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Evaluation of Horizontal Loads in the Concept Design of Tall Building Concrete Structures

Evaluación de las acciones horizontales en el diseño conceptual de las estructuras de hormigón de edificios en altura

Carlos Muñoz-Blanc[*,a] & Inma Fortea-Navarro[b]

[a] *Serra Hunter Fellow, Departament de Tecnologia de l'Arquitectura (TA), E.T.S. Arquitectura Barcelona, Universitat Politècnica de Catalunya (UPC)*

[b] *Lecturer, Departament de Tecnologia de l'Arquitectura (TA), E.T.S. Arquitectura Barcelona, Universitat Politècnica de Catalunya (UPC)*

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ABSTRACT

Horizontal forces, both from wind and seismic actions, govern in most cases the structural design of tall buildings. An accurate assessment of the magnitude of horizontal actions from initial project stages provides a valuable information for typology choice and preliminary sizing of members. This study develops an analytical evaluation of horizontal forces considering the dynamic effects in this type of buildings to be applied in the initial structural design stages. The research uses analytical methods based on current codes and standards together with numerical Finite Element models and graphic tools that provide a set of original data based on a benchmark case-study. It includes a sensitivity analysis that shows the influence that some parameters, such as structural damping, have in the magnitude of horizontal forces. The study provides new data and a visual analysis method for the two most complex actions in the design of tall buildings. The importance of wind against seism is shown while building stiffness decreases and dynamic effects increase transversal wind actions.

KEYWORDS: Wind load; across-wind vibration; vortex shedding; seismic action; modal analysis; structural damping.

RESUMEN

Las fuerzas horizontales, tanto del viento como de las acciones sísmicas, gobiernan en la mayoría de los casos el diseño estructural de los edificios en altura. Una evaluación precisa de la magnitud de las acciones horizontales desde las etapas iniciales del proyecto proporciona información valiosa para la elección de la tipología y para el predimensionado de la estructura. Este estudio desarrolla una evaluación analítica de las fuerzas horizontales considerando los efectos dinámicos en este tipo de edificaciones, de aplicación en las fases iniciales del diseño estructural. La investigación utiliza métodos analíticos basados en las normativas actuales, combinado con modelos numéricos de elementos finitos y herramientas gráficas que proporcionan un conjunto de datos originales basados en el estudio de casos. Incluye un análisis de sensibilidad que muestra la influencia que tienen diversos parámetros, como el amortiguamiento de la estructura, en la magnitud de los esfuerzos horizontales. El estudio aporta nuevos datos y un método de análisis gráfico para las dos acciones más complejas en el diseño de los edificios en altura. Se muestra la importancia del viento frente al sismo a medida que la rigidez del edificio disminuye y los efectos dinámicos incrementan las acciones transversales del viento.

PALABRAS CLAVE: Acción eólica; vibración eólica transversal; desprendimiento de vórtices; acción sísmica; análisis modal; amortiguamiento estructural.

* Persona de contacto / *Corresponding author*:
Correo-e / *e-mail*: carlos.munoz@upc.edu (Carlos Muñoz-Blanc)

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1. INTRODUCTION

The magnitude of horizontal actions, those due to the wind action and those derived from seismic activity, defines the global strategies that lead to the structural design of tall buildings.

In a low-rise building, the main action of the wind is parallel to its direction and the dynamic effects are irrelevant. However, as the height and flexibility of the building increase, the aeroelastic phenomenon might govern its structural design. In such cases, the across-wind load and the torsional vibration must be evaluated in detail [1,2].

Wind is characterised by its dynamic nature [3]. Its global action in a building can be decomposed into three components [4]: the action that corresponds to the mean wind speed, the background component and the resonant component (Figure 1). The background term is related to the quasi-static response of the fluctuating component of wind, considering that there is no dynamic amplification owing to a possible resonance phenomenon. However, the resonant component represents the amplification of the dynamic action when the frequency of wind is close to the frequency of the structure. The first two components depend fundamentally on the geometry of the building, but not on its stiffness. However, the resonant component depends on the dynamic response of the building and has a significant impact on the across-wind action and torsional vibration.

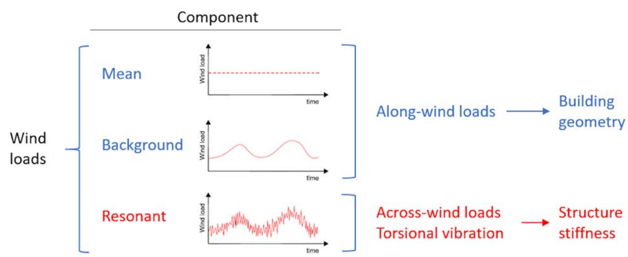


Figure 1. Components of the wind action in tall buildings.

The across-wind action depends mainly on vortex detachment and its detachment frequency [5]. If this frequency is close to the fundamental frequency of the building, resonance will occur, thus affecting the integrity of the structure and people comfort. Fluctuation of the wind pressure on the facades, owing to this vortex detachment and re-adherence, is also responsible for the torsional vibration. This torsional vibration is highly influenced by the torsional stiffness and the geometry of the plan. Torsional effects are relevant in buildings with elongated rectangular plans and when the building has less torsional stiffness. In addition, the eccentricity between the centre of rigidity and the pressure distribution on the facade increases the torsional vibration.

The dynamic wind action in the design of the structure is usually incorporated as equivalent static loads using the equivalent static wind load (ESWL) method. The determination of ESWL allows the combination of the wind action with all vertical loads in the stress analysis of the structural elements.

The procedure for defining ESWL has been widely studied in the field of structural designs. Park et al. presented the most important stages of the method [6], which are explained below in a simplified way from the point of view of the conceptual design of a building for calculating ESWL. The most relevant design stages are:

a) Conceptual design of the structure and typology definition. Determination of the preliminary member sizes.
b) Global analysis of the structure by considering its mechanical and dynamic properties and using a lumped-mass structural model.
c) With the climatological data of the site, the time histories of the aerodynamic loads on each floor are determined from the aerodynamic pressure obtained from a wind tunnel study or using a computational fluid dynamics model.
d) Dynamic analysis of the structure for each wind direction and velocity for obtaining the effective wind loads as a function of time on each floor. The wind loads are obtained as the sum of the inertial and aerodynamic forces applied at the centre-of-mass of each floor.
e) Determination of ESWL. A force is applied at the centre-of-mass of each floor in the two main directions of the building and in torsion.

The ESWL method has been used as a framework for incorporating complex phenomena in the analysis of tall buildings. An example of this is the research carried out by Huang and Chen [7] for evaluating the influence of higher modes of vibration in tall buildings, or the work done by Chan et al. [8] in which the aeroelastic response of buildings has been included in the calculations along with 3-D coupled modes. Chan et al. [9] also highlighted the importance of updating the wind loads in tall buildings while developing their structural design and optimisation of the lateral stiffness.

In the case of earthquakes, the forces that are generated due to the acceleration of the ground increase as a function of the mass of the oscillating body. Although tall buildings have high fundamental periods that usually place them in the constant displacement range of the acceleration spectrum, the Eurocode [10] has established a minimum acceleration which implies that as the height of a building increases, the horizontal seismic forces on it also grow because of the increase in the dynamic mass that must be considered.

In addition, seismic actions also produce torsional effects in buildings that need to be included in the structural design. Although the building has full symmetry of stiffness and masses, and the analysis of horizontal seismic forces do not characterise a torsional response, this effect must always be included. The analysis cannot consider possible variations in the stiffness and the distribution of masses, or a torsional component of the ground vibration. All these accidental torsional effects are considered in codes including an accidental eccentricity.

The project of a tall building structure must always be based on detailed performance-based designs. The precise evaluation of wind actions is usually based on the development of experimental wind tunnels or the computational fluid dynamics models. In the case of tall buildings, the high-frequency force balance method (HFFB) used in wind tunnels has several limitations. The method does not consider the influence of higher vibration modes correctly and some modifications must be made for non-linear modal shapes. In addition, the model must be completely rigid to obtain accurate results. In the case of tall buildings, it is difficult to achieve a completely rigid real model. Hence, it is more appropriate to use the high-frequency pressure integration (HFPI) method, published by Irwin and Kochanski [11], using which the time histories of the modal loads in each vibration mode can be obtained. This is also the case with computational models, in which tall buildings require complex fluid-structure analysis, either coupled or uncoupled, but considering the vortex shedding phenomenon and the dynamic response of the building. Wijesooriya et al. [12] proposed an analytical method to evaluate the structural response of tall buildings.

One of the problems in the analysis of seismic forces is the use of numerical methods for evaluating the effect of a seismic event in a tall building during its concept design stage when some of the structural features of the building could be unknown. Although there are simplified linear elastic meth-

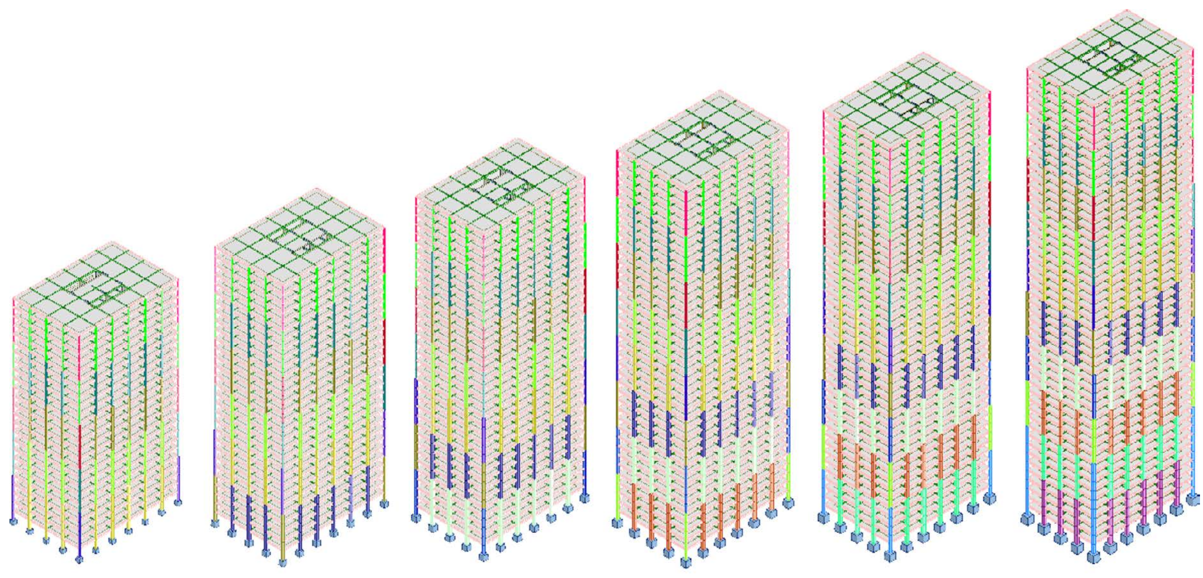


Figure 2. Geometries considered in the study. (a) height of 87.5 m, (b) height of 105 m, (c) height of 122.5 m, (d) height of 140 m, (e) height of 157.5 m, and (f) height of 175 m.

ods such as the lateral force method (LFM) for evaluating the seismic forces, they cannot be used in tall buildings due to the high fundamental periods (always > 2.0 s) that characterise these buildings and the influence of the vibration modes that are higher than the fundamental mode in the vibrational response of the structure.

Another widely used linear elastic method, the modal response spectrum analysis, is applicable to any type of building. However, it requires the development of numerical analysis models and thus cannot be considered as a preliminary evaluation method for the seismic action in the concept design stage of a building.

Alternatively, seismic structural design can also be performed using advanced nonlinear analysis methods such as the nonlinear static analysis (pushover). Liu et al. proposed the use of spectrum-based pushover analysis to evaluate the seismic demand in reinforced concrete shear walls in tall buildings [13], including the consideration of the coupling modes in the vibration analysis of the building. However, the evaluation of seismic forces in the initial design phases is not the objective of these advanced methods.

The inclusion of the horizontal actions that affect the building is a key aspect that must be incorporated at the beginning of the project. The concept design stage is one of the most important stages of the project during which the most appropriate structural typology is defined and the preliminary sizing of the elements is carried out.

Although it is a common practice in the structural design of tall buildings to compare the base shear due to wind and earthquake actions in the early stages of their designing, the objective of this study is to improve this common comparative analysis by developing a graphical and analytical method that aims to be generic enough to define a boundary in the structural behaviour. This makes it possible to differentiate whether the wind governs only the serviceability design or whether it will also influence the resistance design.

In this case, it is important to consider that it is not possible to define universal design rules that cover all design cases because of the large number of parameters that determine the horizontal forces. In addition, they depend on the specific aspects of each project, such as the shape of the building or the building site.

However, it is possible to assess whether the wind or seismic actions will govern the design and also calculate the magnitude of these actions at the initial stage of the project. This is possible as long as the building is sufficiently generic.

This study uses analytical methods to determine the base shears of a building for wind as well as seismic actions. The proposed analytical method defines the horizontal action that governs the design and calculates the magnitude of the forces for different building heights and different ground accelerations. A graphic analysis has been performed to evaluate the influence of these forces on the structural design of the building. This graphic-analytical methodology is complemented with a sensitivity study of the variables that influence the determination of the forces.

The present paper provides graphical and analytical tools to evaluate the relevance of wind and seismic actions in a tall building from its concept design stage to the application of the ESWL method.

2. NUMERICAL MODEL ANALYSIS

In this study, a generic tall building with a rectangular plan shape of 30 m × 45 m has been used for the analysis. The building is the Commonwealth Advisory Aeronautical Research Council (CAARC) building [14], which has been modified to have a different number of storeys. The number of storeys was between 25 and 50 floors, with the total building height between 87.5 m

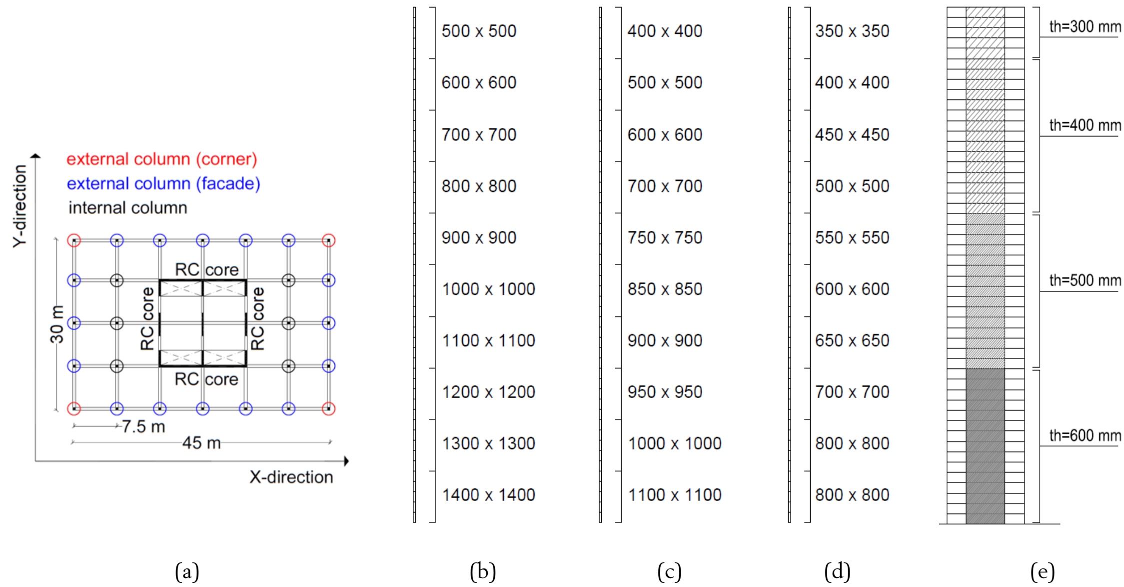


Figure 3. Schematic structural design of the case study in a 50-storey building. (a) Schematic plan view with the concept design of the structure. (b) Internal column size, (c) external column (façade) size, (d) external column (corner) size, (e) central core wall thickness.

and 175 m and geometric slenderness between 2.92 and 5.83 (Figure 2).

The typology is a concrete structure with a central core and frames composed of columns and downstand beams. Figure 3a includes a plan view of the case study with the schematic design of the main structural elements (columns, beams, internal core, and stairs and lift openings). Sizing of the main structural elements are defined considering Service Limit States and Ultimate Limit States criteria. The dimension of columns and thickness of the core walls are defined in Figures 3b to 3e.

The properties of the building, such as the mass per unit volume or frequency, were determined in the study for each case, based on the structural typology considered.

This comparative study has been carried out for locations with low and moderate seismicities, which correspond to the acceleration values ranging between 0.06g and 0.24g. For buildings with these geometric characteristics and range of ground accelerations, the defined structural typology has been characterised by an inelastic response. The details of the data considered in this case study are listed in Table 1.

TABLE 1.
Fundamental parameters considered in the initial case study.

Parameter	Considered value
Main typology	Central concrete core (15 m × 15 m)
Horizontal typology	Beams and solid slabs (30 m × 45 m)
Number of storeys	25 to 50 (87.5 m to 175 m)
Storey height	3.5 m
Distance between columns	7.5 m
Concrete strength for core/columns	C40/50
Concrete strength for slabs/beams	C30/37
Beams (section)	300 mm × 500 mm
Self-weight slab	5 kN/m² (th: 200 mm)
Other dead loads	2 kN/m²
Live loads (private offices, with the repercussion of stairs)	2.3 kN/m² (30% of the total value)

2.1. Modal analysis of tall buildings

The dynamic behaviour of the building was analysed via the modal analysis using the finite element models, including the three-dimensional response of the structure in an elastic linear regime [15]. In the case of wind action in tall buildings, Feng and Chen [16] developed a method of evaluating the effect of using non-linear analysis methods in the design of the structure. Thus, it was determined that the influence on the along-wind response is not relevant, whereas there would be advantages in the across-wind response. The bending stiffnesses of the slabs and beams were considered in the evaluation of the modal analysis of the structure.

The horizontal actions were evaluated in each of the two main directions of the building and were subsequently combined. It is recommended to avoid the dynamic response of a building characterised by three-dimensional (3-D) coupled modes [17]. This effect implies that each mode would be characterised by simultaneously having two translational movements and one rotational movement. The design of structures with different stiffnesses in the two main translational directions avoids 3-D coupled modes. The case study presented in this study consists of the first two modes of oscillation separated by more than 10% of the period, thus avoiding 3-D coupled modes. If 3-D coupled modes are not avoided in the design phase, it would mean a significant increase in the horizontal actions.

The dynamic behaviour of a building is characterised by Eq. (1). This movement can be caused either by a dynamic excitation force or by a movement of the ground where the building is founded. The first case corresponds to the wind forces applied to the building, producing its dynamic response, whereas the second case corresponds to the seismic action.

$$M \cdot \ddot{x}(t) + C \cdot \dot{x}(t) + \mathrm{K} \cdot x(t) = f(t) \tag{1}$$

where M is the mass matrix of the building, C is the structural damping, K is the stiffness matrix of the structure [18], x

TABLE 2.
Modal analysis: Structural translational vibration periods of tall buildings in the X-direction (seismic case).

Building height (m)	Mode 1 Period (s)	Mode 1 Mass (%)	Mode 2 Period (s)	Mode 2 Mass (%)	Mode 3 Period (s)	Mode 3 Mass (%)	Mode 4 Period (s)	Mode 4 Mass (%)
87.5	2.00	62.92	0.46	20.00	0.21	7.26	0.14	3.46
105.0	2.64	62.65	0.60	19.63	0.27	7.29	0.17	3.58
122.5	3.39	62.70	0.77	19.13	0.34	7.25	0.20	3.72
140.0	4.09	61.78	0.93	18.78	0.40	7.32	0.24	3.83
157.5	4.87	61.68	1.12	18.64	0.48	7.28	0.28	3.83
175.0	5.72	61.78	1.32	18.37	0.57	7.15	0.33	3.86

TABLE 3.
Modal analysis: Structural translational vibration periods of tall buildings in the Y-direction (seismic case).

Building height (m)	Mode 1 Period (s)	Mode 1 Mass (%)	Mode 2 Period (s)	Mode 2 Mass (%)	Mode 3 Period (s)	Mode 3 Mass (%)	Mode 4 Period (s)	Mode 4 Mass (%)
87.5	2.77	70.37	0.86	13.94	0.44	4.94	0.28	2.69
105.0	3.27	70.23	1.01	14.64	0.53	4.72	0.34	2.61
122.5	4.02	69.83	1.23	15.23	0.64	4.61	0.42	2.53
140.0	4.74	68.75	1.44	15.75	0.75	4.69	0.49	2.56
157.5	5.53	68.16	1.67	16.28	0.86	4.8	0.57	2.54
175.0	6.38	67.70	1.91	16.71	0.98	4.92	0.65	2.47

is the displacement, t is time, and f are the dynamic external actions.

A modal analysis was developed for characterising the dynamic response of the building. The general equation of motion (Eq. (1)) is solved. Using this method, which is exclusively applicable to linear problems, the vibration modes were obtained, and a frequency value was associated with each vibration mode. Equation (2) is considered to obtain the modal shape, being ω_n the natural frequency of the system and ϕ_n the natural mode shapes of vibration.

$$(K - \omega_n \cdot M)\, \phi_n = 0 \qquad (2)$$

In addition to the mass of the building, M, the second relevant parameter applied in the modal analysis is the building stiffness, K. In tall buildings, geometry and typology are the most important parameters that define their stiffness. However, the type and magnitude of seismic and wind actions produce different levels of cracking in the structural elements.

In a seismic event, the cracking of reinforced concrete elements can significantly reduce the stiffness of the structure and increase the fundamental periods of the building. Thus, in the present study, reduced inertia of the structural elements equivalent to half of their inertia in a seismic event [10] was taken as a reference.

The modal analysis of the case study was developed using 3-D computational models employing the Autodesk Robot Structural Analysis software. In the modal response spectrum analysis used subsequently to determine seismic loads it is necessary to consider all the vibration modes contributing significantly to the global response. According to [10], this requirement is satisfied when the effective modal masses for the modes taken into account reaches a minimum value of 90% of the total mass of the structure. In the case study it was necessary to consider the first four modes for mobilising this 90% of the dynamic mass in each of the two translation directions. Tables 2 and 3 summarizes the most relevant results obtained in the modal analysis for the first 4 modes. In these Tables, the column "Mass" shows the mass of the building that is mobilized in each vibration mode.

Figure 4 shows the modal analysis in the Y-direction for the 175 m tall building, performed on the basis of the finite element models and considering four translational vibration modes.

Although the level of cracking in the structure will be much higher in an earthquake event than in the case of wind events, it is important to consider that wind actions also produce cracking in the concrete core, which implies an important reduction in its stiffness. If the building has not been subjected to an earthquake, the stiffness that should be considered for evaluating the wind actions is as explained below. If an earthquake occurred, the stiffness to be considered would be the same in the case of seismic as well as wind actions.

Specific structural models were developed to analyse cracking under the wind action. The along-wind and across-wind loads were considered as a function of the height of the building [19]. Cracking is not relevant in buildings having up to 35 storeys. However, the concrete core exhibits cracking in buildings having 35 storeys or more (see Figure 5). In the tallest building (175 m), this stiffness reduction occurs in the lower one-third part of the core.

Table 4 shows the fundamental periods of the buildings when they are under the wind action (if an earthquake did not occur). In this case, a stiffness value that is half of the total iner-

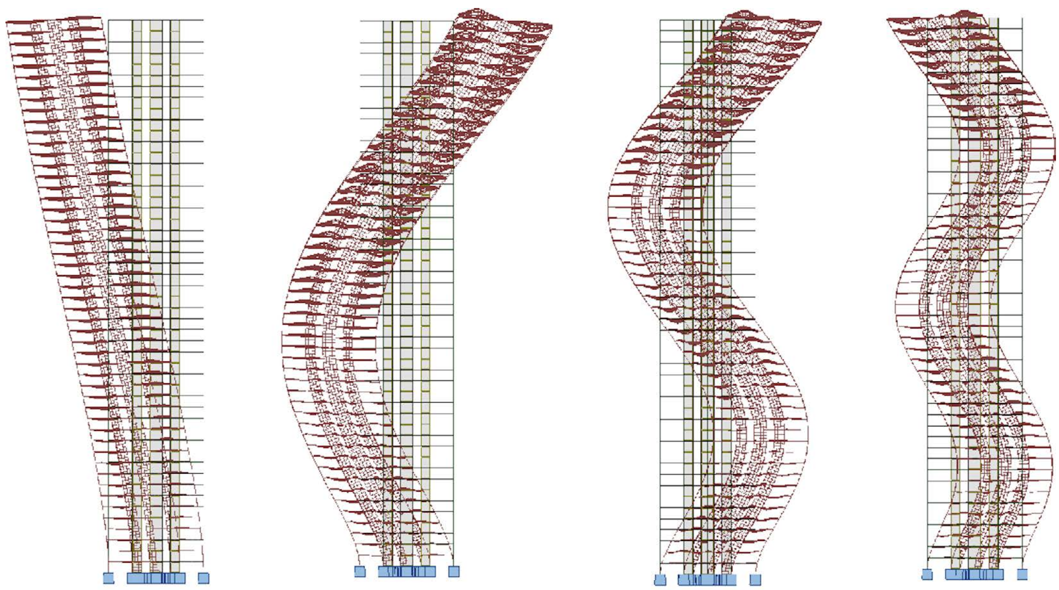


Figure 4. Modal analysis of a 175 m high building with the first 4 translational modes in the Y-direction. (a) 1st mode, with a period of 6.38 s, (b) 2nd mode, with a period of 1.91 s, (c) 3rd mode, with a period of 0.98 s, and (d) 4th mode, with a period of 0.65 s.

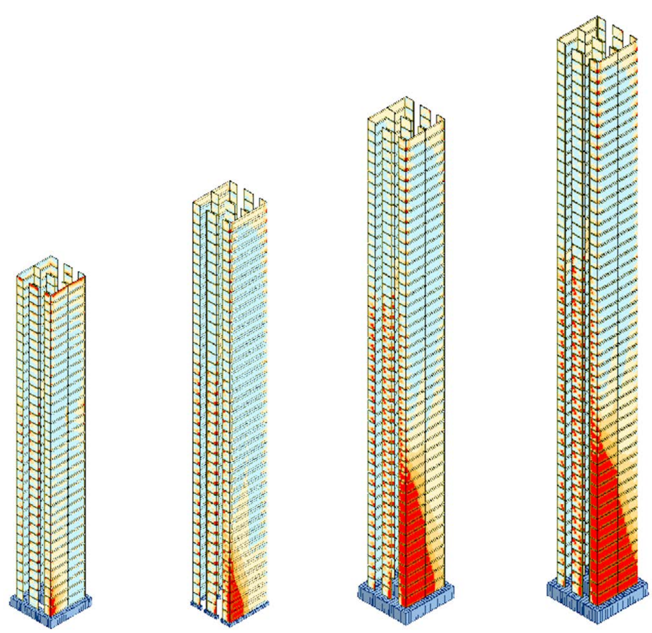


Figure 5. Qualitative stress distribution and core cracking due to wind action for buildings of different heights. (a) height of 122.5 m, (b) height of 140 m, (c) height of 157.5 m, (d) height of 175 m. The cracked area is shown in red colour.

tia in the lowest part of the core was taken for the 40, 45, and 50-storey buildings. It can be observed that the periods in the wind and seismic cases are similar. In other words, inertia reduction due to cracking is applied in the lower third of the concrete core or in the entire building. This is a consequence of the structural typology and the global stiffness that the core provides in relation to the stiffness of the facade columns and slabs.

The structural damping ratio considered for the seismic analysis was 5% [18,20]. This ratio of 5% is a standard value that was considered to obtain the corresponding spectral acceleration. This universal value of damping ratio is associated in codes with the elastic response spectra [28]. Nonlinear behaviour was included through the behaviour factor q in the present study. In the case of wind actions, a damping ratio of 1.6% was considered [21,22].

2.2. Wind action

In a tall building, it is fundamental to consider the simultaneity of the along-wind and across-wind vibration loads [23,24] along with the torsional effect, which is relevant in tall and flexible buildings. In the case of a tall building, the across-wind

TABLE 4.
Modal analysis: Structural periods of tall buildings (Wind cases).

Building height (m)	X-translation T (s)	Dif. (%)	X-translation T (s)	Dif. (%)	Cracked core (m)
87.5	1.83	–8.5	2.36	–14.8	-
105.0	2.40	–9.1	2.82	–13.8	-
122.5	3.03	–10.6	3.46	–13.9	-
140.0	4.00	–2.2	4.37	–7.8	0.16H
157.5	4.85	–0.4	5.25	–5.1	0.25H
175.0	5.70	–0.3	6.13	–3.9	0.35H

loads can take values much higher than the along-wind loads. This is because when wind flows around a tall building, oscillatory flow and vortex shedding occurs. In addition, the frequency coupling can occur when the building is flexible and the flow shedding tends to reach a frequency that is near the fundamental frequency of the tower. When both frequencies reach the same value, resonance occurs and the across-wind loads would clearly affect the design of the structure [25].

Hence, the along-wind loads as well as the across-wind vibrations, occurring because of vortex shedding, were analysed in this study. The analysis method included in the Japanese code AIJ [19] was applied, since the European codes [26] only evaluate the across-wind actions in the worst-case scenario, i.e., when frequency coupling occurs. The consistency between the results of the longitudinal wind actions obtained with the European and Japanese regulations was studied by Muñoz et al. [27].

For determining the wind action, a terrain category II was considered, which corresponds to the suburban development. The basic wind velocity at 10 m height and averaged over 10 min was 29 m/s.

The longitudinal and the transversal actions were determined using an analytical method based on the criteria established in the AIJ Recommendations for Loads on Buildings (AIJ-RLB) [19]. Although the transversal wind action is relevant when the building slenderness, λ_g, as defined in Eq. (3), exceeds the value of 3, its incidence at the proposed heights was shown in order to analyse its effect on each building.

$$\lambda_g = \frac{H}{\sqrt{B \cdot D}} \geq 3 \tag{3}$$

Where H the height of the building, B the dimension of the building perpendicular to wind direction, and D the dimension of the building parallel to wind direction.

The along-wind loads were determined as follows (Eq. (4)):

$$W_D = q_H \cdot C_D \cdot A \cdot G_D \tag{4}$$

where q_H is the design velocity pressure in the top part of the building, C_D is the drag coefficient of the building (aerodynamic factor), A is the projected area perpendicular to the wind direction, and G_D is the gust effect factor given by Eq. (5) below.

$$G_D = 1 + g_G \frac{C'_g}{C_g} \sqrt{(1+\phi_D^2 \cdot R_D)} \tag{5}$$

Where g_G is the peak factor, C_g' and C_g are the fluctuating and mean coefficients for along-wind overturning moment, ϕ_D is the correction factor depending on mode shape and R_D is the resonance factor.

Similarly, the wind loads caused by the across-wind vibration was determined using Eq. (6) given below, where g_L is the transversal peak factor, ϕ_L is the correction coefficient for the vibration mode, R_L is the resonance factor, and C_L' is a parameter that depends on the plan dimensions.

$$W_L = 3 + q_H \cdot C_L' \cdot A \frac{z}{H} g_L \sqrt{(1+\phi_L^2 \cdot R_L)} \tag{6}$$

Once the along-wind and across-wind vibration loads are determined, selecting the appropriate combination of the wind effects in both directions is the main issue. The simultaneous effects of both these actions must be considered.

In buildings with a slenderness ratio greater than 3, the wind action was calculated using Eqs. (7), (8), and (9), as specified in AIJ-RLB [19], considering the longitudinal gust factor G_D, which characterises each height and the building flexibility. Equation (9) corresponds to the predominant effect of the torsional moment, W_T. The correlation between the across-wind vibration and torsional vibration was considered using the coefficient k.

$$Wind_1 = W_D + 0.4 \cdot W_L + 0.4 \cdot W_T \tag{7}$$

$$Wind_2 = \left[(0.4 + {}^{0.6}/_{G_D}) \cdot W_D \right] + W_L + \mathrm{k} \cdot W_T \tag{8}$$

$$Wind_3 = \left[(0.4 + {}^{0.6}/_{G_D}) \cdot W_D \right] + \mathrm{k} \cdot W_L + W_T \tag{9}$$

2.3. Seismic action

The first key aspect in determining the seismic action is the situation of the building. Each place has its own specific seismic risks and a soil stiffness that can either be favourable or unfavourable for the construction of tall buildings. Construction difficulties usually limit the building situation to the specific soil characteristics. It can be stated that these buildings are mainly founded on medium quality soils having a certain degree of stiffness and bearing capacity, which implies them being supported in ground types B and C. Thus, the analysis in this study was first developed for ground type B and extended to ground type C. Ground type A was excluded because of its singularity, which implies a ground that has high-quality rock soil that is favourable for tower construction.

Provided that this study must allow the designer to know the main horizontal action in a specific situation depending on the building characteristics, a general seismic hazard was defined from a probabilistic point of view, considering that the soil acceleration interval has been limited to low and moderate accelerations for a return period of 475 y, i.e. for a probability of exceedance of 10% in 50 y. For high soil accelerations, a specific detailed analysis will be necessary from the first steps of the project. Considering this general approach, the study did not include risk maps associated with the structural collapse.

With the aim of developing the analysis in a linear elastic regime and not requiring an nonlinear analysis, the determination of the forces was based on the modal response spectrum analysis, applying a behaviour factor, q. The analysed typology, including a central concrete core and rigid frames made of downstand beams and columns with thin slabs, allowed for the appropriate energy dissipation, and in a seismic event, the damage would be concentrated in the bottom part of the core. From this consideration, a behaviour factor of 2 was applied in the study on the safety side [28].

It is also important to highlight that the first oscillation mode in the analysed tall building mobilises between 60% and 70% of the dynamic mass and always has a period of over 2 s. In addition, only the 25-storey tall building oscillating in the most rigid direction (X-direction) is not affected by the β factor.

Figure 6 shows the interval of the design spectrum considered on the basis of the analysed seismic direction for the first oscillation mode, for the maximum acceleration in the study.

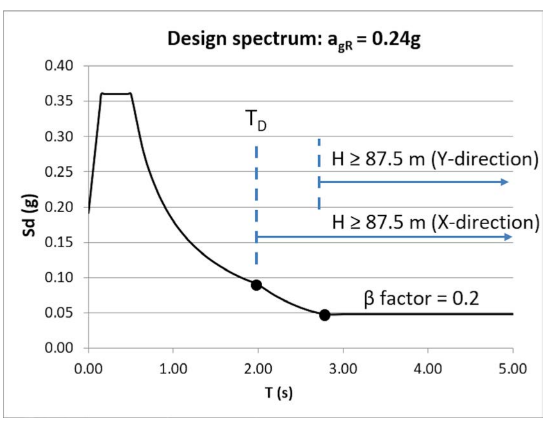


Figure 6. Acceleration design spectrum for 0.24g acceleration and ground type B.

For calculating the base shear due to seismic action, an eccentricity, between the mass and the stiffness, of 5% of the floor dimension in each direction was considered. This eccentricity was applied in the modal analysis but not in the subsequent seismic analysis. Consequently, the condition of the total mobilised mass in the torsion modes of being null was avoided.

Once the base shears were calculated in both orthogonal building directions, the design seismic action was obtained from a combination of both directions as follows in Eq. (10):

$$V_k = \sqrt{(E_1^2 + 0.3 \cdot E_2^2)} \tag{10}$$

where V_k the shear in the base of the building, E_1 the seismic action in one orthogonal direction, and E_2 the seismic action in the other orthogonal direction of the building.

3. RESULTS

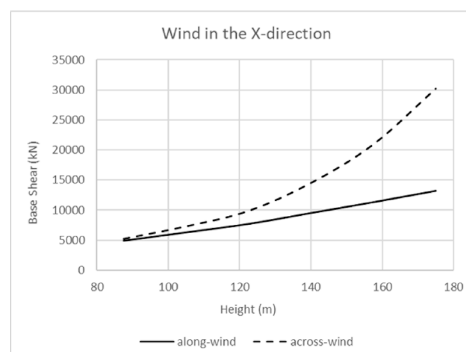
This section presents the results of the analysis of the wind and seismic action for different building heights for the analysed cases, obtained using the analytical methods that have been described in the previous section.

3.1. Wind action evaluation

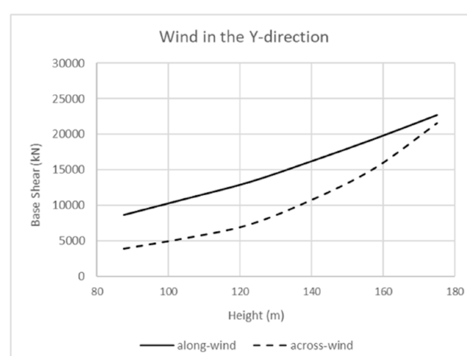
Table 5 shows the along-wind loads and loads produced by the across-wind vibration. The gust factor has been included for each height and wind direction. Partial factors were not applied.

The slenderer and the more flexible a building is, the nearer is the fundamental vibration frequency to the frequency with which the vortex shedding occurs, which produces the oscillatory forces in the transverse direction. This proximity between frequencies indicates the increase in the wind action that follows an exponential law.

The more flexible the building is in the direction perpendicular to the wind direction, the higher will be the increase in the global forces. Figure 7 shows the along-wind and across-wind vibration loads for the two wind directions from which the exponential behaviour of the transverse component (dashed line) can be observed.



(a)



(b)

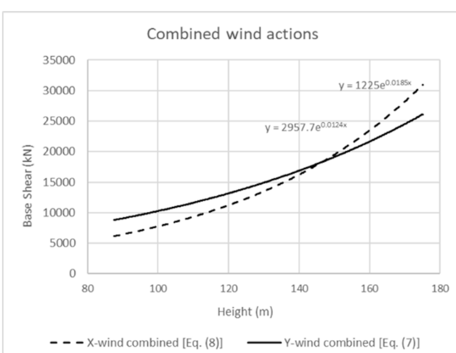
Figure 7. Along-wind and across-wind vibration loads as a function of the height of the building. (a): Wind in the X-direction. (b): Wind in the Y-direction.

When the along-wind and across-wind components were analysed together for the studied cases, the global wind actions could be approximated by exponential laws. Figure 8 shows the most

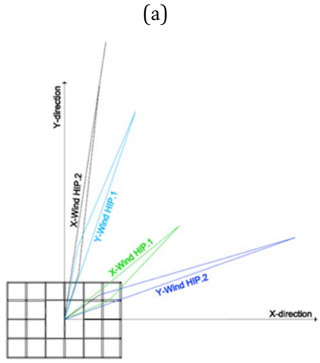
TABLE 5.
Along-wind and across-wind vibration loads.

Building height (m)	X-wind direction GD	X-wind direction Along (kN)	X-wind direction Across (kN)	Y-wind direction GD	Y-wind direction Along (kN)	Y-wind direction Across (kN)
87.5	1.85	4943	5174	1.87	8599	3902
105.0	1.87	6329	7266	1.87	10911	5405
122.5	1.91	7731	9841	1.90	13236	7283
140.0	1.93	9536	14510	1.93	16217	10773
157.5	2.01	11332	20964	1.99	19369	15206
175.0	2.04	13208	30271	2.04	22734	21560

unfavourable combination for each wind direction, obtained using Eqs. (7) and (8) respectively. It is important to highlight that for both wind directions, the worst combination produces a base shear with the main direction next to the Y-axis. When the wind is parallel to the X-direction, the across-wind vibration load is relevant due to the plan proportion and smaller rigidity of the building in the Y-direction. However, when the wind is parallel to the Y-direction, the along-wind load is unfavourable because of its larger facade surface, and not because of the forces generated due to vortex shedding, except for the 45, and 50 storey buildings, in which the across-wind vibration loads predominate.



(a)



(b)

Figure 8. Unfavourable along-wind and across-wind vibration load combinations. (a): Combined wind actions depending on the height of the building. (b): Direction and relative magnitude of wind load combinations

The increase in the global wind actions as a function of the height of the building can be expressed based on the exponential laws with relative errors of less than 4.7% for the case study. The wind base shear due to the X-direction wind was calculated using Eq. (11) and it was considered as the main component of the force due to the across-wind vibration load (Wind2 – Eq. (8)). Similarly, the wind base shear due to the Y-direction wind was calculated using Eq. (12) and the main component of the force was the along-wind action (Wind1 – Eq. (7)).

$$F_{w\text{-}X:29} = 1225 \cdot e^{0.0185H} \tag{11}$$

$$F_{w\text{-}Y:29} = 2958 \cdot e^{0.0124H} \tag{12}$$

3.2. Seismic action evaluation

Table 6 lists the base shear resulting due to the different values of the ground accelerations (low and moderate) for the studied building heights, combined in the two orthogonal directions with the X-direction as the predominant direction (Eq. (10)). Similarly, Table 7 lists the base shears when the Y-direction predominates. The vector angle that results from the combination of the seismic cases in both the orthogonal directions has also been given in this table.

Figure 9 shows the combined seismic base shears for a 0.24g ground acceleration. The vectorial representation of these seismic forces includes not only the value of the force but also their direction (for positive X and Y axes).

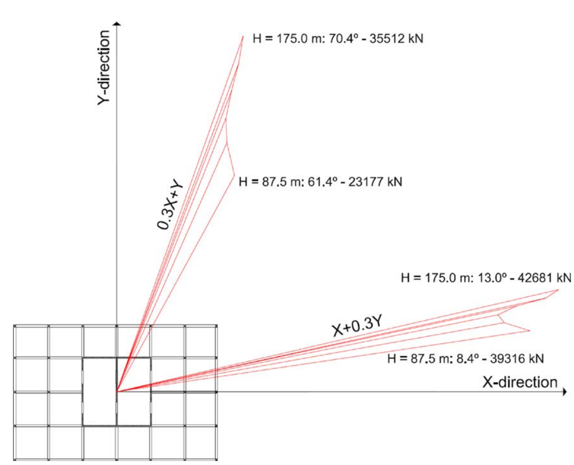


Figure 9. Seismic base shears in the case study (global force and direction) for a 0.24g acceleration.

TABLE 6.
Seismic base shears (kN) with load combination [100 %X + 30 %Y].

height	Ground acceleration										α_1
(m)	0.06g	0.08g	0.10g	0.12g	0.14g	0.16g	0.18g	0.20g	0.22g	0.24g	(°)
87.5	9678	12903	16131	19357	22583	25807	29036	32261	35527	39316	8.4
105.0	9259	12346	15433	18519	21606	24692	27779	30865	33953	37038	10.1
122.5	9161	12213	15266	18320	21373	24426	27481	30534	33587	36641	11.4
140.0	9620	12828	16034	19241	22456	25662	28870	32069	35275	38483	12.1
157.5	10338	13785	17231	20677	24131	27578	31026	34472	37920	41367	12.3
175.0	10670	14226	17783	21339	24897	28463	32010	35567	39124	42681	13.0

TABLE 7.
Seismic base shears (kN) with load combination [30 %X + 100 %Y].

height	Ground acceleration										α_1
(m)	0.06g	0.08g	0.10g	0.12g	0.14g	0.16g	0.18g	0.20g	0.22g	0.24g	(°)
87.5	5671	7563	9451	11345	13233	15125	17014	18905	20819	23177	61.4
105.0	6403	8537	10671	12805	14940	17074	19208	21342	23476	25611	66.1
122.5	6924	9233	11539	13848	16157	18465	20772	23080	25388	27696	68.2
140.0	7588	10118	12648	15177	17712	20242	22772	25296	27826	30355	69.1
157.5	8238	10985	13730	16476	19229	21976	24722	27470	30217	32964	69.6
175.0	8878	11836	14796	17756	20714	23683	26634	29593	32553	35512	70.4

3.3. *Sensitivity analysis*

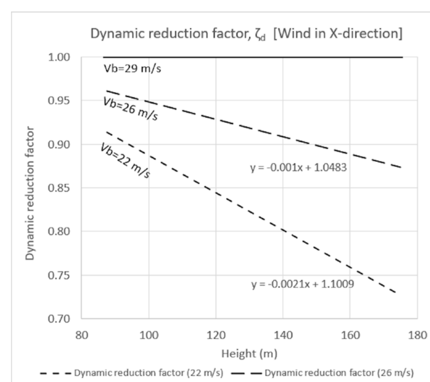
This section presents the results of the numerical evaluation of several parameters that affect the base shear. With regards to the wind actions, the effect of the mean wind velocity reduction from 29 m/s to 22 m/s was analysed together with the consequences of the variation in the damping of the structural system, a parameter that is difficult to set at the beginning of the project. With regards to the seismic actions, the influence of a less stiff soil (ground type C) was analysed.

3.3.1. *Wind velocity*

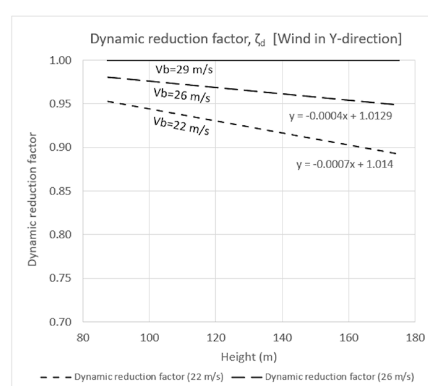
This subsection discusses the influence of the variation in the mean wind velocity on the dynamic response of tall buildings.

In a rigid building, an increase in the mean wind velocity produces an increase in the wind pressure, which is related to the square relation between the wind velocities. However, in a flexible building, it is not possible to define a direct square relation between different wind velocities owing to the dynamic response and the vortex shedding phenomenon.

The analysis of the behaviour of tall buildings as a function of different mean wind velocities allows an accurate distinction between the cases in which the wind actions predominate over the seismic actions. The addition of the dynamic wind behaviour of the building to the vortex shedding phenomenon implies that the taller the building and the higher its transversal flexibility, the bigger is the difference in the wind actions with respect to the exact square relation of the peak wind pressure. As shown in Figure 10, a family of straight lines represented with negative slopes is more pronounced whereas the dynamic component has less influence on the behaviour of the building with respect to the quadratic relation of the wind velocities with a dynamic reduction factor, ζ_d, of 1.0 for the mean wind velocity considered initially.

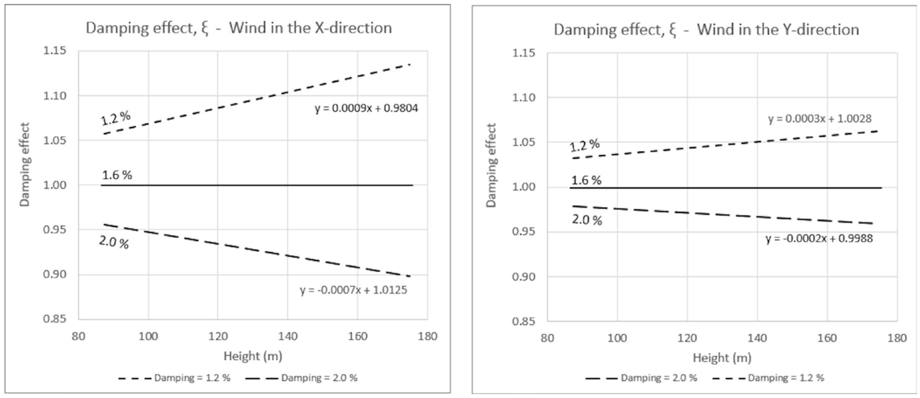


(a)



(b)

Figure 10. Influence of the mean wind velocity on the dynamic component of the global wind action in the case study. Definition of a dynamic reduction factor ζ_d depending on mean wind velocity. (a): Dynamic reduction factor considering wind in X-direction. (b): Dynamic reduction factor considering wind in Y-direction.



(a) (b)

Figure 11. Wind base shear variation coefficient, depending on the amount of damping, in the case study. (a): Damping effect considering wind in X-direction. (b): Damping effect considering wind in Y-direction.

For the X as well as Y wind directions, a reduction factor of the wind base shear was applied for the mean wind velocity ranging between 22 and 29 m/s. For the mean wind velocity, the dynamic reduction factor ζ_d must be considered to be equal to 1. For mean wind velocities lower than 29 m/s, this coefficient, which is lower than 1, represents the wind action as a function of the wind velocity, as given by Eqs. (13) and (14) respectively.

$$F_{w\text{-}X} = 1225 \cdot e^{0.0185H} \cdot \left(V_b/29\right)^2 \cdot \zeta_{d\_x} \tag{13}$$

$$F_{w\text{-}Y} = 2958 \cdot e^{\,0.0124H} \cdot \left(V_b/29\right)^2 \cdot \zeta_{d\_y} \tag{14}$$

with $\zeta_{d\_x} = [(0.00027 \cdot V_b \cdot \mathrm{H} - 0.010373 \cdot V_b - 0.0802 \cdot H + 1.308)]$ and

$$\zeta_{dy} = [(0.000075 \cdot V_b \cdot \mathrm{H} + 0.000437 \cdot V_b - 0.00235 \cdot H + 1.0)]$$

These dynamic reduction factors modify Eqs. (11) and (12) to consider a reduction in the vibration when the wind velocity is lower. These factors have a relative error lower than 2% for all cases included in the study except for the tallest building in the Y-direction. In this case, the building is very slender and flexible and the fundamental frequency of the building (0.16 Hz) is close to the vortex shedding frequency for a wind velocity of 29 m/s. This resonance effect produces an equivalent wind load amplification that is not produced at lower wind velocities [26].

3.3.2. *Structural damping*

One of the main aspects in the calculation of wind actions in tall buildings is the structural damping. This study only considered damping from the structure and excluded the damping from finishes and other construction elements. Aerodynamic damping was also discarded, provided that its value was null or even negative in the analysis of the across-wind actions. Although the study considered damping of 1.6% of the critical damping as the reference value, it is not possible to accurately determine the final value for a real building during its design stage [29].

Thus, it was important to determine the influence of the variation of structural damping on the dynamic component of wind actions. The analysis interval range was from 1.2% to 2.0% of the critical damping.

Figure 11 shows the influence of structural damping on the wind load characterisation. It can be observed that a higher variability is obtained because of the building oscillation due to the across-wind actions in the X-direction. This indicates that for the tallest building analysed in this study, the increase in the base shear is nearly 15% when the damping is reduced to 1.2%.

3.3.3. *Soil stiffness*

This study assumed that tall buildings will usually be constructed in stiff soils. Excluding the rocky grounds, the type B ground was taken. Although it is unusual to build this type of building in low compacity grounds, it is sometimes necessary to find them in type C grounds. It has previously been shown that only in the stiffest direction of the case study (X-direction) and the shortest buildings (87.5 m and 105 m), the first oscillation mode is in the spectre zone that corresponds to constant displacement, in the interval between Td and the part affected by the β factor. For tall buildings in the X-direction and all heights analysed in the Y direction, the first oscillation mode is in the constant acceleration zone that is affected by the β factor (Figure 12). This consideration implies that founding a tall building in a type B or type C ground has no significant effect on the maximum base shear.

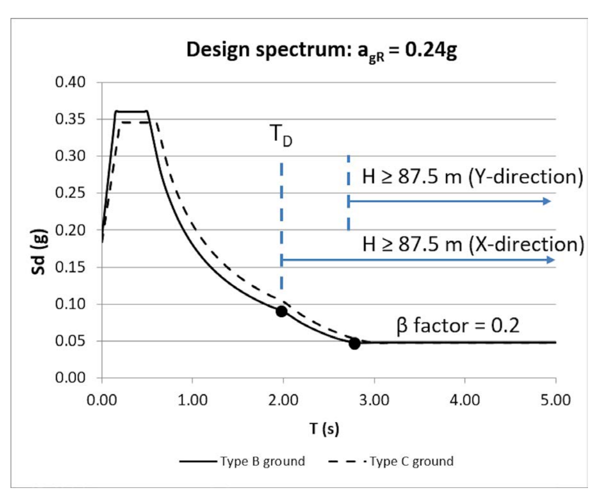


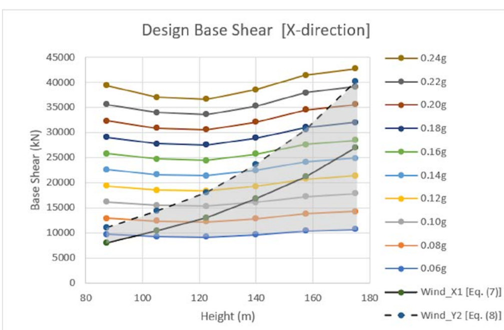
Figure 12. Acceleration design spectrum showing the comparison for type B and C grounds for the case study buildings.

4.
DISCUSSION

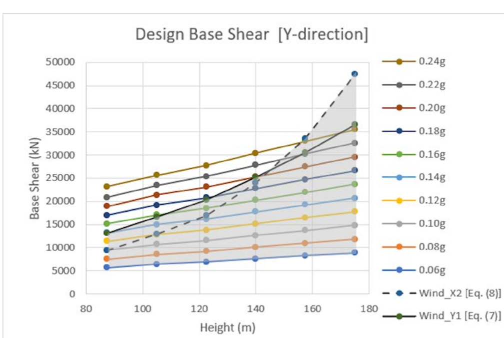
It is fundamental to be able to compare the different influences that wind and seism have on the structural design of tall buildings. In the case of the horizontal actions in the analysed buildings, important differences are observed between the two orthogonal directions.

4.1. Comparison of the forces

The seismic and wind actions are graphically compared in Figure 13. The seismic actions are shown for different ground accelerations and building heights with parallel lines, whereas the wind actions are represented by two exponential laws with the worst areas hatched, which show the maximum wind action in the X-direction (the across-wind vibration load component always predominates in terms of the maximum wind force characterisation) and in the Y-direction (the along-wind load component predominates for up to 40 storey buildings).



(a)



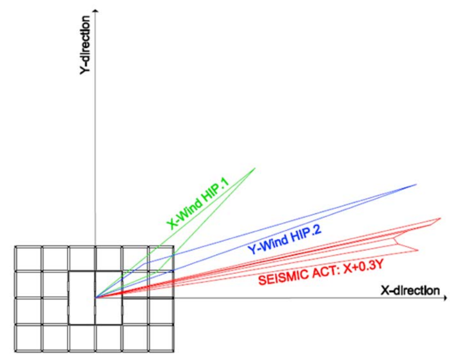
(b)

Figure 13. Wind actions (the hatched areas) and seismic actions overlapped for the cases where, (a): X-direction dynamic action predominates, and where (b): Y-direction dynamic action predominates.

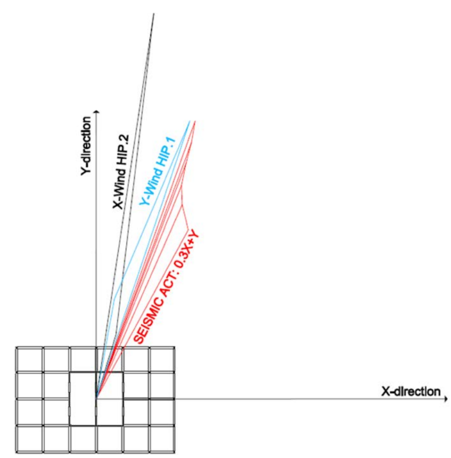
When the X-direction (Fig. 13a) corresponding to the larger rigidity is analysed, the seismic actions are predominant over the wind actions in 57% of the analysed cases. This result is obtained even though the seismic actions vary slightly with the building height and the wind action exhibits exponential behaviour.

In contrast, when the Y-direction is analysed (Fig. 13b), the conclusions are different. In this case, the dynamic response of the building produced by the across-wind vibration loads, added to the larger facade surface exposed to wind, makes the global wind action acquire a greater relevance. In this case, the wind action is predominant in 75% of the cases. Even for those buildings which are lower (25 storeys), the horizontal wind actions are unfavourable until the ground acceleration reaches a value of 0.14g. With the increasing height of the building, from 157.5 m, the wind action is observed to govern all the evaluated cases.

Figure 14 compares the wind and seismic actions for the two main vectorial compositions (X-and Y-directions), taking the maximum moderate ground acceleration of 0.24g as the reference. It shows a clear predominance of the seismic action in the X-direction, whereas, in the Y-direction, the wind action is slightly more than the seismic action.



(a)



(b)

Figure 14. Graphic analysis of the wind and seismic actions at the base of the building (0.24g). (a) X-direction dynamic action predominates, and (b) Y-direction dynamic action predominates.

4.2. Torsional moment at the base

Figure 15 shows that in buildings having between 25 and 35 storeys, the torsional moments due to the wind and seismic actions are remarkably similar. However, from the height of 122.5 m (35 storeys), the torsional moment due to the wind action acquires relevance and it is maximum in the X-direction of wind action. Not only does the rectangular proportion of the building increase the torsional vibration, but the loss of torsional rigidity with the height increase also amplifies the torsion because of the aeroelastic phenomenon. One of the physical phenomena that increases this torsional behaviour is the vortex shedding, characterised by its oscillatory nature.

This implies a temporal and spatial variability in the application of the resulting across-wind vibration loads. This effect increases the torsional dynamic oscillation, whereas the building is less rigid to torsion and with a lengthening proportion in the wind direction.

Hence, as the height of the building increases, it becomes necessary to include the rigid facades in the structural system to add the torsional rigidity, for example, the 'tube in tube' typology.

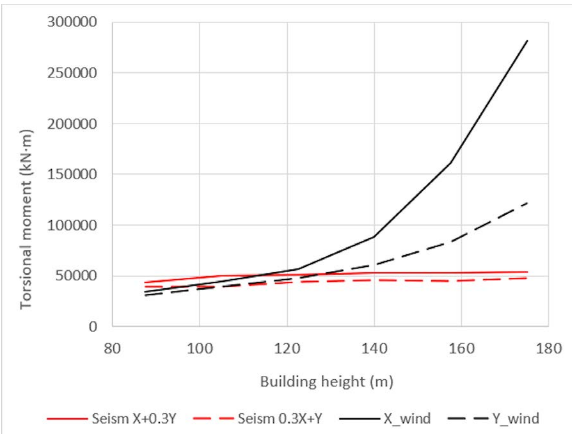


Figure 15. Comparison of the design torsional moments due to the wind and seismic actions as a function of the height of the building.

5. CONCLUSIONS

This paper presents an analytical-graphic method that allows stating the rule of horizontal wind actions in rectangular plan tall buildings (CAARC) during the concept design stage. The case study included a central reinforced concrete core and rigid frames with downstand beams and slabs. The design horizontal forces due to wind and seismic actions at the base of the building are compared. An evaluation is done of which horizontal action will govern the building design.

Exponential laws are defined for the wind actions, including the along-wind and across-wind vibration loads. These laws include the dynamic behaviour of the building on the obtained values.

For the case of seismic actions, the base shear is obtained for a low and moderate ground acceleration, which enables the evaluation of its influence in the two main directions. For both types of actions, their magnitude and the global force direction is analysed.

When the maximum resultant values are compared in groups depending on the resultant force directionality, it is observed that in the most rigid building direction, the seismic action is predominant in approximately half of the analysed cases. However, in the Y-direction, the seismic action has less relevance as compared to the wind action because of the lower stiffness of the buildings. This implies lower seismic action and an equivalent increase in the across-wind vibration action loads with an X-direction wind action.

The paper also presents the sensitivity analysis, in which the influence of the mean wind velocity and structural damping on the dynamic behaviour in different wind cases has been investigated. From this analysis, a clear influence of the dynamic response is observed for high velocities. Damping is also used for calculating the across-wind vibration with an X-direction wind. In terms of the seismic action, an almost null influence is observed on the buildings when the ground type is changed from B to C.

All the conclusions deduced in this research are inherently limited because they are based on a simple CAARC benchmark case. Furthermore, the effect of soil-structure interaction has been disregarded in the study.

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