.44	0.1351	391.14	312.91
11th	2058.09	0.1507	454.55	272.73	0.1517	438.97	351.17
Roof	2058.09	0.1672	504.22	302.53	0.1685	487.72	390.18
Σ (Base)	24697.07	1.0	3015.28		1.0	2894.48	

Table 4. Building with concrete frame structural system: relative displacements and stability coefficients from the ELF procedure.

Storey	Direction x				Direction y			
	δ_{xe} [cm]	δ_x [cm]	Δ_x [cm]	θ_x [$\times 10^{-4}$]	δ_{ye} [cm]	δ_y [cm]	Δ_y[cm]	θ_y [$\times 10^{-4}$]
1st	0.25	0.64	0.64	85.19	0.25	0.64	0.64	88.80
2nd	0.69	1.72	1.08	66.91	0.70	1.74	1.10	71.00
3rd	1.15	2.88	1.16	44.38	1.17	2.94	1.20	47.60
4th	1.62	4.04	1.16	30.87	1.66	4.14	1.20	33.37
5th	2.07	5.16	1.12	22.37	2.13	5.32	1.18	24.34
6th	2.49	6.23	1.07	16.58	2.58	6.45	1.13	18.16
7th	2.89	7.23	1.00	12.40	3.00	7.50	1.06	13.66
8th	3.25	8.13	0.90	9.22	3.39	8.47	0.96	10.24
9th	3.57	8.91	0.79	6.73	3.73	9.32	0.85	7.55
10th	3.83	9.57	0.65	4.73	4.01	10.03	0.72	5.38
11th	4.03	10.07	0.50	3.09	4.24	10.60	0.56	3.61
Roof	4.17	10.41	0.35	1.79	4.40	11.01	0.41	2.19

3.3.2 Seismic analysis by the Modal Response Spectral analysis

An important factor in dynamic analysis is the mass mobilization of the structure analyzed by the vibration mode. According to ABNT NBR 15421 [3], the number of modes to be considered in the spectral analysis must be sufficient to capture at least 90% of the total mass in each of the orthogonal directions considered in the analysis. For the 12-story building with framed system, 90% of the building mass is mobilized upon reaching the 3rd vibration mode, that is, 90.3% (direction x) and 90.2% (direction y). Figure 5 shows the mass participation of each mode in relation to the total. Figure 6 and Figure 7 present the three fundamental modes and vibration period of building, in x and y directions.

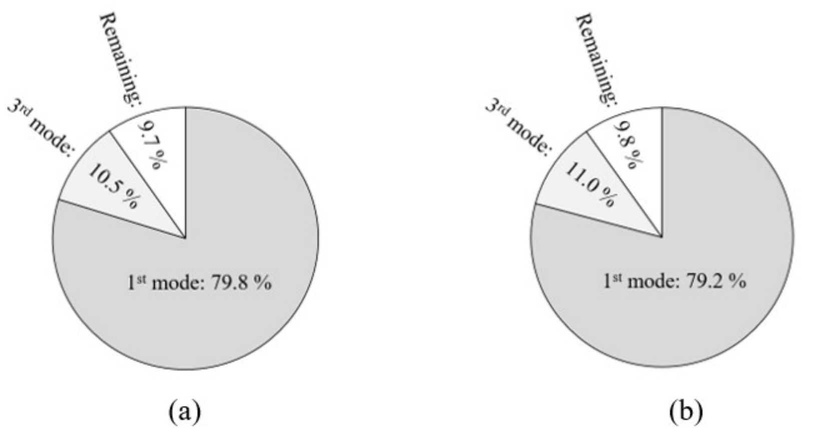


Figure 5. Mass mobilized by vibration mode: (a) x direction; (b) y direction.

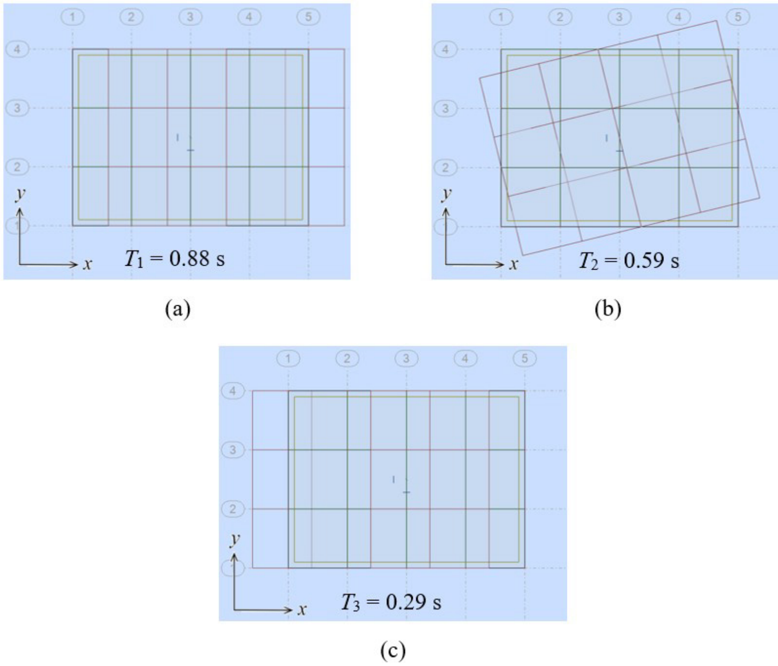


Figure 6. Structural system formed only by concrete frames - three fundamental modes of vibration in *x* direction: (a) 1st mode: predominantly translation; (b) 2nd mode: predominantly torsional; (c) 3rd mode: translation.

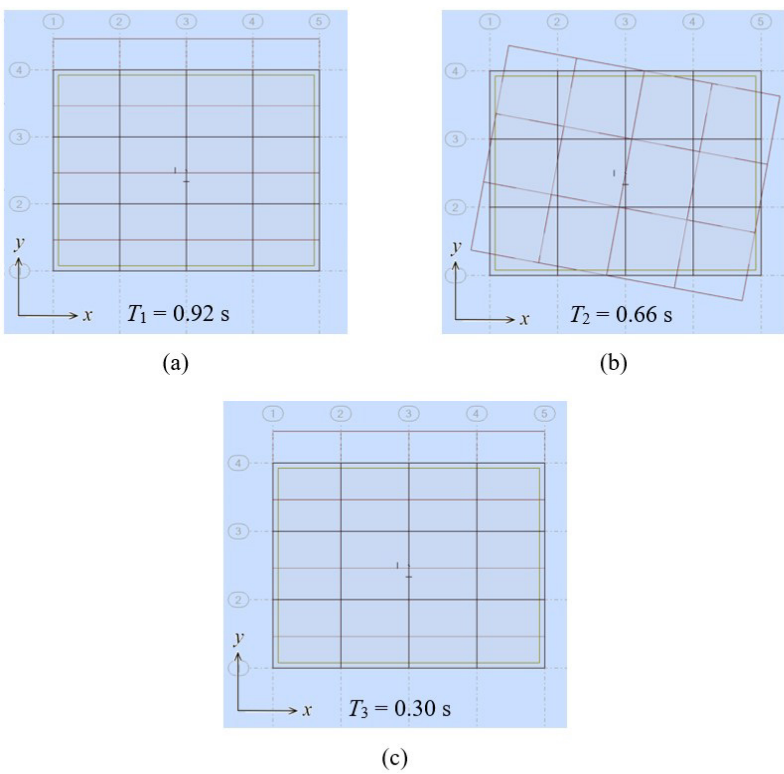


Figure 7. Structural system formed only by concrete frames - three fundamental modes of vibration in *y* direction: (a) 1st mode: predominantly translation; (b) 2nd mode: predominantly torsional; (c) 3rd mode: translation.

Considering the periods of vibration in the building, their highest values occur in the 1st mode in the *x* and *y* directions and are, respectively, 0.88 s and 0.92 s.

By the Modal Response Spectral analysis, the displacements δ_e were obtained directly by the computer program. The absolute displacements are obtained by the multiplication of δ_e by C_d / R equal to 0.8333. This value is valid for the *x* and *y* directions of the structure. Table 5 shows the values of these displacements.

Table 5. Building with concrete frame structural system: nodal displacements with the Modal Response Spectral analysis.

Storey	Direction *x*			Direction *y*		
	δ_{xe} [cm]	δ_x [cm]	Δ_x [cm]	δ_{ye} [cm]	δ_y [cm]	Δ_y[cm]
1st	0.63	0.52	0.52	0.63	0.52	0.52
2nd	1.68	1.40	0.88	1.70	1.42	0.89
3rd	2.80	2.34	0.93	2.85	2.37	0.96
4th	3.90	3.25	0.91	3.98	3.32	0.94
5th	4.94	4.12	0.87	5.07	4.22	0.91
6th	5.92	4.93	0.81	6.09	5.07	0.85
7th	6.80	5.67	0.74	7.02	5.85	0.78
8th	7.59	6.33	0.66	7.87	6.55	0.70
9th	8.26	6.89	0.56	8.59	7.16	0.61
10th	8.81	7.34	0.46	9.20	7.66	0.50
11th	9.22	7.68	0.34	9.66	8.05	0.39
Roof	9.50	7.92	0.23	10.00	8.33	0.28

3.4 Model building with dual system, composed by frame and frame/shear walls

For the building with dual system the weight of the shear walls equal to 2.5 kN/m^2 was included. The total permanent load is 26856 kN. In this case, the earthquake-resistant system is different in both directions: (*i*) on the *x* axis, it is composed of frames; (*ii*) on the *y* axis, it is formed by a dual system (frames and shear walls).

3.4.1 Seismic analysis by the Equivalent Lateral Forces procedure

As well as the building previously analyzed, in this one, in the direction *x*, the seismic horizontal forces are resisted only by concrete frames. Therefore, $C_T = 0.0466$ and $x = 0.9$ (see item 9.2 of ABNT NBR 15421 [3]). In the direction *y*, the seismic resistant system is dual, composed of concrete frames and shear walls, both with usual detailing. In this direction, $C_T = 0.0488$ and $x = 0.75$. The values in each direction are show in Table 6 and the values of F_x, F_y, $M_{ta,x}$ and $M_{ta,y}$ are shown in Table 7.

Table 6. Building with dual frame system: Parameters.

Parameters	Direction *x*	Direction *y*
1st period of structure (T) – obtained computationally:	0.92 s	0.65 s
Period limitation coefficient (C_{up}) - Table 10*:	1.5	1.5
Period coefficients: C_T:	0.0466	0.0488
x:	0.9	0.75
Height of the structure above the base (h_n):	36 m	36 m
Approximate natural period of the structure (T_a): $C_T \cdot h_n^x$	1.17 s	0.72 s
$C_T \cdot h_n^x \cdot C_{up} > T$	1.78 s	1.08 s
Use importance factor (I) - Table 4*:	1.0	1.0
Response modification coefficient (R) – Table 6*:	3.0	4.5
I / R ratio:	0.3333	0.2222
Design characteristic acceleration (a_g): 0.15 g (Zone 4) – Table 1*:	1.47 m/s^2	1.47 m/s^2
Seismic amplification factor for the period of 0.0 s (C_a) – Table 3*:	1.5	1.5
Spectral acceleration for the period of 0.0 s (a_{gs0}): $C_a \cdot a_g$	2.21 m/s^2	2.21 m/s^2
Seismic amplification factor for the period of 1.0 s (C_v) – Table 3*:	2.2	2.2
Spectral acceleration for the period of 1.0 s (a_{gs1}): $C_v \cdot a_g$	3.23 m/s^2	3.23 m/s^2
Seismic response coefficient (C_s): 2.5 $I\, a_{gs0} / (g\, R)$	0.19	0.13
Maximum seismic response coefficient (C_s): $I\, a_{gs1} / (g\, R\, T)$	0.12	0.11
Distribution exponent (k) for 0.5 s < T < 2.5 s: $(T + 1.5) / 2$	1.21	1.08
Amplification coefficient of the displacements (C_d) – Table 6*:	2.5	4.0
C_d / R ratio:	0.8333	0.8889

*See ABNT NBR 15421 [3].

Table 7. Building with dual frame system: Horizontal forces F per floor with the ELF procedure.

Storey	w_x [kN]	Direction x			Direction y		
		C_{vx}	F_x [kN]	$M_{ta,x}$ [kN·m]	C_{vy}	F_y [kN]	$M_{ta,y}$ [kN·m]
1st	2238.00	0.0083	26.81	16.09	0.0110	33.34	26.67
2nd	2238.00	0.0193	62.00	37.20	0.0232	70.24	56.19
3rd	2238.00	0.0315	101.24	60.75	0.0358	108.61	86.89
4th	2238.00	0.0446	143.38	86.03	0.0488	147.97	118.38
5th	2238.00	0.0584	187.80	112.68	0.0621	188.08	150.47
6th	2238.00	0.0728	234.14	140.48	0.0755	228.81	183.05
7th	2238.00	0.0878	282.12	169.27	0.0891	270.05	216.04
8th	2238.00	0.1031	331.57	198.94	0.1029	311.73	249.39
9th	2238.00	0.1189	382.34	229.40	0.1168	353.81	283.05
10th	2238.00	0.1351	434.30	260.58	0.1308	396.24	316.99
11th	2238.00	0.1516	487.37	292.42	0.1449	438.99	351.19
Roof	2238.00	0.1684	541.46	324.84	0.1591	482.04	385.63
Σ (Base)	26856.00	1.0	3214.54		1.0	3029.91	

The obtained displacements δ_e, δ, Δ and θ are shown in Table 8. Structural irregularities are not observed; therefore, it was not necessary to increase the displacements or amplify the accidental torsion moment.

Table 8. Building with concrete frame structural system: relative displacements and stability coefficients with the ELF procedure.

Storey	Direction x				Direction y			
	δ_{xe} [cm]	δ_x [cm]	Δ_x [cm]	θ_x [× 10^{-4}]	δ_{ye} [cm]	δ_y [cm]	Δ_y[cm]	θ_y [× 10^{-4}]
1st	0.26	0.66	0.66	88.11	0.06	0.26	0.26	23.13
2nd	0.72	1.79	1.13	70.39	0.20	0.79	0.53	21.88
3rd	1.21	3.01	1.23	46.99	0.38	1.50	0.71	18.28
4th	1.70	4.24	1.22	32.75	0.59	2.34	0.84	15.03
5th	2.17	5.43	1.19	23.74	0.81	3.25	0.91	12.27
6th	2.62	6.56	1.13	17.60	1.05	4.20	0.95	9.98
7th	3.05	7.61	1.05	13.15	1.29	5.15	0.95	8.09
8th	3.43	8.57	0.95	9.78	1.52	6.08	0.93	6.53
9th	3.76	9.40	0.83	7.14	1.74	6.96	0.89	5.25
10th	4.04	10.09	0.69	5.01	1.95	7.80	0.84	4.21
11th	4.25	10.62	0.53	3.29	2.15	8.58	0.78	3.37
Roof	4.40	10.99	0.37	1.92	2.33	9.31	0.73	2.64

3.4.2 Seismic analysis by the Modal Response Spectral analysis

By the Spectral analysis, the absolute displacements δ were obtained by multiplying δ_e by a C_d / R factor, and for each direction different values resulted (see item 3.4.1 – Table 6). Thus, in the x direction, $\delta_x = 0.8333\ \delta_{xe}$, and in the y direction, $\delta_y = 0.8889\ \delta_{ye}$. The displacements values can be seen in Table 9.

Table 9. Building with dual frame system: nodal displacements with the Modal Response Spectrum analysis.

Storey	Direction x			Direction y		
	δ_{xe} [cm]	δ_x [cm]	Δ_x [cm]	δ_{ye} [cm]	δ_y [cm]	Δ_y[cm]
1st	0.64	0.53	0.53	0.21	0.19	0.19
2nd	1.73	1.44	0.91	0.65	0.58	0.39
3rd	2.89	2.41	0.97	1.23	1.10	0.52
4th	4.03	3.36	0.95	1.92	1.71	0.61
5th	5.11	4.26	0.90	2.67	2.37	0.66
6th	6.12	5.10	0.84	3.44	3.06	0.69
7th	7.05	5.87	0.77	4.22	3.75	0.69
8th	7.86	6.55	0.68	4.98	4.43	0.68
9th	8.56	7.13	0.58	5.71	5.07	0.64
10th	9.12	7.60	0.47	6.39	5.68	0.61
11th	9.55	7.96	0.35	7.03	6.25	0.57
Roof	9.84	8.20	0.24	7.62	6.77	0.52

For the 12-storey building with framed and dual system, 93.6% of the building mass is mobilized in the 4th vibration mode in direction x. In the direction y, 92.6% is mobilized in the 6th mode. Figure 8 shows the mass participation of each mode in relation to the total. Figures 9-10 present the three fundamental modes and vibration period of building, in x and y directions.

The highest values of periods of vibration in the building with dual system occur also in the 1st mode in the x and y directions and are, respectively, 0.92 s and 0.65 s.

By the Spectral Analysis, the displacements δ_e were obtained directly in the computer program. The absolute displacements δ are obtained by the multiplication of δ_e by C_d / R, with C_d and R having different values in the x and y axis. Tables 4 and 6 shows the values of these displacements.

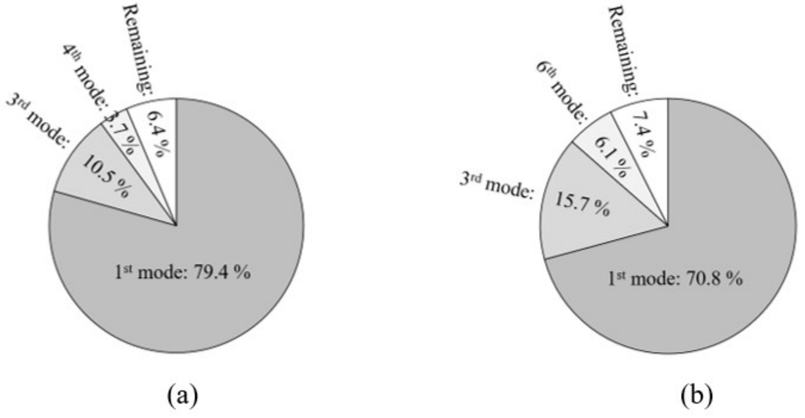


Figure 8. Mass mobilized by vibration mode in building with dual frame system: (a) x direction; (b) y direction.

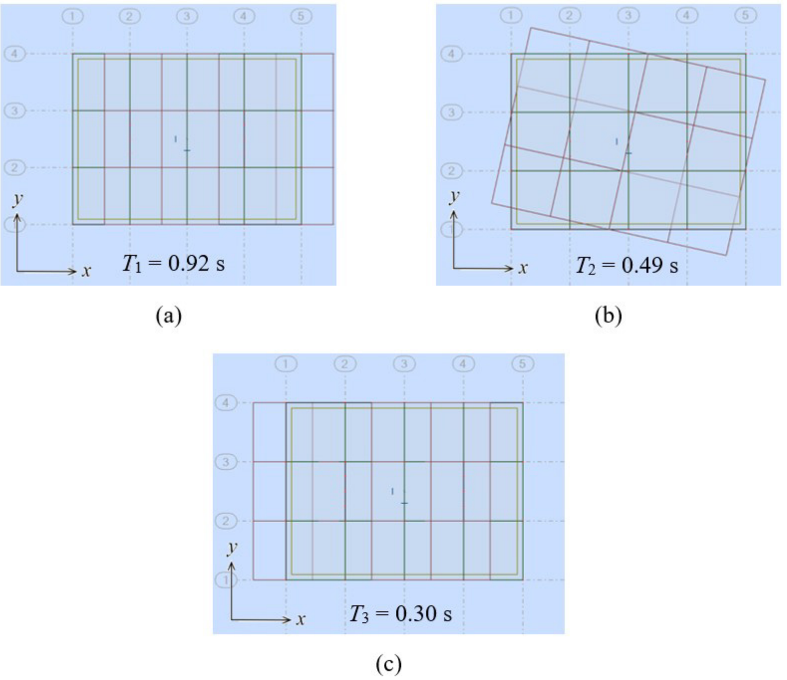


Figure 9. Building with dual frame system - three fundamental modes of vibration in x direction: (a) 1st mode: predominantly translation; (b) 2nd mode: predominantly torsional; (c) 3rd mode: translation.

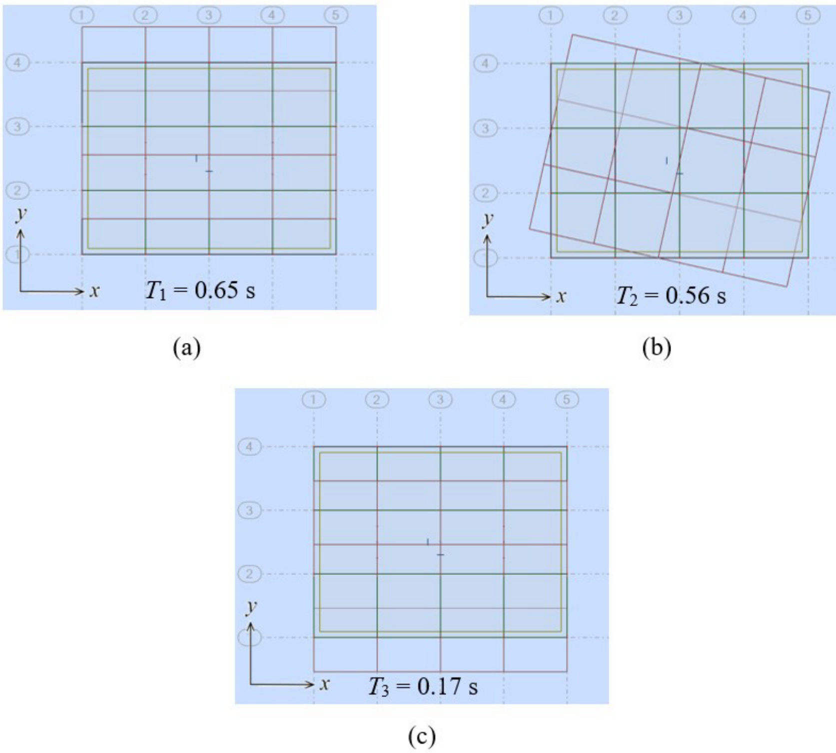


Figure 10. Structural system formed only by concrete frames - three fundamental modes of vibration in y direction: (a) 1st mode: predominantly translation; (b) 2nd mode: predominantly torsional; (c) 3rd mode: translation.

4 RESULTS AND DISCUSSIONS

This section discusses the results obtained in terms of displacements and forces at the base of buildings from the analyzes by the Equivalent Lateral Forces procedure and Modal Response Spectrum analysis described in the previous items.

4.1 Model building with concrete frame structural system

The ABNT NBR 15421 [3] stipulates that the project modal responses at the base of the building (H_{tx} and H_{ty}) are obtained by multiplying the elastic responses (H_{txe} and H_{tye}), arising from the computational analysis, by the I / R factor.

In the case of a building with an earthquake-resistant system formed only by frames, $I / R = 0.3333$, in both directions. As $H_{txe} = 7459$ kN and $H_{tye} = 7152$ kN, then $H_{tx} = 2486$ kN and $H_{ty} = 2384$ kN. By the Equivalent Lateral Forces procedure, the forces at the base are $H_x = 3015$ kN and $H_y = 2895$ kN.

However, if the total horizontal force at the base determined by the spectral process, H_t, in one direction, is less than 0.85 H (less than 85% of the horizontal force determined by the static process), all elastic forces obtained in this direction must be multiplied by 0.85 H/H_t. This correction does not apply to absolute and relative displacements. Therefore, in direction x, 0.85 $H_x = 0.85 \times 3015 = 2563$ kN.

As 2486 kN is less than 2563 kN, so $H_x = 3015 \times 0.85\ (3015 / 2486) = 3198$ kN. In direction y, 0.85 $H_y = 0.85 \times 2895 = 2461$ kN. As 2384 kN is less than 2461 kN, so $H_y = 2895 \times 0.85\ (2895 / 2384) = 2988$ kN.

In summary, the modal responses are $H_{tx} = 2486$ kN and $H_{ty} = 2384$ kN; the elastic responses are $H_x = 3198$ kN and $H_y = 2988$ kN. Differences are of approximately 22% and 20% between the values obtained by the two methods in each direction.

Figure 11 presents the responses in terms of absolute displacements on each floor in the x and y directions obtained by the two analyses methods and the horizontal resulting forces in the base of the building. The percentages in parentheses represent the differences in terms of absolute displacements between the two methods. The black arrows are the horizontal forces at the base by the Equivalent Lateral Forces (ELF) procedure and the gray arrows are the forces at the base obtained by the Modal Response Spectrum analysis.

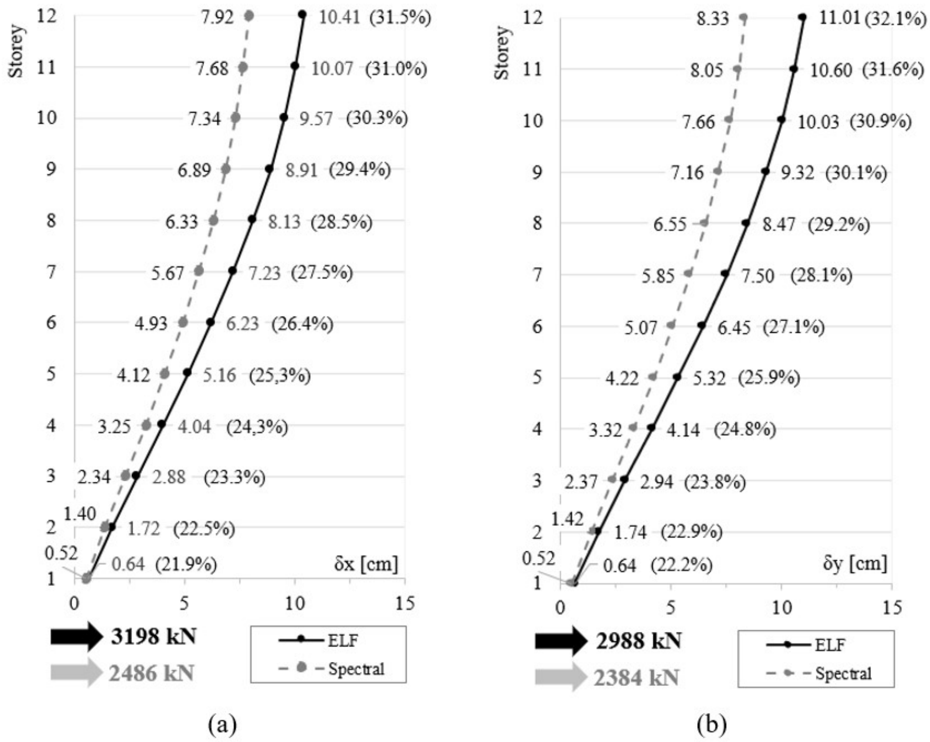


Figure 11. Frame system: Absolute displacements and total horizontal forces at the base: (a) Direction *x*; (b) Direction *y*.

According to Figure 11, the static method is more conservative, since the absolute displacements obtained were greater than those obtained by the spectral method in both directions. These are greater in *y*, since in this direction the stiffness of the structure is lower. As expected, the largest displacements were on the 12th floor with the greatest difference also between the two methods. The relative displacements, however, were higher on the lower floors (see Tables 4-5).

4.2 Model building with dual system with concrete frame and shear walls

In the case of the building with dual earthquake-resistant system, the I / R ratio has different values in the *x* and *y* directions (see item 3.4.1). The modal responses in terms of forces at the base of the building obtained by the computational analysis, H_{txe} and H_{tye}, are, respectively, 7845 kN and 10008 kN, and that multiplied by the I / R factors equal to 0.3333 and 0.2222 generated H_{tx} = 2615 kN and H_{ty} = 2224 kN. By the Equivalent Lateral Forces procedure, the forces at the base are H_x = 3215 kN and H_y = 3030 kN.

In direction *x*, 0.85 H_x = 0.85 × 3215 = 2733 kN. As 2615 kN is less than 2733 kN, so H_x = 3215 × 0.85 (3215 / 2615) = 3360 kN. In direction *y*, 0.85 H_y = 0.85 × 3030 = 2576 kN. As 2224 kN is also less than 2576 kN, so H_y = 3030 × 0.85 (3030 / 2224) = 3509 kN.

In summary, the modal responses are H_{tx} = 2615 kN and H_{ty} = 2224 kN; the elastic responses are H_x = 3360 kN and H_y = 3509 kN. Differences are of approximately 22% and 37% between the values obtained by the two methods in each direction.

Figure 12 shows the responses in terms of absolute displacements on each floor in the *x* and *y* directions obtained by the two analyses methods and the horizontal resulting forces in the base of the building. The percentages in parentheses represent the differences in terms of absolute displacements between the two methods. The black arrows are the horizontal forces at the base by the Equivalent Lateral Forces (ELF) procedure and the gray arrows are the forces at the base obtained by the Modal Response Spectrum analysis.

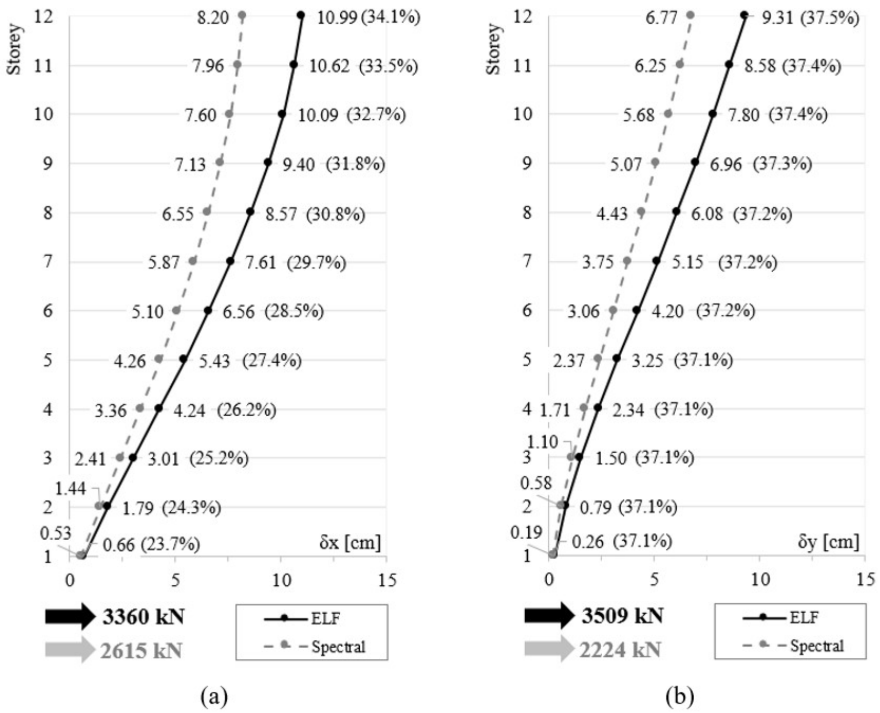
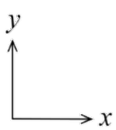
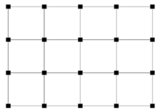
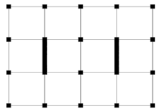


Figure 12. Frame / shear walls system: Absolute displacements and total horizontal forces at the base: (a) Direction x; (b) Direction y.

According to Figure 12, the static method is also more conservative for this building, with the absolute displacements in y greater than those in x. As in the building with only frames, the relative displacements were higher on the lower floors (see Tables 8-9).

Finally, Table 10 presents a summary of the results obtained by the two analyses in terms of loads applied to the base of the buildings and absolute displacements in the roof. The positive sign in the differences in results between the two analysis methods indicates an increase in the evaluated term; negative sign indicates reduction.

Table 10. Results in terms of loads applied at the base of building and absolute displacements in the roof.

	**Direction x**				**Direction y**			
	ELF		**Spectral**		**ELF**		**Spectral**	
	H_x [kN]	**δ_x [cm]**	**H_{tx} [kN]**	**δ_{tx} [cm]**	**H_y [kN]**	**δ_y [cm]**	**H_{ty} [kN]**	**δ_{ty} [cm]**
	3198	10.41	2486	7.92	2988	11.01	2384	8.33
	3360	10.99	2615	8.20	3509	9.31	2224	6.77
Difference [%]	+ 5.1	+ 5.6	+ 5.2	+ 3.5	+ 17.4	- 15.4	- 6.7	- 18.7

According to the results presented in Table 10, it is possible comment:

(i) In the direction x, by the two analysis methods, building formed with the dual structural system presented a higher load on the base in relation to the one formed only by frames. A similar increase of about 5.1% in responses. The absolute displacements in the roof were also higher in this direction in the building with shear walls (about 5.6% in those obtained by the static method and 3.5% in those obtained by the dynamic method).

(ii) In the direction y, the increase in the dual building occurred only in the load obtained by the horizontal forces method and it was more significant than that in the direction x, that is, an increase of 17.4%. As for that provided by the spectral method, there was a reduction of about 6.7%. The displacements also suffered a reduction of about 15.4% and 18.7%, respectively provided by the static and dynamic analysis, despite the increase in the applied load in the direction y. It can be concluded that the shear walls provide greater stiffness in the structure when arranged parallel to evaluated axis.

5 CONCLUSIONS

This research aimed to verify the influence of shear walls on the stiffness of the reinforced concrete structure in a multi-storey hypothetical building subjected to seismic action. Two models were developed in a computer program. One with an earthquake-resistant system composed only by frames in both directions and the other one with a dual system, composed of concrete shear walls, both with the usual detailing. Two analysis methods were considered according to the Brazilian Code ABNT NBR 15421 [3]: Equivalent Lateral Forces procedure and Modal Response Spectrum analysis.

Analyzing the results, it was found that, by the static method, the absolute displacements obtained were greater than those achieved by the spectral method in both directions and in both buildings. In the building with a seismic resistant system formed only by frames, the displacements were higher in the y direction, even with the lower applied load in this direction. This can be explained by the lower stiffness of the structure in this direction. The building with shear walls, on the other hand, the opposite occurs. That is, greater displacements in the x direction, since the walls provide greater stiffness to the structure in the y direction.

Comparing the displacements in the roof in the x direction between the two buildings (Table 10), in both methods, the building with shear walls presented higher values than the one with the structure formed only by frames, because its dead load is greater, and the seismic forces are directly related to these loads. However, such values were shown to be equivalent in both models, since the relationship between applied loads and displacements is similar, that is, with a difference of about 5%.

Thus, it can be concluded that shear walls arranged perpendicularly to the x-axis do not provide a significant increase in building stiffness in this direction. In the y-axis, there is a reduction in absolute displacements of approximately 15.4% from the structural system composed only of concrete frames to the one with shear walls by the static method and 18.7% by the spectral method. It can be finally concluded, for this case, that the shear walls located parallel to axis y provided a significant increase in the building stiffness.

However, it is worth mentioning that, to draw more general conclusions, that analyses should be expanded. It would be more appropriate to analyze other structures (symmetrical or not) in different seismic regions with different shear walls (simple and composite).

REFERENCES

[1] W. L. Santos, C. A. Crisóstomo, A. R. F. Barbosa, P. M. Silva, and F. I. C. Nascimento, “Atividades sísmicas na Amazônia: levantamento e caracterização de terremotos na Amazônia Sul-ocidental – Acre – Brasil,” *GeoUECE.*, vol. 8, no. 15, pp. 66–77, Jul. 2019. Accessed: Oct. 10, 2021. [Online]. Available: https://revistas.uece.br/index.php/GeoUECE/article/view/2164

[2] J. B. Paiva No., P. G. B. Nóbrega, and S. H. S. Nóbrega, “Avaliação da resposta sísmica de edifícios de concreto na região Nordeste segundo métodos da NBR 15421,” in *Proc. 58º Congr. Bras. Concreto.*, Oct. 2016, pp. 1–16.

[3] Associação Brasileira de Normas Técnicas, *Projeto de Estruturas Resistentes a Sismos – Procedimento, NBR 15421*, 2006.

[4] D. Giardini, G. Grünthal, K. M. Shedlock, and P. Zhang. “The GSHAP global seismic hazard map,” *Ann. Geophys.*, vol. 42, no. 6, pp. 1225–1230, Dec. 1999. https://doi.org/10.4401/ag-3784.

[5] P. S. T. Miranda, H. S. A. Varum, and N. S. Vila Pouca, “Sismicidade brasileira: registros sísmicos indicam a configuração de um novo mapa,” in *XIV Int. Conf. Building Pathology Constructions Repair.*, Jun. 2018, pp. 1–14.

[6] M. H. Beedalah and S. K. Mohaisen, “Performance of reinforced concrete building with shear wall under the effect of seismic loads,” *Int. J. Modern Res. Eng. Technol.*, vol. 6, no. 21, pp. 367–375, Sep. 2021.

[7] M. A. Elwi and W. G. A. Hussein,“Study of effect of shear wall in the seismic response of existing buildings,” *Periodicals Eng. Natural Sciences*, vol. 9, no. 3, pp. 494–508, Aug. 2021.

[8] S. A. Hossain and A. Bagchi, “Seismic performance of existing buildings concrete shear wall,” in *5th World Cong. Civil, Structural, Environmental Eng.*, Oct. 2020, pp. 1–10.

[9] D. Ugalde and D. Lopez-Garcia, "Analysis of the seismic capacity of Chilean residential RC shear wall buildings," *J. Building Eng.*, vol. 31, 101369, Sep. 2020.

[10] S. A. Ahamad and K. V. Pratap, "Dynamic analysis of G + 20 multi storied building by using shear walls in various locations for different seismic zones by using Etabs," *Materials Today: Proc.*, vol. 43, Pt 2, pp. 1043–1048, Apr. 2021. https://doi.org/10.1016/j.matpr.2020.08.014.

[11] H. Alimohammadi, M. D. Esfahani, and M. L. Yaghin, "Effects of openings on the seismic behavior and performance level of concrete shear walls," *Int. J. Eng. Appl. Sciences*, vol. 6, no. 10, pp. 33–39, Oct. 2019. Accessed: Dec. 6, 2022. [Online]. Available: https://www.researchgate.net/publication/337596926

[12] T. A. Ozkul, A. Kurtbeyoglu, M. Borekci, and B. Zengin, "Effect of shear wall on seismic performance of RC frame buildings," *Eng. Failure Anal.*, vol. 100, pp. 60–75, Feb. 2019. https://doi.org/10.1016/j.engfailanal.2019.02.032.

[13] H. H. Jammani, J. V. Amiri, and H. Rajabnejad, "Energy distribution in RC shear wall-frame structures subject to repeated earthquakes," *Soil Dy. Earthq. Eng.*, vol. 107, pp. 116–128, Apr. 2018. https://doi.org/10.1016/j.soildyn.2018.01.010.

[14] J. Tarigan, J. Manggala, and T. Sitorus, "The effect of shear wall location in resisting earthquake," *IOP Conf. Series Mater. Sci. Eng.*, vol. 309, pp. 1–6, Mar. 2018. http://dx.doi.org/10.1088/1757-899X/309/1/012077.

[15] D. Ugalde and D. Lopez-Garcia, "Behavior of reinforced concrete shear wall buildings subjected to large earthquakes," *Procedia Eng.*, vol. 199, pp. 3582–3587, Sep. 2017.

[16] E. M. V. Pereira, G. H. F. Cavalcante, I. D. Rodrigues, L. C. M. Vieira Jr., and G. H. Siqueira "Seismic reliability assessment of a non-seismic reinforced concrete framed structure designed according to ABNT NBR 6118:2014," *Rev. IBRACON Estrut. Mater.*, vol. 15, no. 1, e15110, Aug. 2022. https://doi.org/10.1590/S1983-41952022000100010.

[17] Associação Brasileira de Normas Técnicas, *Projeto de Estruturas de Concreto – Procedimento*, NBR 6118, 2014.

[18] R. R. Aguero, J. R. Y. Aguirre, M. Schmitz, and C. H. H. Viegas. "Structural retrofitting method for evaluation RC jacketing in columns with amplification of moments under seismic loads," *Rev. IBRACON Estrut. Mater.*, vol. 15, no. 5, e15510, Apr. 2022. https://doi.org/10.1590/S1983-41952022000500010.

[19] D. B. M. Matos, "Otimização do posicionamento da alvenaria estrutural de vedação para minimização de deslocamentos laterais em edifícios submetidos a excitações sísmicas," M.S. thesis, Univ. Federal Rio Grande do Sul, Porto Alegre, Brazil, 2021.

[20] G. M. S. Alva, A. O. Rigão, J. Kaminski Jr., and M. A. S. Pinheiro "Seismic analysis of reinforced concrete building with participating masonry infills," *Rev. IBRACON Estrut. Mater.*, vol. 14, no. 3, e14315, Mar. 2021. https://doi.org/10.1590/S1983-41952021000300015.

[21] G. M. S. Gidrão, P. O. B. Costa, M. Della-libera, and R. M. Bosse, "Simulação dos efeitos sísmicos em um edifício de 12 pavimentos em concreto armado," *Rev. Eng. Tecnol.*, vol. 12, no. 4, pp. 13–25, Dec. 2020.

[22] L. A. P. Peña and G. N. D. Carvalho, "Estudo da influência da configuração estrutural na resposta sísmica de uma estrutura de concreto armado," *Rev. IBRACON Estrut. Mater.*, vol. 8, no. 6, pp. 800–826, Sep. 2017. https://doi.org/10.1590/S1983-41952015000600005.

[23] Autodesk. "Robot Structural Analysis professional version 2021." Autodesk. https://www.autodesk.com/products/robot-structural-analysis/overview (accessed May 8, 2021).

[24] E. L. Wilson, A. D. Kiureghian, and E. P. Bayo, "A replacement for the SRSS method in seismic analysis," *Earthq. Eng. Structural Dyn.*, vol. 9, no. 2, pp. 187–192, Jan. 1981.

[25] Associação Brasileira de Normas Técnicas, *Cargas para o Cálculo de Estruturas de Edificações, NBR 6120*, 2019.

Author contributions: CAS: conceptualization, methodology, formal analysis, writing, data curation, original draft preparation; EFL: formal analysis; original draft preparation; SSL and SHCS: review, edition, supervision.

Editors: José Marcio Calixto, Guilherme Aris Parsekian.