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FOUR TALL BUIDINGS IN MADRID

STUDY OF THE WIND-INDUCED RESPONSE IN SERVICEABILITY LIMIT STATE

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SUMMARY

Limiting wind-induced building motion is one of the governing criteria in tall building design. Building motion should be limited, on one hand, to prevent damage in partitions, façade elements and interior finishes. On the other hand, human motion perception can induce concern regarding the structural integrity and cause nausea to the occupants.

Recently, four tall buildings have been erected at the former sports complex of the local football club Real Madrid in Madrid, Spain. As far as their lateral load design is concerned, the principal load has been the wind action since Madrid is located in a low seismicity zone. The global dimensions and geotechnical conditions of the buildings are very similar, however differences exist in the adopted lateral load resisting systems.

A study has been carried out on the wind-induced structural response in the serviceability limit state of the aforementioned four tall buildings. Finite element models have been developed to obtain, in combination with the recommendations laid down in Eurocode 1, the equivalent static wind load and the lateral displacements and accelerations.

1. INTRODUCTION

This chapter presents a short introduction on tall buildings and their historical and structural context. Chapter 2 contains the objective of this study and chapter 3 deals with the main approach and treats some general considerations. The architectural and structural properties, as well as the lateral load resisting characteristics are discussed in chapter 4. Chapter 5 provides the obtained results, whereas chapter 6 and 7 contain the acknowledgements and a list of consulted literature respectively.

1.1 HISTORICAL CONTEXT

From the first tall buildings built in the United States during the late 19th century, an accelerating increase of the presence of tall buildings in our cities is observed. The three following figures 1, 2 and 3 are taken from reference [3] and illustrate the trend among the hundred tallest buildings in the world with regard to height, region and function respectively.

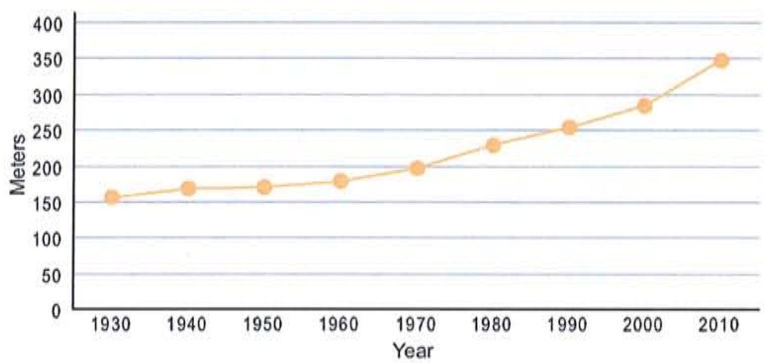

Figure 1: Mean height of hundred tallest buildings [3]

Figure 1 shows the mean height of the hundred tallest buildings in the world. Note that the mean height increases modestly until approximately 1950 whereafter a clear acceleration sets in. At present, we find ourselves in a period of truly spectacular growth in height as expressed, for instance, by the over 800 m tall Burj Dubai currently under construction in Dubai (United Arab Emirates).

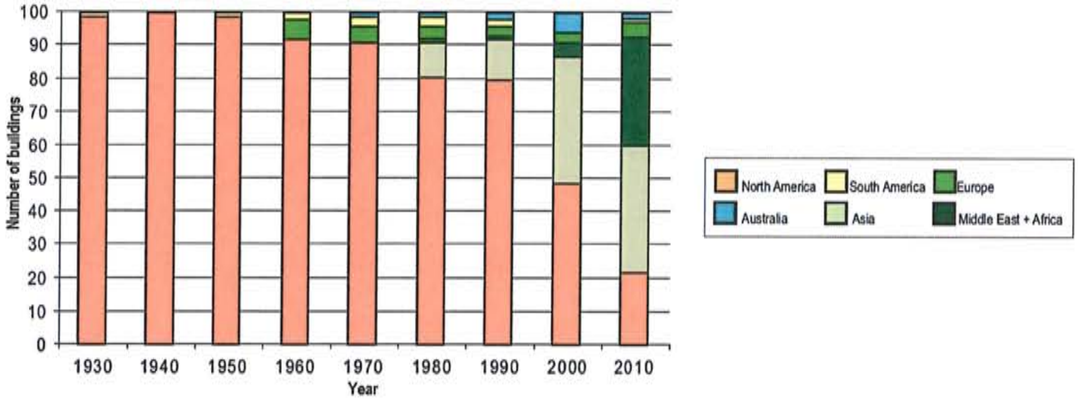

Figure 2: Hundred tallest buildings by region [3]

The above figure illustrates a shift of the centre of gravity in high-rise construction. North America, traditionally the leading region, is increasingly losing its dominant position as far as building height is concerned. By 2010 the vast majority of tallest skyscrapers will be located in Asia and the Middle East. Note the rather modest position of Europe represented by only three tall buildings: the Commerzbank Zentrale (259 m) in Frankfurt, the Triumph Palace (264 m) and the Naberezhnaya Tower C (268 m), being the latter two both located in Moscow. The four tall buildings in Madrid, analysed in this study, are comparable to the aforementioned buildings with heights roughly comprised between 220 m and 250 m.

Figure 3 presents the hundred tallest buildings by their function. Whereas until 2000 tall buildings were almost exclusively designed to provide office space, the trend towards mixed-use and residential buildings stands out in the present decade.

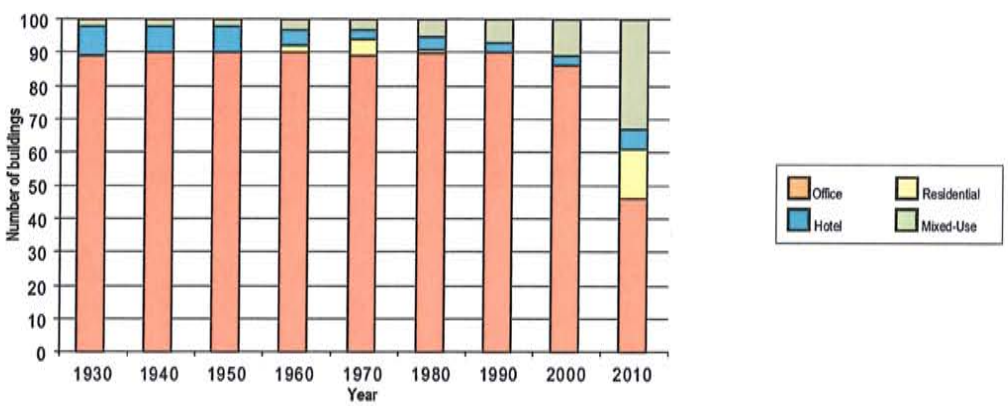

Figure 3: Hundred tallest buildings by function [3]

As far as the material of the structural systems is concerned, early high-rise building structures were predominantly constructed in steel. However, in the last few decades reinforced concrete and composite structural systems are gaining ground.

1.2 STRUCTURAL CONTEXT

It is known that the total gravity load acting on a building structure is roughly proportional to the building height, whereas the bending moment due to lateral loads (such as wind and earthquake action) is proportional to the squared height. Furthermore, lateral displacements of a bending-dominated structure depend on the height elevated to the fourth power. The foregoing demonstrates the increasing importance with height of lateral building motions and the problems associated with these motions.

For non-earthquake prone regions, limiting wind-induced building motions normally results governing. These motions are to be kept within acceptable limits as they can cause damage in partitions, façade elements and interior finishes, as well as adversely affect human comfort.

The trend in high-rise design is towards buildings with greater heights, built with light and high-strength materials. This leads to more flexible buildings with less mass, because of which the susceptibility of these structures to wind action will increase. As a consequence, motions and motion-induced problems in tall building design will also gain importance.

2. OBJECTIVE

The objective of this study is to analyse, according to the relevant Eurocodes, the along-wind structural behaviour of four tall buildings located at the former sports complex of Real Madrid in Madrid, Spain.

3. GENERAL CONSIDERATIONS

3.1 CALCULATION AND MECHANICAL REPRESENTATION

Complete finite element models have been developed of each high-rise building (as shown in figure 4, 5, 6 and 7) to obtain their vibration modes and frequencies, as well as to calculate the 50-year along-wind displacements. The human comfort is assessed, according to the recommendations of Eurocode 1 part 4 [4], in terms of the 5 and 10-year along-wind building acceleration at the top occupied floor. The across-wind structural response is not considered in this study, nor is the interference effect caused by nearby surrounding buildings.

Figure 4: Torre Caja Madrid

Figure 5: Torre Sacyr Vallehermoso

Figure 6: Torre de Cristal

Figure 7: Torre Espacio

The wind-induced displacement can be thought of as consisting of three parts: a static part due to the 10-minute averaged extreme wind velocity, a static part due to the wind turbulence and a dynamic response caused by resonance with the turbulent wind gusts.

The vibration frequency, on one hand, is needed to calculate the resonant response. On the other hand, it is the fundamental mode shape in combination with the mass distribution along the height that determines the excited building mass in the fundamental vibration mode. The accelerations due to along-wind turbulence are higher for lower fundamental frequencies and fundamental modal mass.

Vibration frequencies and mode shapes are obtained by means of a modal analysis applied to the finite element models. The quasi-permanent load combination, acting on the buildings in serviceability limit states, is provided in expression (1).

$$\sum_{j\geq 1} G_{k,j} + \sum_{i\geq 1} 0{,}3 \cdot Q_{k,i} \tag{1}$$

Geometric nonlinearity (P-delta effect) and soil-structure interaction are accounted for in the performed calculations. The materials of the structural members are assumed to behave elastically. Gross concrete section properties are adopted for concrete members predominantly subjected to bending moments, whereas the effective axial stiffness is computed for compression members. For elements subjected to important bending moments and compressive forces (for example cores and shear walls), it is verified that tensile stresses in the serviceability limit state do not exceed the concrete tensile resistance.

The modulus of elasticity of steel elements is assumed to be 210.000 MPa, while the secant modulus of elasticity for concrete members is calculated according to expression (2).

$$E = 9500 \cdot \sqrt[3]{f_{ck} + 8MPa} \tag{2}$$

where f_{ck} is the characteristic concrete compressive strength in MPa.

3.2 WIND ACTION

The calculation of the equivalent static wind load, according to Eurocode 1, as applied to every storey of the finite element models is shown in formula (3).

$$F_w = c_s \cdot c_d \cdot c_f \cdot q_p(z) \cdot A_{ref} \tag{3}$$

where F_w equivalent static force acting on an element or structure [N]

c_s size factor [-]

c_d dynamic factor [-]

c_f force coefficient [-]

q_p peak pressure [Pa]

A_{ref} reference area [m^2]

The above size factor accounts for the lack of full spatial correlation between the peak wind gusts acting on the building surface. The amplification of the wind-induced response due to resonance with the wind gusts is represented by the dynamic factor.

In general, and especially for irregular geometries, force coefficients of tall buildings are determined by means of wind tunnel studies.

The basic wind velocity, with a return period of 50 years, for Madrid is 26 m/s and the local terrain roughness corresponds to terrain category III. The resulting mean and peak wind pressure profile is provided in figure 8.

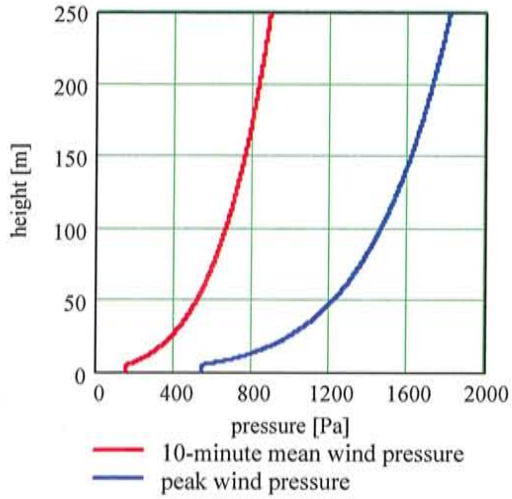

Figure 8: Wind pressure profile

The evaluation of human comfort is carried out for winds with a 5 and 10-year recurrence interval.

3.3 CRITERIA AND BUILDING CODES

Serviceability limit state evaluation of wind-induced structural behaviour is generally carried out in terms of lateral displacement and human motion perception. These aspects, as well as the adopted criteria, are dealt with hereafter.

3.3.1 LATERAL DISPLACEMENT

Excessive lateral building displacements can cause damage in façade elements, partitions and interior finishes (doors, elevators, etc.). To prevent damage to, or malfunctioning of, these non-structural elements, inter-storey drifts have to be limited.

No universally accepted criterion exists with regard to maximum lateral displacements in serviceability limit state. However, inter-storey displacements of up to H/400 (being H the storey height) are normally considered as being acceptable. In order to avoid the calculation of all inter-storey displacements, usually a global drift limit is adopted of H/500 (where H refers to the total building height). Note that the overall drift criterion can be more or less conservative depending on the shape of the actual displacement diagram, being the sum of the displacement due to bending, shear and the rotation of the foundation.

3.3.2 HUMAN COMFORT

The perception of oscillatory motion in tall building structures can cause serious comfort issues to occupants, such as concern about the quality and integrity of the structure and even nausea.

It is noted that no consensus has yet been reached among structural engineers with regard to human motion perception criteria.

Human motion perception varies considerably from person to person depending on:

- Frequency of oscillation.
- Physiological factors: body orientation, posture and movement.
- Psychological factors: expectancy of movement, comments by other occupants as well as audio and visual cues.

The importance of the abovementioned factors is subjected to much discussion. The frequency dependence of motion perception thresholds, for instance, is not universally recognised; reference [9] claims complete frequency independence. Furthermore, observations during real building vibrations show that audio and visual cues play an important role in comparison to the physical motion perception.

Below a brief summary is presented of some research on human motion perception, the discussion as whether to use the peak or root-mean-square accelerations and, thereafter, the human comfort criteria adopted in this study are presented.

Research

Traditionally, research has been focussed on the experienced body forces as a result of horizontal accelerations. Some studies suggest that human motion perception thresholds are better described by the rate of change of the acceleration (so-called *jerk*).

Proposed human motion perception thresholds have, historically, been based on the reactions of people subjected to so-called *"moving room"* experiments. This experiment consists of a closed, acoustically insulated, motion simulator generally excited by a uni-axial sinusoidal motion. Important early studies of this type were carried out by CHEN & ROBERTSON [1] and IRWIN [8]. In both studies a frequency dependence of perception thresholds was reported; the human motion perception decreases with decreasing frequency.

However, the above described *moving room* experiments do not correspond to the real situation of tall building occupants under natural wind excitation because of the following:

- Tall building motion due to along-wind buffeting is a random instead of a sinusoidal motion. The peak factor, being the relation between the peak and root-mean-square (rms) value of the acceleration, is not the same for sinusoidal and random motion. Therefore, applying perception thresholds from sinusoidal *moving room* experiments to random building vibration does not yield the same results in terms of rms or peak accelerations.
- Tall building motion is a spatial combination of two translational and a rotational motion instead of a uni-axial motion. In-plan rotation obviously increases the resultant lateral acceleration towards the building envelope. In addition, human beings are especially sensitive to a rotation of the horizon (visual cue).
- Psychological factors are not accounted for: people do not usually expect building vibration and the adverse effect of comments by other occupants is not considered.
- Audio and visual cues are not taken into account: the noise of the wind, cracking of the structure, the aforementioned effect of a rotating horizon, etc.

HANSEN et al. [5] were the first to provide subjective reactions to real tall building low-frequency motion. After a wind storm they performed a survey among occupants of two buildings and asked them with what recurrence they would accept such building movements. Furthermore, they asked to several prominent building owners: *"What percentage of the top one-third of the building can object to the sway motion every year without seriously affecting your renting program?"* The authors deduced from these results that 2% of the population could perceive the motion. This, in combination with the measured and calculated top-floor accelerations of the two considered buildings, led to the proposal to limit the 6-year rms accelerations to 0,005 g (0,049 m/s^2).

IRWIN, on one hand, calibrated his motion perception thresholds (maintaining the frequency dependence) with HANSEN et al.'s comfort criterion [1]. A slight adjustment was made to use a 5-year recurrence interval instead of the 6-year interval adopted by HANSEN et al. This proposal was later adopted by the ISO 6897 guideline for the evaluation of human response to low-frequency horizontal motion [7].

On the other hand, DAVENPORT proposed a 2 and 10-percentile human comfort curve relating acceleration to recurrence interval, without frequency dependence. These curves were drawn in agreement with HANSEN et al.'s criterion and the 2% and 10% motion perception thresholds proposed by CHEN & ROBERTSON for very short recurrence intervals. DAVENPORT's human comfort curves finally developed into the criteria adopted by the Boundary Layer Wind Tunnel Laboratory (BLWTL) of the University of Western Ontario [1].

More recently, vibration perception experiments have been conducted by TAMURA et al. [11] for a wide frequency range from 0,125 Hz to 6,0 Hz, comprising both stiff and flexible buildings. This research carried out by TAMURA et al. served as a basis for the Guidelines for the Evaluation of Habitability to Building Vibration of the Architectural Institute of Japan (AIJ). Some significant conclusions drawn in this study are listed below.

- No significant difference in human motion perception exists between sinusoidal and random motion in terms of the peak acceleration.
- Frequency dependence was found to exist throughout the entire frequency range of 0,125 Hz - 6,0 Hz: human motion perception decreases with decreasing frequency for flexible buildings, whereas for stiff buildings the motion perception decreases with increasing frequency. The human motion perception threshold is lowest for the intermediate frequency zone. The above is illustrated in figure 9 (taken from reference [11]), where the annual peak acceleration is shown as a function of the vibration frequency and the percentile of the population objecting to such a vibration.
- Frequency dependence seems to be weaker for random than for sinusoidal motion.

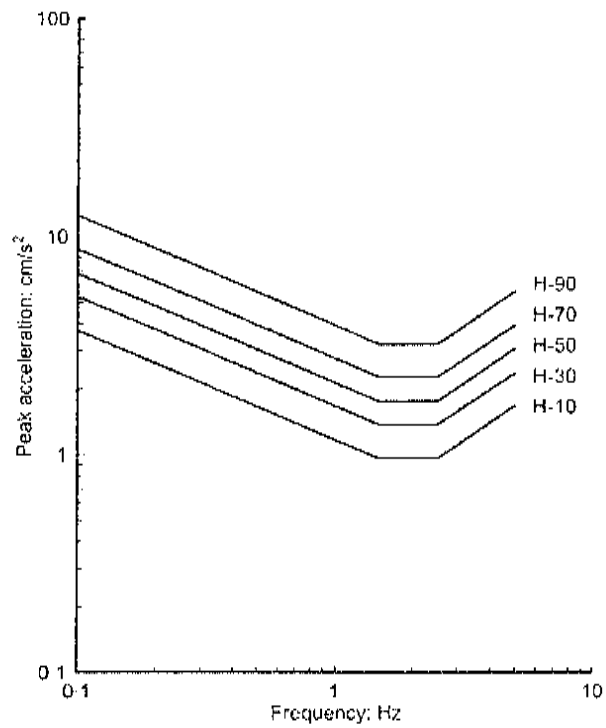

Figure 9: Perception thresholds in terms of annual peak acceleration according to [11]

Root-mean-square or peak value

The majority of research on motion perception thresholds has been presented in terms of the root-mean-square (rms) acceleration. Nevertheless, others claim that the peak acceleration is a better measure of human motion perception.

Motion-induced discomfort can be thought of as being the result of a sustained phenomenon, best described by an average effect over a period of time. The rms acceleration can represent this time-averaged effect, i.e. the intensity of acceleration.

However, some suggest that people are affected mostly by the largest peak of acceleration and tend to forget less-than-peak or mean accelerations.

Adopted criteria

Various guidelines are available to the structural engineer to assess the comfort of tall building occupants. In this study, two of these guidelines are considered, namely the ISO 6987 and BLWTL criteria.

ISO 6987 proposes maximum values of the 5-year rms acceleration at the top occupied floor, during the 10 severest minutes of a wind storm, as a function of the vibration frequency. The denominated "curve 1" in figure 10 represents the limit for general-purpose buildings, whereas "curve 2" represents the comfort criterion for offshore structures. Note that within general-purpose buildings no distinction is made between different occupancy rates following from, for example, a residential or office use.

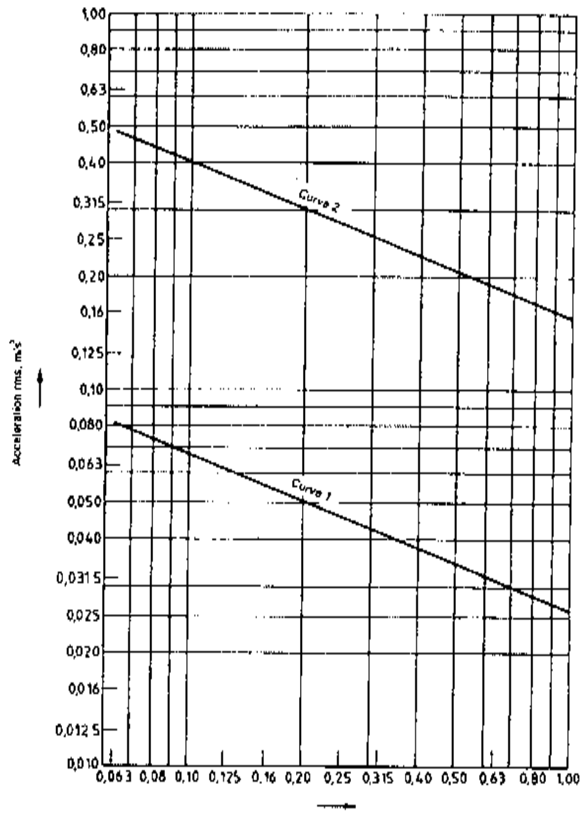

Figure 10: Maximum 5-year rms accelerations according to ISO 6897 [7]

The BLWTL criteria are widely used in North American structural engineering practice. Reference [9] provides maximum values of the 10-year peak accelerations for office and residential buildings. Afterwards, an intermediate criterion for hotels has been drawn up. The BLWTL maximum acceleration ranges are provided in table 1. Maximum 10-year peak accelerations of 25 milli-g (0,245 m/s^2) are proposed for office buildings. Note that a distinction is made between different building occupancy rates as a function of the building's use. This reflects, furthermore, the fact that during a sever wind storm an office building would probably be evacuated and people would take shelter in their homes.

	range $[g.10^{-3}]$
office	20 - 25
hotel	15 - 20
residence	10 - 15

Table 1: Maximum 10-year peak accelerations according to BLWTL [9]

4. BUILDING DESCRIPTION

In what follows a description is provided of the four tall buildings that are analysed in this study. The architectural and structural characteristics are dealt with superficially, as well as the lateral load resisting properties.

4.1 TORRE ESPACIO

Architecture

The appearance of Torre Espacio is characterised by the progressive change of its geometry throughout the building height. The square floor plan at base level has approximate dimensions of 42 m x 42 m and towards the top it changes into a quasi-elliptical shape composed of two arcs of a circle with in-plan dimensions of 59 m x 30 m. The total building height above grade is approximately 223 m, while the substructure consists of a six-level basement with a total depth of 18 m. A typical floor plan of a basement, low-, mid- and high-rise section is provided in figure 11.

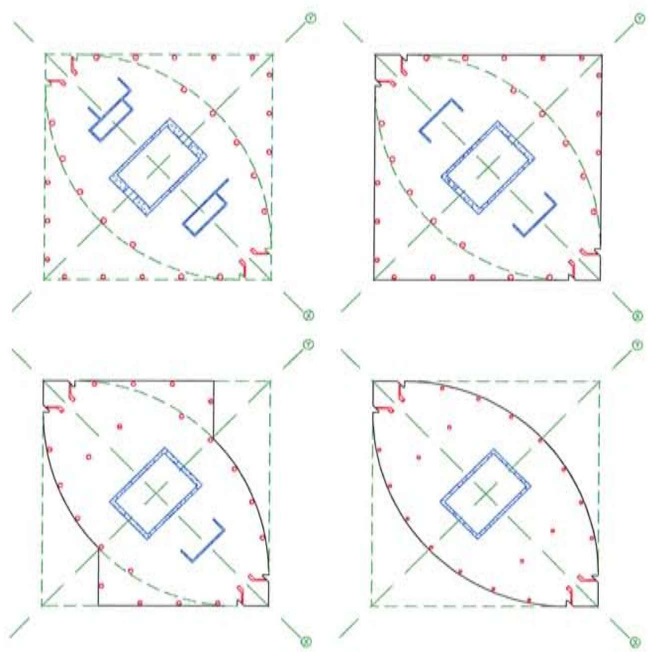

Figure 11: Torre Espacio with typical floor plan at basement, low-, mid- and high-rise section

Structure

The vertical structure is composed of three reinforced concrete cores with two groups of composite or reinforced concrete columns; one interior group that describes the quasi-elliptical shape and another exterior one that forms the square geometry of the low-rise zone.

The central box-shaped core runs along the total height of the building and the two lateral C-shaped cores stop at approximately one-third and two-third of the height. The concrete quality used in the supports is C70/85, C40/50 and C30/37 in ascending order.

A 280 mm two-way reinforced concrete plate constitutes the floor system throughout the entire building. Concrete qualities C40/50 and C30/37 are employed in the lower and upper part respectively.

At approximately two-third of the building height, a mechanical floor accommodates an outrigger structure composed of a series of one-storey high reinforced concrete walls protruding from the central core. In this way a series of stiff I sections is rigidly connected to the core. The outrigger and belt structure connect five perimeter columns at both faces of the building to the central core, as can be seen in figure 12. The favourable location in combination with the outrigger's great moment of inertia considerably increases the lateral building stiffness. A secondary effect, caused by its large flexural stiffness, is that it transfers a great part of the column loads above the outrigger to the core. This reduces the effects of differential shortening between the vertical load bearing elements, but at the same time induces tensile stresses in the upper floor. These tensile stresses are compensated by post-tensioning the upper floor.

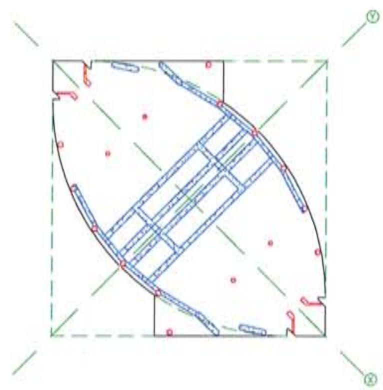

Figure 12: Outrigger and belt structure

The building and basement structure outside the building projection are two completely independent structures. The wind-induced shear forces in the building structure are entirely transmitted to the soil by means soil-foundation friction, because the building is not horizontally supported by the basement structure.

A shallow, slab-on-grade foundation is employed to transmit the loads to the underlying soil. The foundation consists of a 4 m post-tensioned reinforced concrete slab.

Lateral load resisting characteristics

The great bending stiffness of the reinforced concrete, flat plate, floor structure results in a large contribution of the columns to the building's lateral stiffness. This contribution is further enhanced by the outrigger and belt structure at two-third of the building height. The contribution of the cores and columns to the lateral stiffness in the y-direction is approximately 35% and 65% (40% due to the floors and 25% due to the outrigger and belt structure). In the x-direction the stiffness distribution results 30% - 70% for the cores and columns respectively.

The first three vibration frequencies of the system under the quasi-permanent load combination are provided in table 2. These values incorporate the P-delta effect, being approximately 13% and 11% in the x and y-direction respectively. The force coefficient profile of Torre Espacio is shown in figure 13.

mode	frequency [Hz]	period [s]
X_1	0,125	7,98
Y_1	0,135	7,39
T_1	0,322	3,11

Table 2: Vibration frequencies

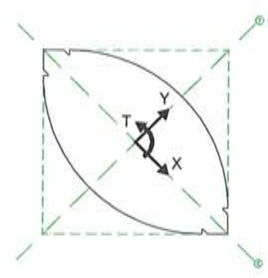

Figure 13: Axes definition

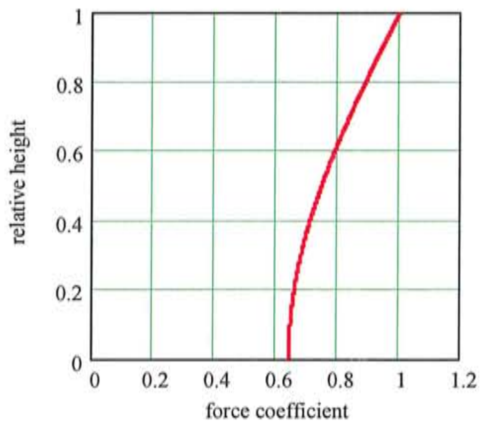

Figure 14: Force coefficient

The progressive reduction of the floor plate with height reduces the modal mass and, therefore, the period of fundamental vibration. The, nevertheless, relatively high fundamental periods reveal the flexibility of the building due to the large mass of the adopted floor system.

4.2 TORRE DE CRISTAL

Architecture

The in-plan geometry at ground level of Torre de Cristal is rectangular with approximate dimensions of 49 m x 33 m. The geometry changes towards the top into an irregular hexagon (six-sided polygon), as illustrated in figure 15. The building height above grade is approximately 249 m, whereas the substructure consists of a six-level basement with a total depth of 19 m.

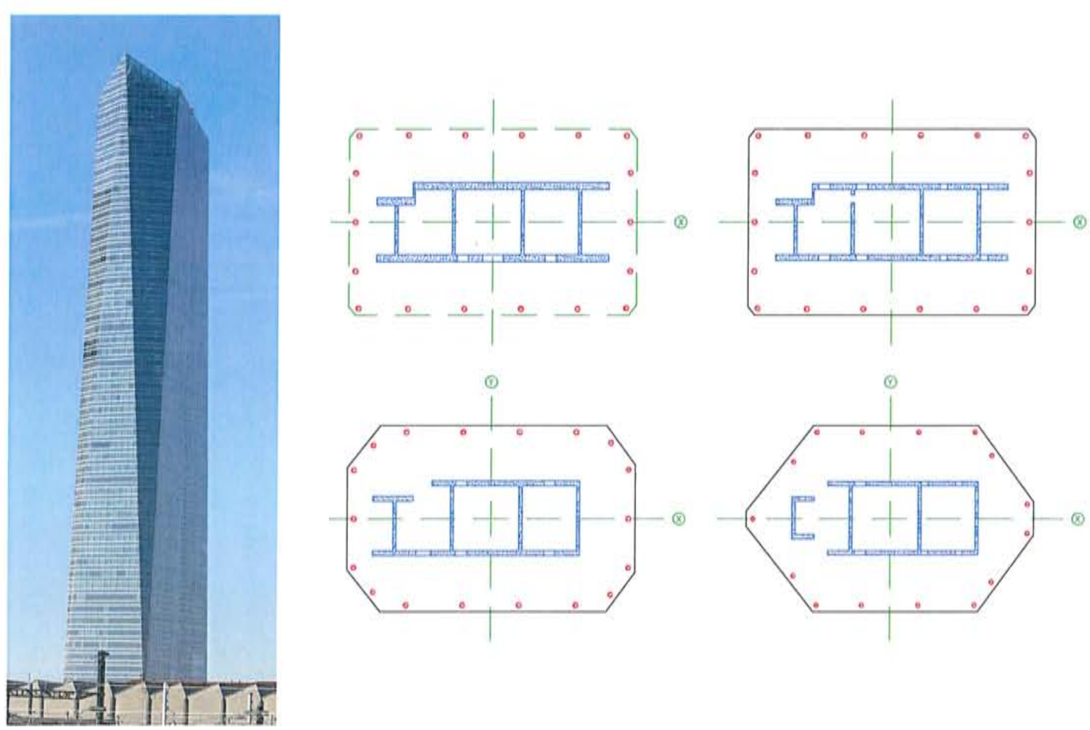

Figure 15: Torre de Cristal with typical floor plan at basement, low-, mid- and high-rise section

Structure

The vertical structure is composed of a large reinforced concrete core and composite perimeter columns. A C45/55 concrete quality is employed in all supports and throughout the entire building height.

A composite metal deck floor constitutes the horizontal structure within the core. Outside the core, a precast concrete hollow-core slab is employed with an in-situ concrete topping. The hollow-core planks are supported by steel beams spanning between the core and the columns or perimeter beam.

The building experiences a flexible horizontal support from basement level 2, constituted by the perimeter slurry walls. Part of the shear in the building structure is transmitted to the slurry walls by means of the diaphragm action of the basement floors.

The deep foundation system is composed by 20 m deep, load bearing, reinforced concrete slurry walls and a 1,5 m thick reinforced concrete slab at basement level 6.

Lateral load resisting characteristics

The central core is the main lateral load resisting element. The contribution of the columns to the lateral building stiffness is small, being approximately 5% in x-direction and 10% in y-direction. The obtained P-delta effect for Torre de Cristal is 2% and 7% in the x and y-direction respectively.

mode	frequency [Hz]	period [s]
Y_1	0,156	6,43
X_1	0,296	3,38
Y_2	0,769	1,30
T_1	0,818	1,22

Table 3: Vibration frequencies

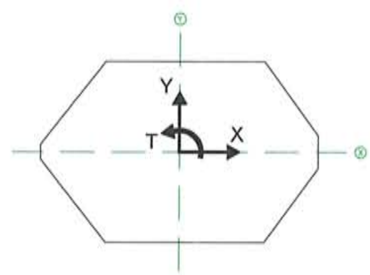

Figure 16: Axes definition

From a structural viewpoint, the orientation of the core is not optimal, because the building structure has the largest lever arm in the x-direction in which the building is less excited by the wind action. As a consequence, the building's lateral stiffness in y-direction (subjected to the highest wind loads) is approximately 3,6 times smaller than in the perpendicular direction. The effect of this architectural condition is reflected in table 3.

The force coefficient throughout the building height is illustrated in figure 17.

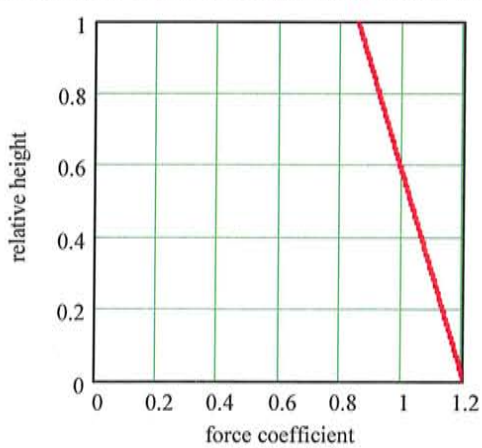

Figure 17: Force coefficient

4.3 TORRE SACYR VALLEHERMOSO

Architecture

Torre Sacyr Vallehermoso is characterised by its uniform geometry, composed of three arcs of a circle. The total building height is 232 m and the largest in-plan dimension is approximately 46 m. A six-level parking basement is located below ground level with a total depth of approximately 21 m. The lower two-third of the building is designed to house a hotel and the upper one-third to accommodate offices. Figure 18 presents a typical floor plan of a basement, low-, mid- and high-rise section.

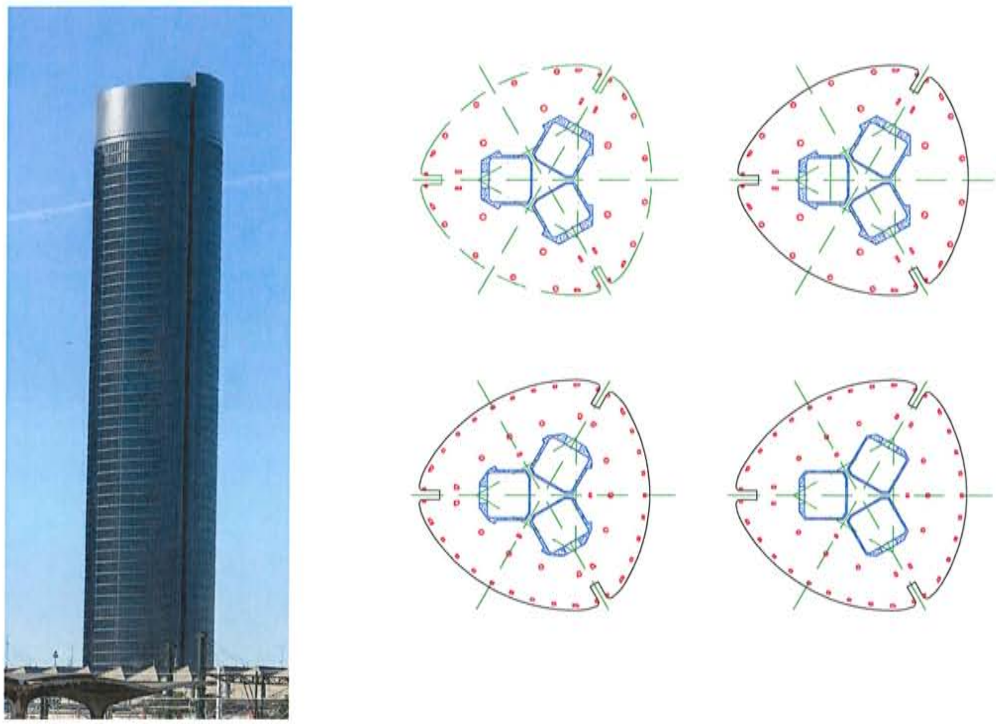

Figure 18: Torre Sacyr Vallehermoso with typical floor plan of basement, low-, mid- and high-rise section

Structure

The vertical structure is principally composed of a central reinforced concrete core and two rings of reinforced concrete or composite columns. The complex core structure consists of three interconnected square-shaped cores. In ascending order, concrete qualities C45/55 and C30/37 are adopted in the core structure, while C70/85, C45/55 and C30/37 are employed in the columns.

A composite metal deck with composite beams, and a uniform reinforced concrete plate constitutes the floor system inside and outside the core respectively. The metal deck is supported by steel beams and girders and the concrete slab is connected to these beams by means of welded studs. The girders are considered as pin connected to the vertical supports.

One of the upper mechanical levels accommodates a 5 m high reinforced concrete outrigger structure, as shown in figure 19. The six connected columns are partly suspended from the core because of the large flexural stiffness of the outrigger. Part of the load acting on the storeys below the outrigger is, therefore, transmitted to the core.

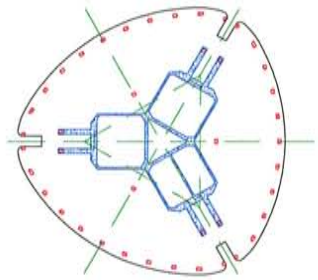

Figure 19: Outrigger structure

The wind-induced shear in the building structure is partly transmitted to the perimeter slurry walls by means of the diaphragm action of the basement floors.

The shallow foundation system is a 4 m thick post-tensioned reinforced concrete slab.

Lateral load resisting characteristics

The lateral building stiffness is largely attributed to the central core. The outrigger structure activates six columns of the interior ring as a result of the core rotation. The stiffness distribution between the core and the columns is approximately 85% - 15%. The contribution of the outrigger structure is relatively modest, because of a non-optimum vertical and in-plan location. For a given geometry, the optimum outrigger location depends of the stiffness relation between the core, the outrigger and the columns connected to the outrigger. However, in general it can be stated that single-outrigger optimum locations are comprised between 0,6 H and 0,7 H. As far as the in-plan location is concerned, only one pair of columns effectively increases the lever arm in each principal direction.

The thirst three vibration frequencies of Torre Sacyr Vallehermoso are listed in table 4. In both translational directions a P-delta effect of around 9% was found.

mode	frequency [Hz]	period [s]
X_1	0,131	7,65
Y_1	0,135	7,39
T_1	0,402	2,49

Table 4: Vibration frequency

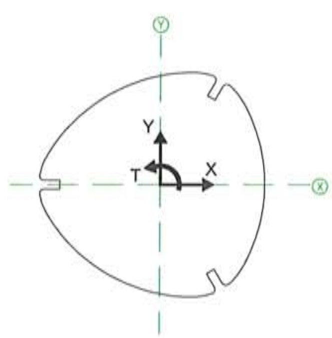

Figure 20: Axes definition

The aerodynamic, almost circular, geometry of the building results in a constant force coefficient of 0,71. The calculation of the wind-induced structural response has not accounted for the, possibly positive, effect of the façade's porosity.

4.4 TORRE CAJA MADRID

Architecture

The office building Torre Caja Madrid has a total height of 250 m above grade, whereas the five-level basement extends to approximately 18 m below ground level. The appearance of the building is determined by two lateral cores, supporting the entire building, connected at the top by an arch-like structure. The approximate dimensions of a typical cross section are 53 m x 43 m. Figure 21 shows four typical floor plans of a basement, low-, mid- and high-rise section.

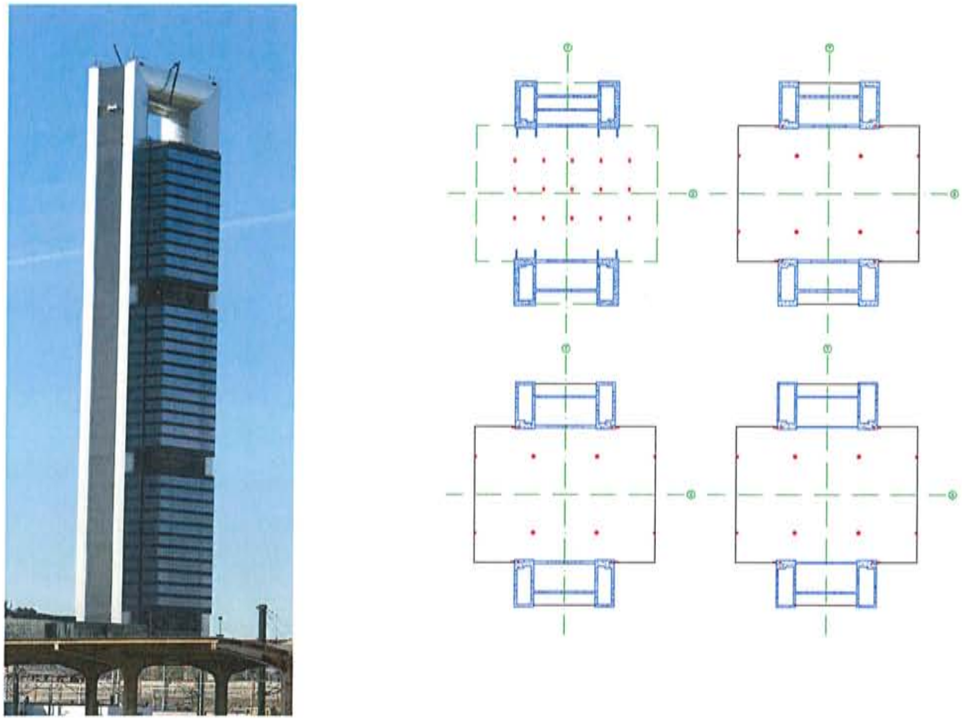

Figure 21: Torre Caja Madrid with typical floor plan of basement, low-, mid- and high-rise section

Structure

Two reinforced concrete cores constitute the principal vertical structure, between which three office blocks are supported by three steel trusses spanning between the lateral cores. These two-storey high trusses are efficiently located at mechanical levels. The employed concrete quality in the cores varies between C55/67, C50/60 and C40/50.

The secondary trusses, depicted in figure 22 in horizontal sense, support each of the eight columns. The loads from these columns are transmitted to the lateral cores by means of the primary trusses, as illustrated in figure 23 and 24 taken from reference [6].

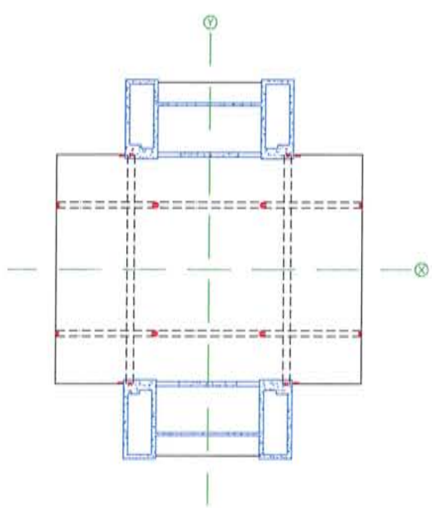

Figure 22: Steel transfer trusses

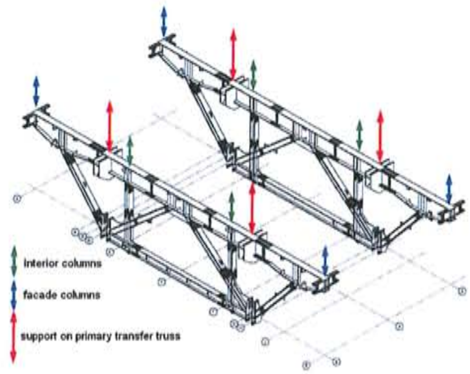

Figure 23: Secondary transfer truss [6]

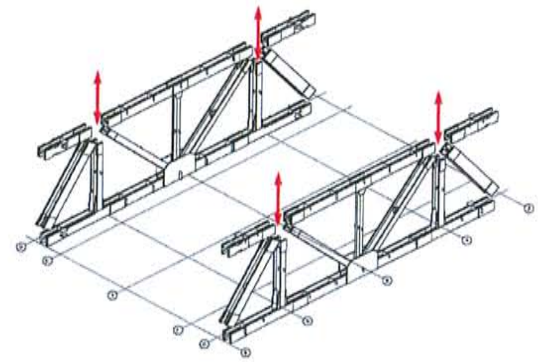

Figure 24: Primary transfer truss [6]

A composite metal deck floor with a light-weight concrete slab constitutes the secondary horizontal structure. Uniform, 400 mm thick, reinforced concrete plates are typically employed inside the core. At levels where the truss chords frame into the lateral cores, a 1,9 m thick post-tensioned reinforced concrete floor is adopted. The steel structure at the top of the building is pin connected to the cores, because of which it does not transmit any bending moments.

The diaphragm action of the basement floors, in combination with the large in-plane stiffness of the slurry walls around the plot, results in a flexible horizontal support from ground level.

The loads from the building are transmitted to the soil by means of a 5,0 thick reinforced concrete foundation slab.

Lateral load resisting characteristics

The bending-dominated lateral load resisting system in x-direction is composed of the two concrete cores. In y-direction, a megaframe behaviour is obtained due to the rigidly connected transfer trusses; about 60% of the stiffness in y-direction is attributed to this frame action. Note that the top steel structure does not have any influence on the global frame action. P-delta effects were found to be rather small, being 4% and 5% respectively in the x and y-direction.

The frequencies of the first three harmonic vibrations are provided in table 5.

mode	frequency [Hz]	period [s]
X_1	0,185	5,41
Y_1	0,187	5,35
T_1	0,487	2,05

Table 5: Vibration frequencies

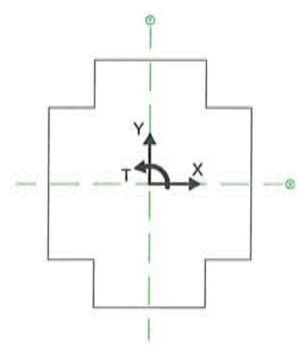

Figure 25: Axes definition

The x-direction is the governing wind direction, having the highest fundamental period and wind excitation. The force coefficient is constant along the building height and equals 1,20.

5. RESULTS

This chapter presents and discusses the results obtained in this study. The 50-year along-wind displacements and the 5 and 10-year along-wind accelerations are provided in section 5.1 and 5.2 respectively.

5.1 LATERAL DISPLACEMENTS

Along-wind displacements, with a return period of 50 years, have been computed for the governing wind direction of each building. The structural and aerodynamic damping has been estimated according to Eurocode 1. Total damping values of approximately 2% of critical have been obtained for all buildings in the governing wind direction.

The resulting size and dynamic factors, as defined in expression (3), are listed in table 6.

	c_s [-]	c_d [-]
Torre Espacio	0,84	1,12
Torre de Cristal	0,84	1,10
Torre Sacyr Vallehermoso	0,85	1,16
Torre Caja Madrid	0,84	1,06

Table 6: Size and dynamic factor

The first column represents the spatial average of the peak wind gusts acting on the building surface. Practically the same values are found for all buildings which is explained by their similar global dimensions.

The dynamic factor, however, shows considerable variation among the analysed tall buildings. The dynamic amplification depends on the damping, the fundamental period and the building dimensions in relation to the mean length of turbulent vortices. Because the damping, global dimensions and turbulence characteristics are practically equal, the variation in dynamic amplification reflects the differences between the earlier reported fundamental vibration periods.

Figure 26 depicts the force coefficient profile for each building, while figure 27 provides the 50-year equivalent static wind load per unit length as applied to the finite element models.

Figure 27 demonstrates the aerodynamic design of Torre Sacyr Vallehermoso, being subjected to a relatively low wind load despite experiencing the highest dynamic amplification. The contrary holds for Torre Caja Madrid; it experiences the highest wind excitation in spite of undergoing the smallest dynamic amplification.

Torre Espacio's wind load diagram is characterised by an increasing force coefficient with height, whilst the geometry of Torre de Cristal becomes more aerodynamic with increasing height.

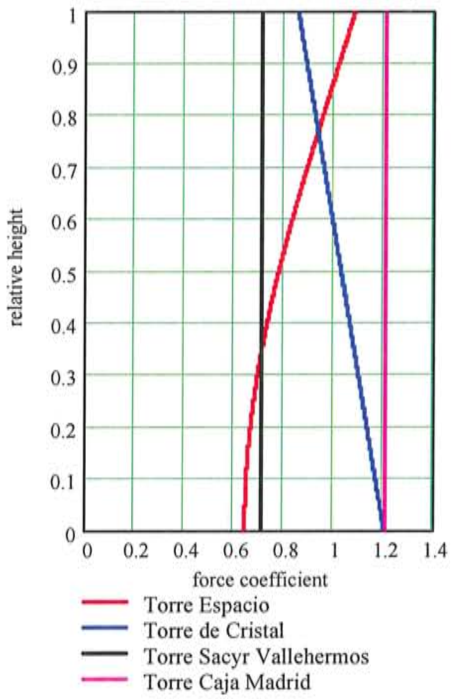

Figure 26: Force coefficient

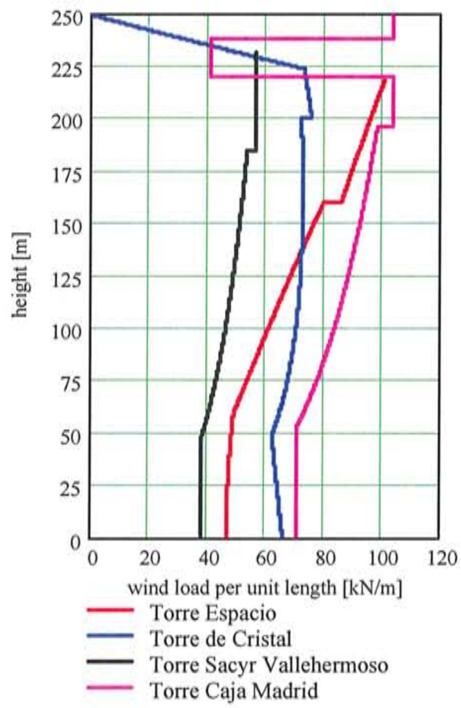

Figure 27: Equivalent static wind load

Table 7 provides the calculated lateral displacements and the drift ratio for each building.

	height [m]	displacement [m]	ratio [-]
Torre Espacio	219	0,449	488
Torre de Cristal	210	0,315	666
Torre Sacyr Vallehermoso	232	0,249	931
Torre Caja Madrid	220	0,189	1.164

Table 7: 50-year lateral displacements

The displacements are calculated between grade and a high-rise storey above which excessive displacements cannot cause any significant damage.

It is noted that maximum inter-storey drifts of H/420 have been computed for Torre Espacio.

5.2 LATERAL ACCELERATIONS

The horizontal acceleration at the top occupied floor has been adopted in this study to assess the human comfort. Both 5-year rms accelerations and 10-year peak accelerations have been calculated.

The structural damping depends, among other things, on the expected building displacement because of which different damping values have to be adopted 50, 10 and 5-year winds. For the latter two return periods, a total damping value of 1% of critical has been employed for all four buildings.

The acceleration diagrams as a function of the building height are shown in figure 28 and 29. The 5-year root-mean-square acceleration is presented in figure 28, whereas figure 29 provides the 10-year peak acceleration diagram for each building.

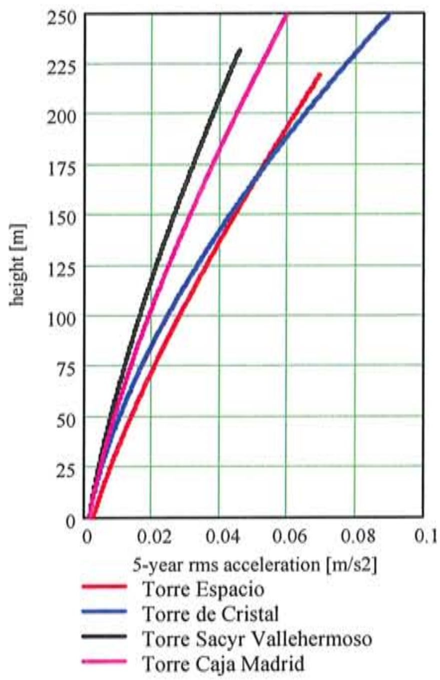

Figure 28: 5-year rms acceleration

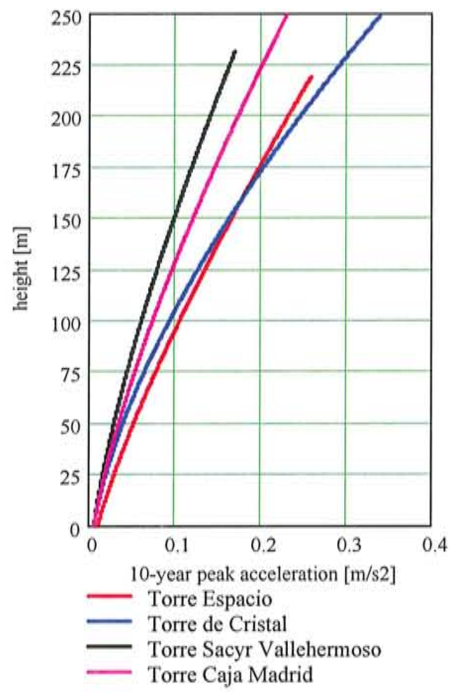

Figure 29: 10-year peak acceleration

Along-wind accelerations at the top occupied floor for a 5 and 10 year recurrence interval are listed in table 8.

	height [m]	$\sigma_{5\,year}$ [m/s^2]	$a_{10\,year}$ [m/s^2]
Torre Espacio	204	0,063	0,235
Torre de Cristal	210	0,073	0,263
Torre Sacyr Vallehermoso	196	0,037	0,136
Torre Caja Madrid	210	0,049	0,189

Table 8: 5 and 10-year lateral accelerations

6. ACKNOWLEDGEMENTS

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