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Structural Analysis of "La Giralda", Bell Tower of the Seville Cathedral

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La Giralda is a tall bell tower, an icon of Seville, the city where it stands. It was built in the 12th century using brick and then modified and increased in the 16th century to reach its current height: more than 90 meters. This height and its other geometric characteristics, together with its resistance to all external agents suffered for more than four centuries (especially earthquakes) make La Giralda an interesting monument on a structural level. For this reason, this article studies and analyzes the structural capacity of this monument from the current engineering vision: a geometric analysis is carried out, an analysis of the foundation is highlighted, a gravitational structural analysis (own weight) is carried out, a third-party analysis is carried out, and fourth are structural analyzes against wind and earthquake. This fourfold analysis will allow a better understanding of the structural functioning of the monument as a whole and its survival after so many years.

Figure 1: East facade of the bell tower Giralda (photograph by the author)

Introduction

Towers have been one of the biggest construction challenges ever. From the Tower of Babel (Gen, 11) until today, the economic power of countries has been measured by their ability to build tall buildings [1].

To build higher and higher means to achieve a balance between what the building is capable of supporting and the movements that the building can accept. Therefore, stability and resistance play a fundamental role in establishing the sizes that can be made, depending on each construction material, and height-to-base ratio.

La Giralda is the bell tower of the Seville Cathedral; it is a great and beautiful tower; it is a fascinating tower (Figure 1 and Figure 2). It has been the tower of Seville by excellence and it still is. It is part of a set, declared a World Heritage by Unesco [2]. This bell tower is a mixture of an ancient Almohad tower and a later Renaissance bell tower [3]. Looking at the Giralda tower we can contemplate the result of fusing the Mozarabic tradition and the Renaissance renewal. The end of one contribution and the beginning of the other cannot be distinguished: their appearance is homogeneous [4].

We can say that it is a perfect symbiosis of the powerful tower of Ben Basso (Mozarabic) and the magnificent design of Hernán Ruiz (Renaissance). The Almohad minaret is from the year 1184 and the Renaissance solution is from the year 1566 [5]. The success of the solution made the Giralda bell tower a model from the Renaissance for other towers in Andalusia and the rest of the Iberian Peninsula [6].

The bell tower is technically a double-tube wall structure, on a square plan of just over 13.50 meters per side. This double tubular structure was used by Hernán Ruiz to surround the top of the old Almohad tower with a supplement of bells and other modules (for clocks and caroms). In this way, the tower was raised to reach the current 94.69 meters high (Figure 4).

The material chosen for its construction is river block brick. This material has been used in Seville since the most remote tradition [7]. The base of the tower is reinforced with limestone (calcarenite to be more precise).

The foundation, based on an extensive improvement of the pre-existing terrain, allowed Hernan Ruiz to add 3,000 tons of weight to the 14,000 tons of the Ben Basso tower, according to data from previous studies that we will later verify [8]. The weather vane that tops the tower is known as "el Giraldillo", which gives its name to the bell tower.

The Giralda bell tower is a masonry bell tower. Masonry structures lack tensile and flexural strength. Therefore, the characteristics of a masonry tower make it vulnerable to external actions: construction interventions, wind, earthquake, etc. Having successfully withstood all the earthquakes that have occurred in Seville since its construction makes the Giralda tower a structurally interesting monument. For this reason, this article presents its analysis.

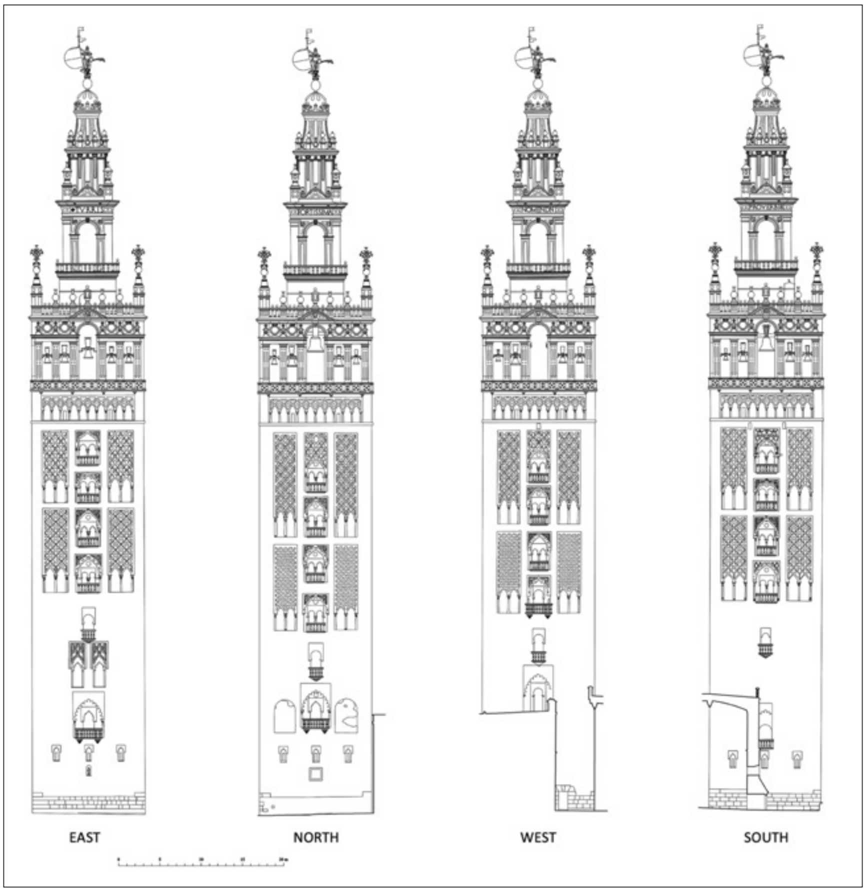

Figure 2: Elevation plans of the Giralda tower, in plans by Antonio Almagro Gorbea [9].

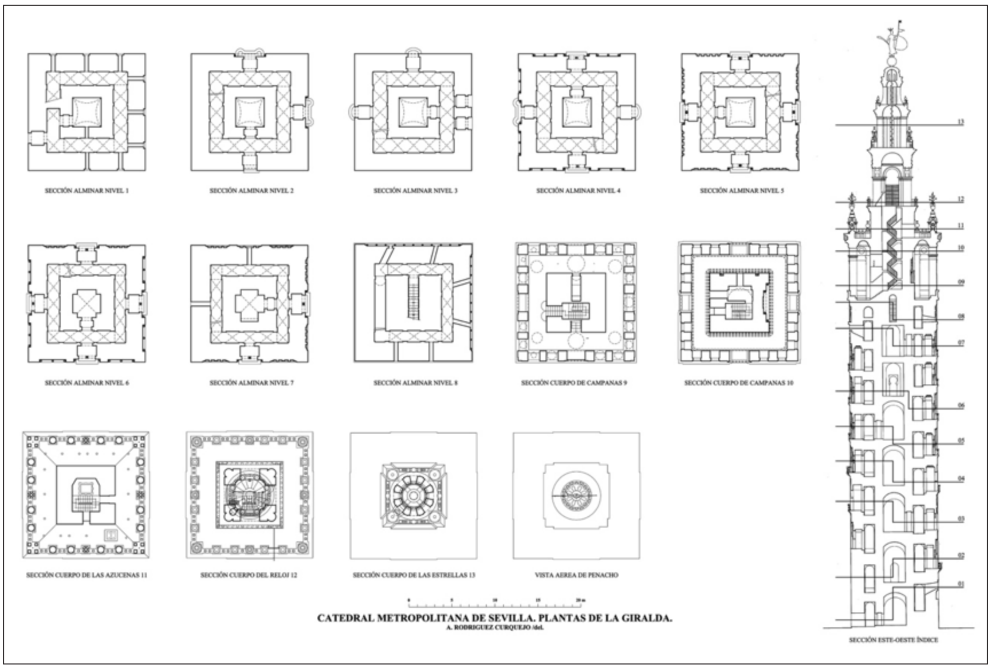

Figure 3: Evolution of the section at the different heights of the bell tower, in plans by Almagro Gorbea [9].

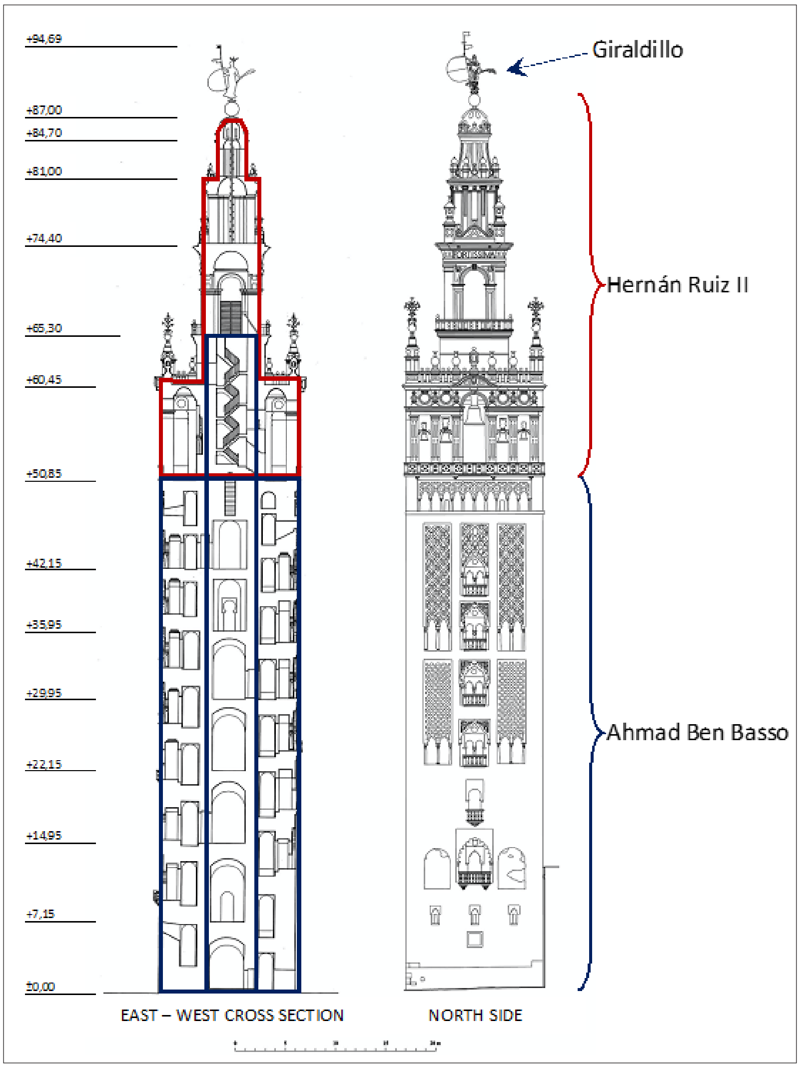

Figure 4: Cross section and north elevation, indicating the heights and the action areas of the two builders involved in the bell tower construction (graph by the author over plans by Almagro Gorbea [9]).

Objectives

The general objective of this article is to carry out a complete structural analysis of a complex old masonry structure with great symbolic value: the bell tower of the Cathedral of Seville.

To Achieve this Objective, the Following Specific Objectives are Set:

1. Carry out a geometric and formal analysis of the bell tower, contemplating its variation throughout history.
2. Carry out an analysis of the foundations of the Giralda bell tower and the land on which this tower is founded, based on geological-geotechnical surveys carried out years ago.
3. Analyze the behavior of the Giralda bell tower against the its own weight action, since old buildings resist mainly by gravity.
4. Analyze the behavior of the Giralda bell tower against the action of the wind, given the high height of the bell tower (94.69 meters).
5. Analyze the behavior of the Giralda bell tower in the event of an earthquake.

Geometric Analysis

The Giralda is not an Almohad tower; the Giralda is not a Renaissance tower. La Giralda is a monument that synthesizes the designs of two excellent architects:

1. Ahmad Ben Baso, who built the Almohad base of the tower at the end of the 12th century;
2. And the Renaissance architect Hernán Ruiz II (1514 – 1569), who designed and directed the construction of the later bell tower.

Structurally, the building built by Ben Basso was outstanding: this made it possible to add a well-imposed Renaissance module. The result of this incorporation was magnificent and unitary [4, 5, 10]. This symbiosis was possible thanks to the use of the same type of bricks and the recovery of Mudejar customs during the Renaissance in Seville [7].

To explain the history of the monument we will use the engraving by Alejandro Guichot (Figure 5). The initial tower was built by Ahmad Ben Baso between 1184 and 1198 (Figure 5, left). Fifty years later, in the year 1248, the tower would already be the bell tower of the mosque, consecrated as a cathedral [2-3].

In the year 1356 an earthquake was registered. This earthquake caused the fall of the initial top of balls. This ornament was replaced by a small belfry (Figure 5, right). This change reduced the height of the bell tower slightly [4].

In the 16th century, Hernán Ruiz II added the upper bell body. The construction of the bell tower, as we know it today, finished in 1568 (Figure 5, center).

Figure 5: Drawing by Alejandro Guichot (1909) representing the Giralda Bell Tower, in Three Different Elevations: Its Three Phases of Construction. It is a Watercolor Outlined With a Nib [11].

The drawing by Alejandro Guichot (Figure 5) shows us that the construction of Hernán Ruiz made full use of the initial construction of Ben Basso. It was built by extending the exterior facades by 10 meters and supporting the body of the clock on the central core of the Almohad tower (Figure 4). The total height, excluding the finishing decorative elements, was extended from 74.40 meters to the current 87.00 meters. As a shot, Hernán Ruiz el Giraldillo, who spins with the wind.

As we already pointed out in the Introduction, the Giralda bell tower has suffered the 1756 earthquake without serious problems and up to seven times, from the year 1700 to the present, winds of more than 140 km/hour.

For the dimensional and structural analysis, we have chosen the crenellated section at level 6 of Figure 3 because it is probably the most representative (Figure 6).

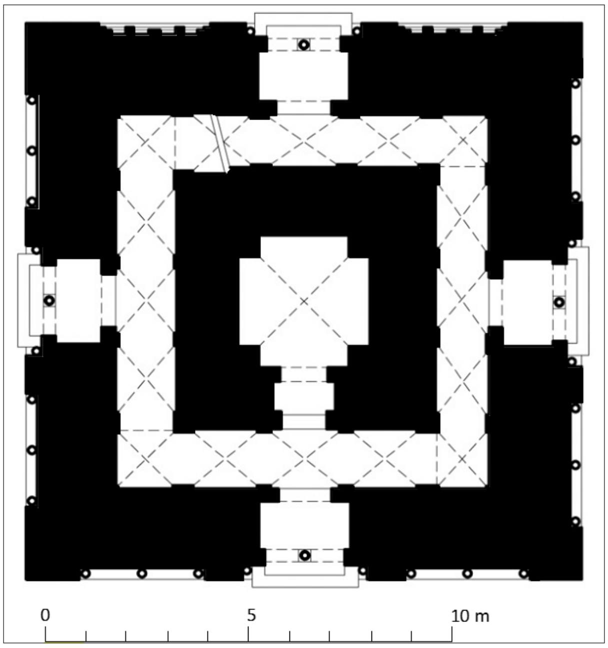

Figure 6: Plan Analysis of the Author, on the Crenellated Section at Level 6 (Elevation +36.00 M), on the Plane of Figure 3.

Measuring on the plane we can conclude that the plant is square with a side of 13.61 meters. Measuring directly on the bell tower we have been able to corroborate the above. The section is made up of an outer tube and an inner tube, with a system of ramps between them. The walls are 2.20 meters thick; they are built with brick masonry with lime mortar, double-layered with filler. The double tube system is typical of the ancient towers of North Africa, its greatest exponent of antiquity being the Lighthouse of Alexandria [6, 12] (Figure 7). This construction system simplifies the construction of the bell tower: the building grows as walls and ramps are built. On the one hand, the ramps guarantee unity between the two tubes and solve warping problems; and, on the other hand, they diagonalize the tower increasing its stability.

Modern skyscrapers still use the double tube structure today. However, they are made of steel, a material with great resistance to bending. Giralda bell tower is a masonry structure and, therefore, has low tensile strength and low flexural capacity. In this case, the double tube solves the stability problems through its own weight, distributing it over the entire surface to reach the foundation with a low stress level.

All the old towers have been built using materials that worked in a uniaxial way; therefore, any instability in shape caused unacceptable traction by masonry materials. For this reason, only a small percentage of the ancient towers have survived to the present day. In the same way, in the face of catastrophes and accidents, tall buildings have been the first to succumb throughout history [1].

The most frequent causes have been ground failures, which have caused cracking states that have modified the way the structure works, generating unacceptable mechanisms that have caused its collapse. Therefore, the following section analyzes the foundation of the bell tower.

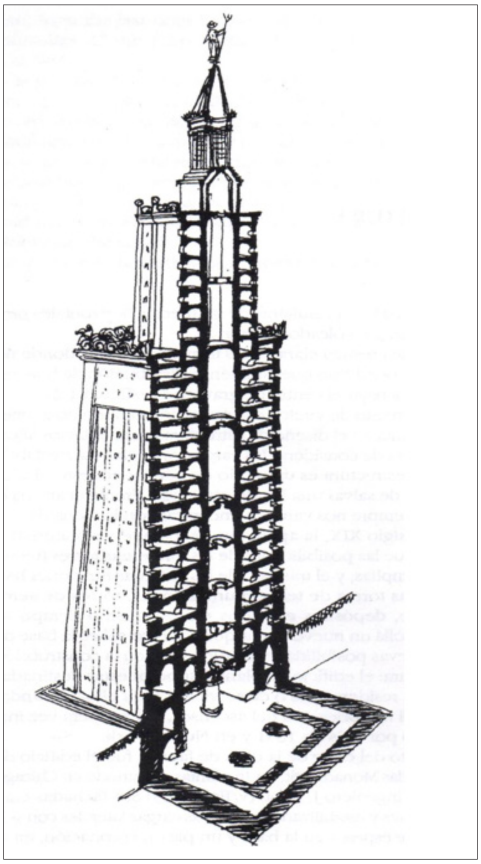

Figure 7: Central Section of Double Tube and Hollows of the Lighthouse of Alexandria [6].

This geometric analysis was complemented with several quick auscultation tests on the monument and its constituent materials [13, 14]. A complete optical analysis of the materials constituting the façade was carried out (Figure 8) and a thermographic analysis that allowed verifying the absence of pathological deficiencies in the perimeter walls (Figure 9). We must point out that at the time the auscultation was carried out, three of the four façades of the bell tower had been recently restored, leaving only one pending restoration: the north facade (Figure 9). The north facade is scheduled to be restored from April 2023, after Easter [15].

Figure 8: Detailed Optical Analysis of the Two Materials that make up the Masonry of the Giralda Bell Tower: Calcarenite at the Base and Ceramic Brick with Mortar (Photographs by the Author on Different Points of the East Facade)

Figure 9: Thermographic Analysis of the North Facade of the Giralda Bell Tower (Photographs by the Author).

Foundation Analysis
As we already pointed out in the Introduction, it is evident that the Giralda bell tower resists loads and external actions well, and has resisted them well throughout the centuries. Such a high construction lacking tensile strength must be properly founded. With the help of soundings, in 1997 it was possible to determine that the Giralda bell tower rests on a lime mortar [16]. This mortar goes from elevation -6.00 m to elevation -2.07 m, as if it were an improvement on the terrain. It has a calculated extension of approximately 360 m2 [10]. On this layer, rests the authentic foundation, formed only by four courses of calcarenite ashlars. These ashlars are padded and staggered and, in addition, they extend the square by one meter on the south side and about forty centimeters on the east and north [10, 16]. In addition, these ashlars incorporate marks of Almohad stonecutters [17]. This information is essential: the ancient Almohad tower transmitted the load to the ground. Given its high weight, it is logical that the clayey soil [18] yielded. For this reason, there is evidence of a slight collapse of the bell tower to the southeast: the collapse represents 0.57%, which meant a settlement of 17 cm [16, 10]. Later, in the 16th century, the Renaissance architect Hernán Ruiz increased the height of the bell tower by 21%, with the consequent increase in weight.

Therefore, the stress at the base of the tower is high; However, this high tension does not fall on the masonry; this load falls on the foundation. The current foundation is a 15 x 15 meter platform located 6 meters below, on already improved ground. The tension in the foundation is 645.00 kN/m2. This required a considerable improvement of the terrain, until reaching a level of -11 meters, where we have sandy clays [16, 18].

According to the geological profile shown in Figure 10, the improved base of the bell tower rests on Level II, which is a recent alluvial with abundant archaeological remains. This level is formed by strata of diverse nature: clayey sands, sandy clays, and clays with gravel and sand. Underneath, at a depth of 7 – 11 m, grayish plastic clays with remains of sand and some archaeological remains and some traces of organic matter are discovered, which reach depths of 11 – 13 m.

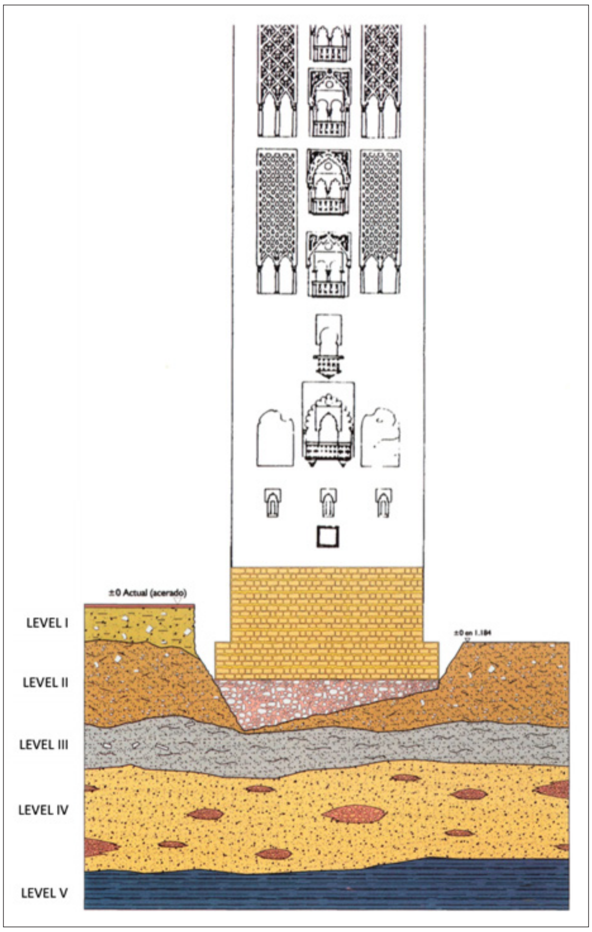

Figure 10: Foundation and Subsoil of the Giralda Bell Tower in Seville [16].

Analysis of Resistance to Its Own Weight
We are analyzing a masonry structure. A construction is a potential energy conservation challenge ($PE = m\ g\ h$) that is generated at the height reached (h). In a tower, this challenge is even greater. In a masonry tower, this challenge is even more complex. The characteristics of a masonry construction make it a very vulnerable element.

The postulates of Professor Jacques Heyman are applicable to this bell tower [20]. According to them:
1. All masonry structures have infinite compressive strength.
2. All masonry structures have zero tensile strength.
3. In all masonry structures failure by sliding is impossible.

According to this, if the Giralda bell tower has resisted so many years, the tensile stresses have been small and the compression stresses adequate [1, 21, 22].

Knowing the geometry, we are going to proceed to the analytical geometric analysis, in order to know the maximum compressive stress transmitted to the ground at the base of the bell tower due exclusively to its own weight. Following the dimensions established in Figure 4. With this, we elaborate the following table:

Scope		Heigh (m)	Mass Volume (m^3)	Density (kN/m^3)	Weight (kN)	Resistance (kN/m^2)
Ahmad Ben Basso	1	(-) 2,50	562,50	20,60	11.587,50	1.250,00
	2	(-) 2,50	463,08	20,60	9.539,45	1.250,00
	3	2,50	350,25	20,60	7.215,15	1.250,00
	4	48,35	6.773,84	17,50	118.542,11	1.250,00
Hernán Ruiz II	5	9,60	927,87	17,50	16.237,78	1.250,00
	6	4,85	257,57	17,50	4.507,50	1.250,00
	7	9,10	218,65	17,50	3.826,34	1.250,00
	8	6,60	74,23	17,50	1.299,04	1.250,00
	9	3,70	18,79	17,50	328,82	1.250,00
	10	2,30	12,31	17,50	215,44	1.250,00
Total		87,00			173.299,14	

These data do not exactly coincide with other investigations [10, 16], since they provide a slightly higher value. For rounding purposes of calculation, from now on, we will consider the total weight of the bell tower of $1.74 \cdot 10^5$ kN.

$$S = \frac{N_{TOT}}{S_{TOT}} = \frac{1{,}74 \cdot 10^5}{140{,}05} = 1242{,}41 \ {}^{kN}/_{m^2}$$

This first approximation is not real: we have not taken into account either the wind load, or the overload of use, or the internal transit ramps, or the real surface of the tower at the base. In fact, Barrios Padura et al. [16] did consider the real surface of the bell tower at the base (not that of the double tube) and the width of the foundation, obtaining with them stress values lower than the previous one. This guarantees that we stay on the side of security.

If the total base of the tower is considered (13.61 m x 13.61 m) and the equivalent foundation (15.00 m x 15.00 m), we would have:

$$S = \frac{N_{TOT}}{S_{TOT}} = \frac{1{,}74 \cdot 10^5}{13{,}61 \cdot 13{,}61} = 939{,}36 \ {}^{kN}/_{m^2} < 1.250 \ {}^{kN}/_{m^2}$$
$$S = \frac{N_{TOT}}{S_{TOT}} = \frac{1{,}74 \cdot 10^5}{15{,}00 \cdot 15{,}00} = 773{,}33 \ {}^{kN}/_{m^2} < 1.250 \ {}^{kN}/_{m^2}$$

No usage overhead was taken into account either. The statue of El Giraldillo at the top of the tower (7.69 m high) and the twenty-four bells are a permanent overload of considerable value:

- Weight of El Giraldillo: 1,26 kN.
- Weight of the bells: 246,54 kN.

It would imply that the total weight would rise:
$173.299{,}14\ kN + 1{,}26\ kN + 246{,}54\ kN = 173.546{,}94\ kN \approx 1{,}74 \cdot 10^5\ Kn$

So the previous numbers would be equally valid. In a second approximation, in addition to taking the real surface of the base of the bell tower, we would have the party walls, since the bell tower is attached to the biggest Gothic cathedral in Spain.

As a complement to the above, we are going to use a finite element model developed by Doctor Miguel Ángel Cobreros Vime [23].

We know that a masonry structure is neither homogeneous nor isotropic. Therefore, a linear elastic finite element model is imprecise for a masonry structure. Furthermore, the links in a masonry structure are unreal and, therefore, the links between finite elements will also be unreal.

Despite the above, a finite element model has been used for its expository clarity and because, if the masonry is globally compressed, with small tractions it is closer to reality. This does not mean that the modulus of elasticity is always approximate: its value is variable in masonry structures.

In addition, the model of Doctor Cobreros Vime has been used for its interest [23]: he made a complete model and two partial models, separating the two actions indicated in section 3 of this article: the initial construction of Ahmad Ben Baso and the later addition by Hernán Ruiz (Figure 11). This division could establish the stress and flexibility influence on the added upper element.

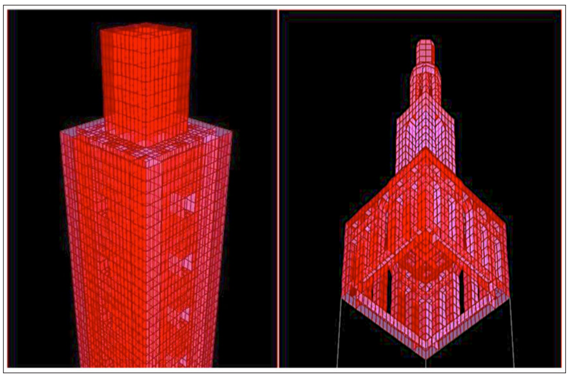

Figure 11: Finite Element Model of the Giralda Tower, Developed By Doctor Cobreros Vime, Where the Initial Construction of Ahmad Ben Baso (Left) and The Additional Construction Of Hernán Ruiz (Right) Are Differentiated [23]

We cannot forget that the bell tower is a double tube structure with solid brick walls. For calculation purposes, we are going to consider mud brick masonry with lime mortar, with the following characteristics:

- Density (γ): 17,50 kN / m^3.
- Modulus of elasticity (E) = $1{,}5 \cdot 10^6$ kN / m^2.
- Poisson's Ratio (ν) = 0,2.

The two tubes are built with 2.20 meter walls in the old part (lower fraction, Almohad). In the finite element model (Figure 11, left), they have been modeled as a double-layer wall with 1.00 meter thick surface elements. This causes a slight increase in overall inertia (14%), which has been offset by a factor that reduces stiffness.

On the other hand, the most modern part (the upper fraction, Renaissance) was also modeled with superficial elements, reproducing the geometry that was analyzed in section 3 of this article.

The ramp system between the two tubes was modeled with 25 cm thick elements, which reproduce the weight of the ramps built on ribbed vaults (Figure 3 and Figure 4). These ramps join the two tubes and diagonally stiffen the tower.

The model visually corroborates what was analyzed about the geometry: the pre-existing tower, the oldest part, is resistant and on it, on the outer tube, the raised bells were supported.

Modal Analysis, Vibration Periods

The complete model gives us a fundamental period of 1.9 seconds (Figure 12). This period is coherent for a tower whose slenderness is average is 5. In addition, it is also adequate for wind load, since it is not excessively flexible; and for the earthquake load it is also suitable, since it is not a rigid structure.

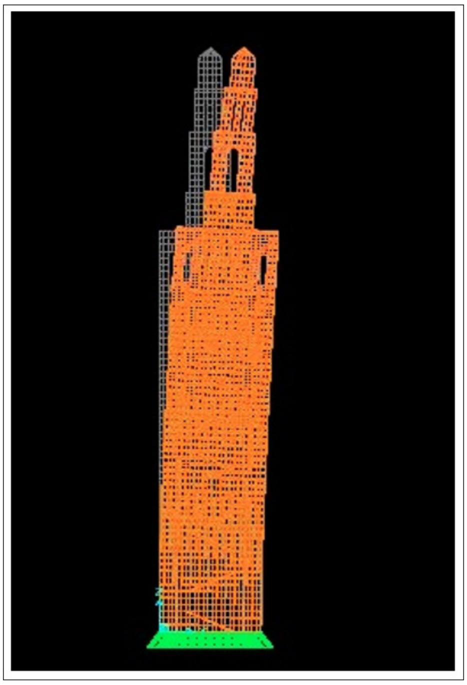

Figure 12: Fundamental Period of Vibration, According To the Model Developed By Doctor Cobreros Vime, For the Entire Set of the Bell Tower (T_1 = 1.9 Seconds) [23].

The movement of the first period of vibration is diagonal, in both models (Figure 12 and Figure 13). The wall structure of the Giralda bell tower forms a regular polygon in plan (Figure 3 and Figure 6) with the same inertia in all directions. The incorporation of ramps reduces inertia and increases flexibility in the diagonal direction. The Renaissance addition of Hernán Ruiz accompanies the movement, perfectly coupled, in the fundamental period (Figure 13, right).

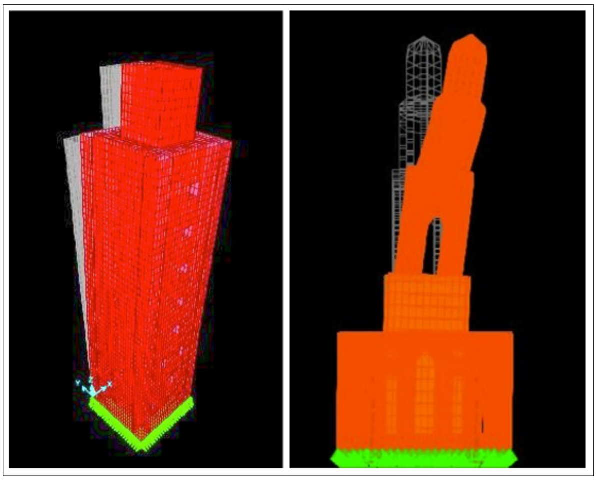

Figure 13: Fundamental Period of Vibration, According To the Model Developed by Doctor Cobreros Vime, for the Initial Construction of Ahmad Ben Baso (Left, T_1 = 1.5 Seconds) and the Additional Construction of Hernán Ruiz (Right, T_1 = 0.75 Seconds) [23].

In the lower part of the bell tower, in the Almohad fraction, the third period is logically one of rotation. For its part, the complete model shows the independent movement of the additional Renaissance body in the third period (Figure 14).

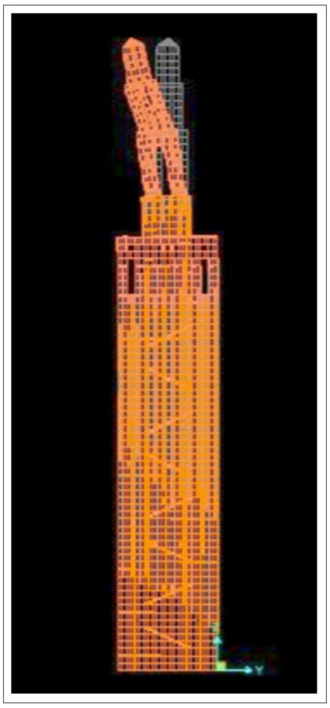

Figure 14: Third Period of Vibration, According to the Model Developed by Doctor Cobreros Vime, for the Entire Set of the Bell Tower (T_3 = 0.7 Seconds) [23].

The Almohad Tower had a period of 1.5 seconds. Therefore, the superior addition of Hernán Ruiz, by increasing the mean slenderness from 4 to 5, proportionally increased the period. This fraction has, as an independent structure, a period of 0.75 seconds for 30 meters high (Figure 13, right), very similar to the third period of the complete tower (Figure 14).

If we were to perform a small computation of the model with a modulus of elasticity halved (0,75 • 10^6 kN / m^2), we would see the period increase to 2.6 seconds. This means that, given the uncertainty in the value of the modulus of elasticity, taking a minimum value among those recommended for masonry structures continues to provide valid periods.

Stress Analysis

The stress analysis obtained from the model accredits the uniformity of the compression efforts. The graphs (Figure 15) show the uniform increase of stresses downwards. This makes possible the compatibility of deformations and the absence of pathological lesions, especially cracks. In line with this would be the absence of great traction: the registered traction values are less than 100 kN/m^2. This data also proves the feasibility of using a finite element method.

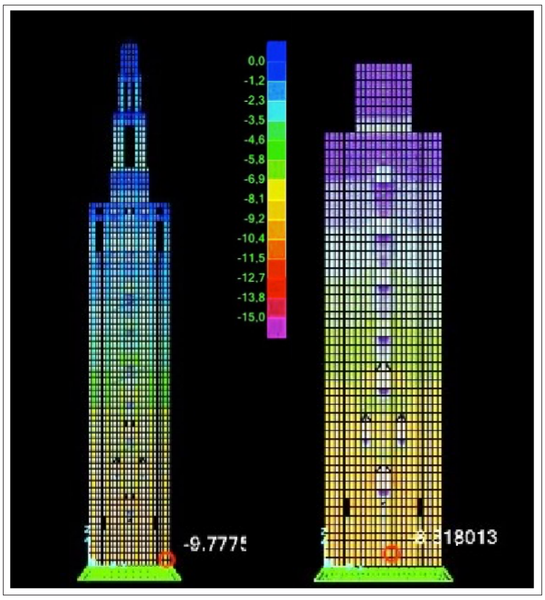

Figure 15: Stress Analysis of Self-Weight, According to the Model Developed by Doctor Cobreros Vime, for the Entire Set of the Bell Tower (Left) and for the Oldest Fraction (Right). Stress Values in the Vertical Direction Expressed in kg/cm^2 [23].

Another notable aspect is the uniformity in plant. In the lower right corner of the complete model we can see a remarkable tensional equality between both tubes (Figure 15), in the complete model. Undoubtedly, the ramps contribute to this.

The tension increase contributed by the addition of Hernán Ruiz meant going from 792,98 kN/m^2 to 935,58 kN/m^2. This increase is compatible with the increase in total weight, which went from 14,978 tons to 17,671 tons. This, as we saw in section 4, has its effect on the foundation.

Finally, there would be the analysis of the deformed self-weight (Figure 16 and Figure 17).

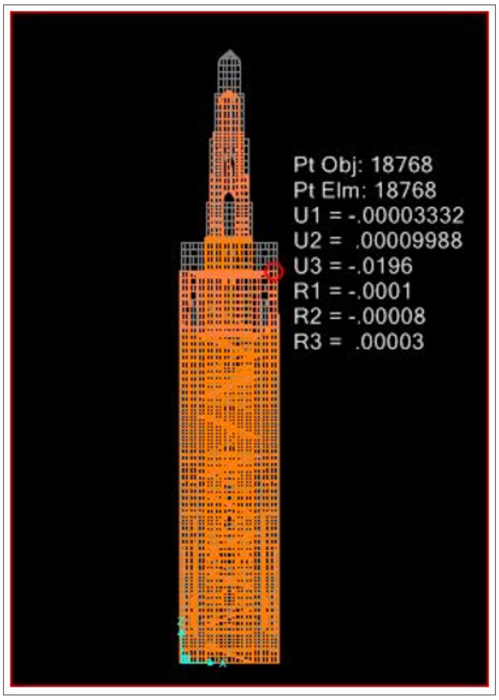

Figure 16: Deformed by its Own Weight of the Bell Tower (Old Part and New Part), on the Model Developed by Doctor Cobreros Vime [23].

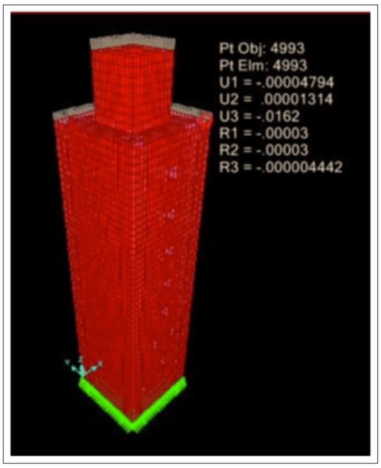

Figure 17: Deformed Under its own Weight of the Initial Construction by Ahmad Ben Baso, on the Model Developed by Doctor Cobreros Vime [23].

The total value of the shortening of the bell tower depends very much on the modulus of elasticity. However, it is worth noting the uniformity with which this shortening occurs, a result of stress uniformity.

The upper module, executed by Hernán Ruiz in the 16th century, represented a 20% increase in deformation (Figure 16). If we take a minimum value for the modulus of elasticity, the shortening would be up to 4 cm, instead of the 2 cm that this model gives us.

Wind Resistance Analysis

For the analysis of the action of the wind, the north and east facades are considered, as they are the only facades without party walls. (Figure 18).

In the event that the intermediate slabs did not participate in the reaction produced in response to the wind effort, the outer perimeter wall would be in charge of resisting it. Without a connection between the two enclosing sheets, the outer wall would be unable to contain the line of thrust that directs the force of the wind to the ground. In plan, the unloading arch has been drawn that conducts the effort of the wind to the side walls (Figure 19). This arc is the one corresponding to the minimum thrust. Without these slabs, the hollow square plan structure would be unable to respond to the wind on both sides of the enclosure: due to a lack of tensile strength, the forces cannot be transmitted within the enclosure.

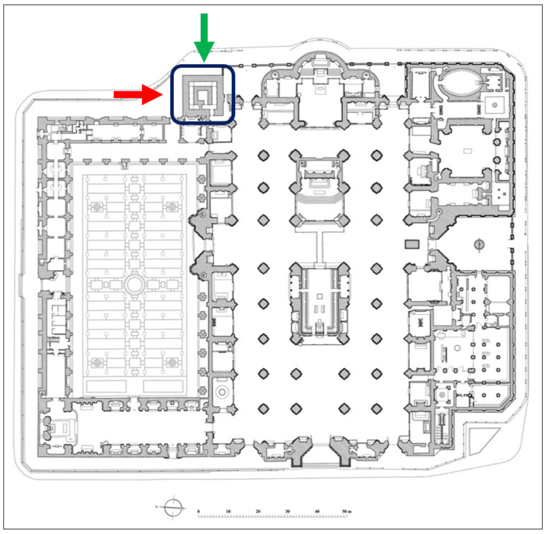

Figure 18: General plan of the cathedral of seville, on which the location of the giralda and its north (red arrow) and east (green arrow) façades have been marked, on which the wind would reach the maximum force due to the absence of party walls (graphic of the author, on map of almagro gorbea [9]).

We start from the premise that the load due to the wind is much less than the load due to its own weight. We are going, in a simplified way, to demonstrate the above considering only the section of brick corresponding to the old part of the bell tower (section of Ahmad Ben Baso).

If we analyze the own weight of the tower in that section, we would have:

$P = A \cdot H \cdot \gamma = [B^2-(B-e)^2] \cdot H \cdot \gamma = H \cdot \gamma \cdot 4e\,(B-e) = 89.350{,}57\ kN$

If we analyze the action of the wind, for a unit wind load (w = 1 kN/m^2), we would have:

$W = B \cdot H \cdot w = 692{,}07$ Kn

Where we appreciate that the load due to its own weight in the oldest section is 129.10 times greater than that of the wind.

To analyze the overturning stability of this fraction of the tower, considered a rigid solid and subjected to a horizontal force (Figure 19) coinciding with the action induced by the wind (w = 1 kN/m2), we equate the moment induced by the wind and the moment due to its own weight, with respect to the pivot point at the bas

$$w \cdot B \cdot H\,\frac{H}{2} = H \cdot \gamma \cdot 4e\,(B-e)\,\frac{B}{2}$$

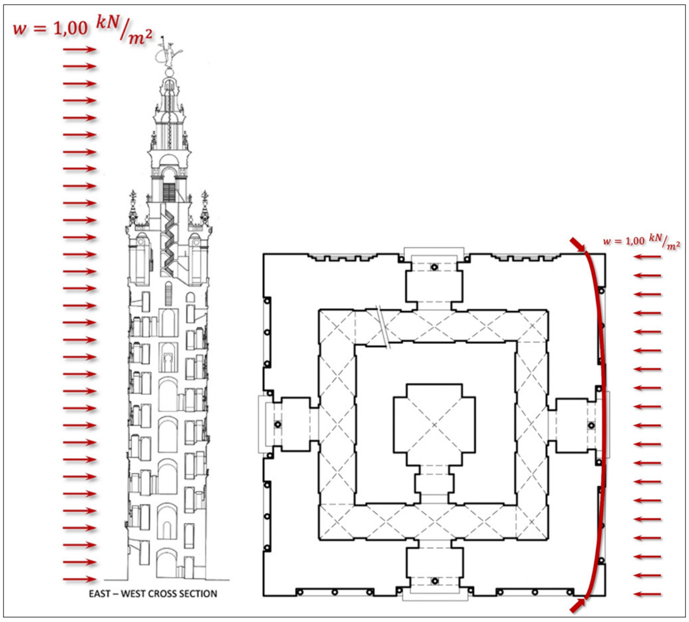

Figure 19: Frontal Wind Action and Plan Thrust Line (Graphics by the Author).

Therefore, the tangent of the angle obtained from the relationship between the action of the wind (W) and the weight (P) is:

$$\tan\frac{w}{P} = \tan\frac{692{,}07}{89.350{,}57} = 0{,}0077457$$

Tangent corresponding to the angle 0.4438º.

The slenderness ratio between height (H) and base (B) is,

$$\tan\frac{B}{H} = \tan\frac{13{,}61}{50{,}85} = 0{,}2742297$$

Tangent corresponding to the angle 15.33º.

This means that the combined action of its own weight + wind passes through the base of the old section of the bell tower at 0.20 m from the central axis, but within the central third (compression zone), with geometric safety for this design of 34.55 against rollover. This absence of traction justifies the absence of cracks.

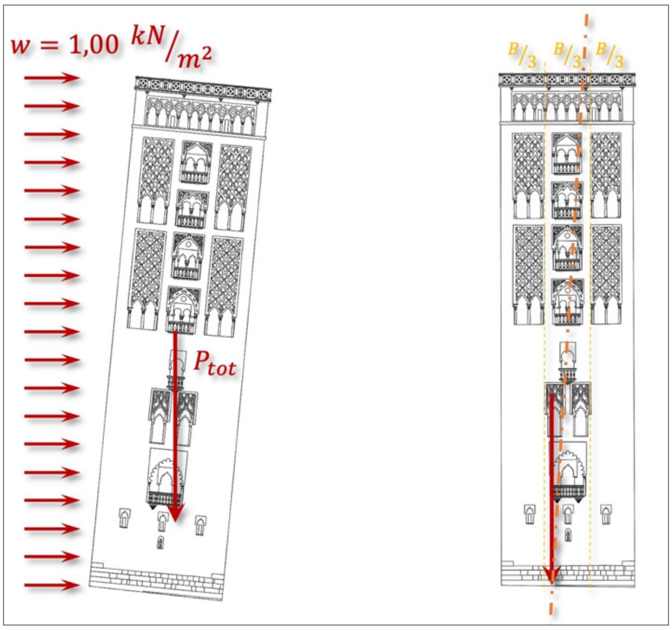

Figure 20: Frontal action of the wind and overturning as a rigid solid of the old section of the giralda bell tower (graphics by the author).

If we consider now the Giralda tower in total, old and new sections, the weight of the building is 1,74 • 10[5] kN and the force of the wind would be 978.55 kN. Therefore, it is verified that the load due to its own weight is much greater still (177.80 times) than that of the wind. Consequently, we can say that the force of the wind will not bring down the tower.

Global moment due to wind at the base:

$$M_w = 978{,}55 \cdot 27{,}25 = 26.665{,}49\ kN \cdot m$$

Stabilizing moment due to the weight of the tower:

$$M_{est} = 1{,}74 \cdot 10^5 \cdot \frac{13{,}61}{2} = 11{,}84\ \cdot 10^5\ kN \cdot$$

As the section is square, the total area of the façade is identical on the north and east flanks. Taking into account the value of the surface, the force of the wind would reach 978.55 kN.

In this way, we mark point A (Figure 21): point at which we can quantify the stress due to own weight and headwind, without the collaboration of intermediate floors.

However, the weight of the slabs influences, although they provide resistant collaboration. Therefore, we are going to calculate the tension in "A" due to its own weight and the frontal wind with the collaboration of intermediate floors. Thus, the intermediate slabs of the ramps participate in the response reaction to the wind stress: the inner and outer sheets of the wall transmit stress, making the tower work as a rigid solid. In response to the stress of the wind, a couple of forces appear to compensate the moment that produces the effect of the wind. This pair of forces is calculated analytically with the equilibrium equations (Figure 21).

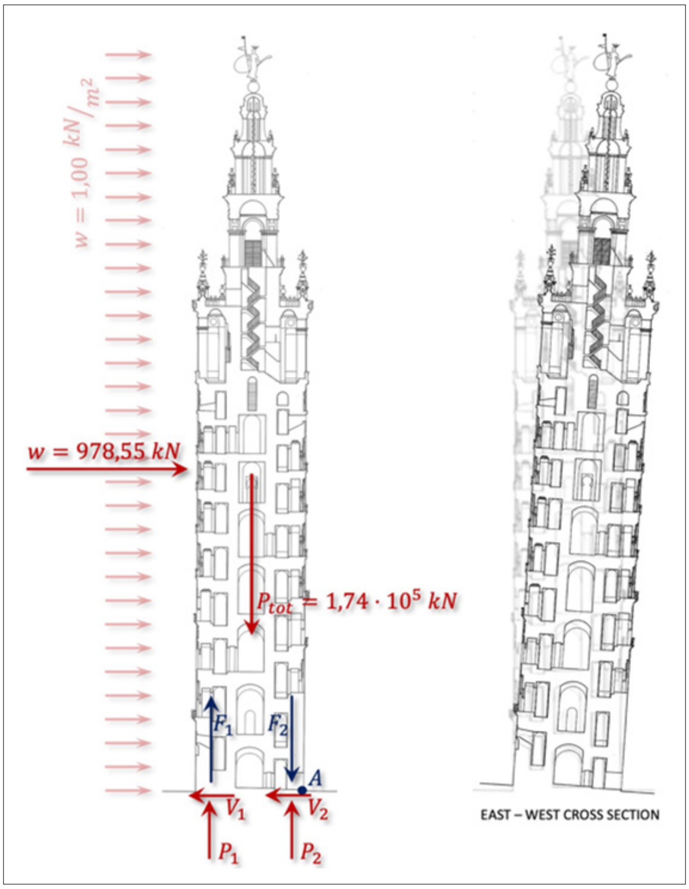

Figure 21: Forces Acting on the Section and Induced Deformations (Graphics by the Author).

We would start by analyzing the vertical forces:

$$\sum F_v = 0 \quad P_{tot} - F_1 - F_2 - P_1 - P_2 = 0$$

Since

$$P_{tot} = P_1 + P_2$$

Being the base of the square bell tower and the doubly symmetrical tower,

$$P_1 = P_2$$
$$P_{tot} = 2\,P_1 = 2\,P_2$$

So:

$$F_1 = -F_2$$

We continue with the analysis of horizontal forces:

$$\sum F_h = 0 \quad w - V_1 - V_2 = 0$$
$$w = V_1 + V_2$$

A Being the base of the square bell tower and the doubly symmetrical tower,

$$V_1 = V_2$$
$$w = 2\,P_1 = 2\,P_2$$

And, finally, we analyze the overturning moments

$$\sum M_A = 0 \quad P_{tot} \cdot dist_{\overline{P_{tot}A}} + w \cdot dist_{\overline{wA}} - P_1 \cdot dist_{\overline{P_2A}} - F_1 \cdot dist_{\overline{F_2A}} = 0$$

From here we deduce that

$$F_1 = -F_2 = \frac{P_{tot} \cdot dist_{\overline{P_{tot}A}} + w \cdot dist_{\overline{wA}} - P_2 \cdot dist_{\overline{P_2A}}}{dist_{\overline{F_2A}}}$$

$$F_1 = \frac{1{,}74 \cdot 10^5 \cdot 6{,}805 + 978{,}55 \cdot 27{,}25 - 489{,}275 \cdot 13{,}61}{13{,}61} = 84.551{,}47\ kN$$

Therefore the tension in the base "A":

$$S_1 = \frac{174.000 + 84.551{,}47}{15{,}00 \cdot 15{,}00} = \frac{258.551{,}47}{225{,}00} = 1.149{,}12\ {}^{kN}/_{m^2} < 1.250\ {}^{kN}/_{m^2}$$

Given that the bell tower has a height of 87.00 m (we discount El Giraldillo) and a base of 13.61 m x 13.61 m, its slenderness is:

$$\lambda = \frac{87{,}00}{13{,}61} = 6{,}39$$

To analyze the effect of the wind load on the tower, we have also resorted to the same finite element model as in the previous section. For this, a uniform and static wind load of 1 kN/m^2 was also considered. We remember the same caveats that we pointed out then: for wind, as was the case for self-weight, we do not need an analysis by the finite element method, except for that deformed by wind action. However, this analysis allows a better description.

The results of the wind resistance analysis show that the base of the tower is completely compressed, with a very low eccentricity of the total load and a geometric coefficient of safety against collapse of 10, which is a very high value. The maximum eccentricity can reach 210 centimeters (2,10 m, Figure 22 and Figure 23).

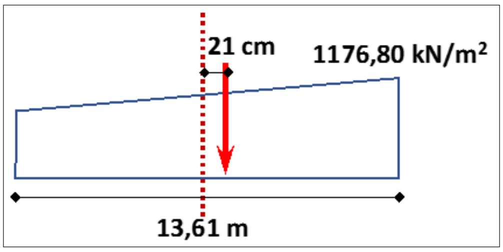

Figure 22: Stress Scheme at the Base and Eccentricity of the Resultant of the Loads, within the Central Core (Compression Zone) [23].

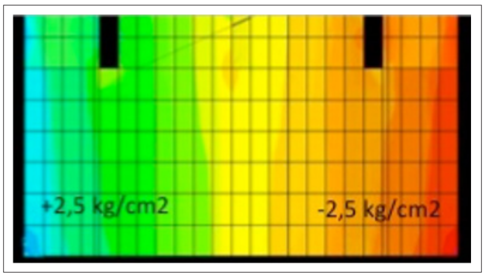

Figure 23: Stress Analysis on the Base, Before the Action of the Wind, According to the Model Developed by Doctor Cobreros Vime [23].

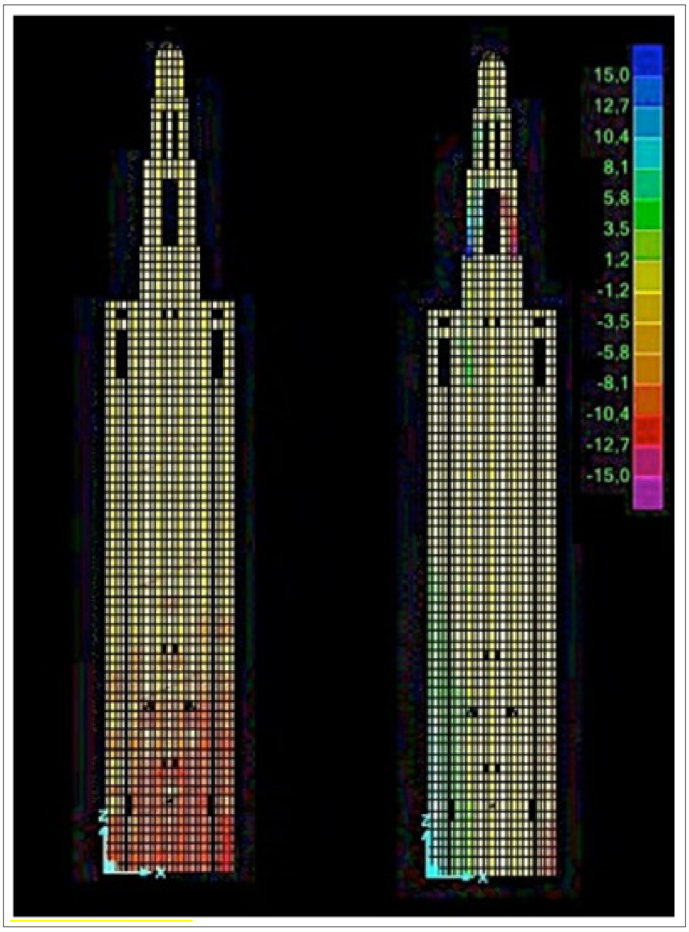

Figure 24: Tension Analysis of the Action of the Wind, According to the Model Developed by Doctor Cobreros Vime, for the Entire Set of the Bell Tower. Stress Values in the Vertical Direction Expressed in kg/cm^2, Combination of Own Weight and Wind (Left) and Stresses with Wind (Right) [23].

The stress level at the base is quite high, given the characteristics of the foundation that we analyzed in section 4 (Figure 24). The deformed shape of the bell tower (Figure 25) is 3 centimeters at the highest level of the bell body. This is a very small deformation compared with building collapse. It is also small compared to the level of H/500 that would bring the tower to the level close to global buckling (12 centimeters).

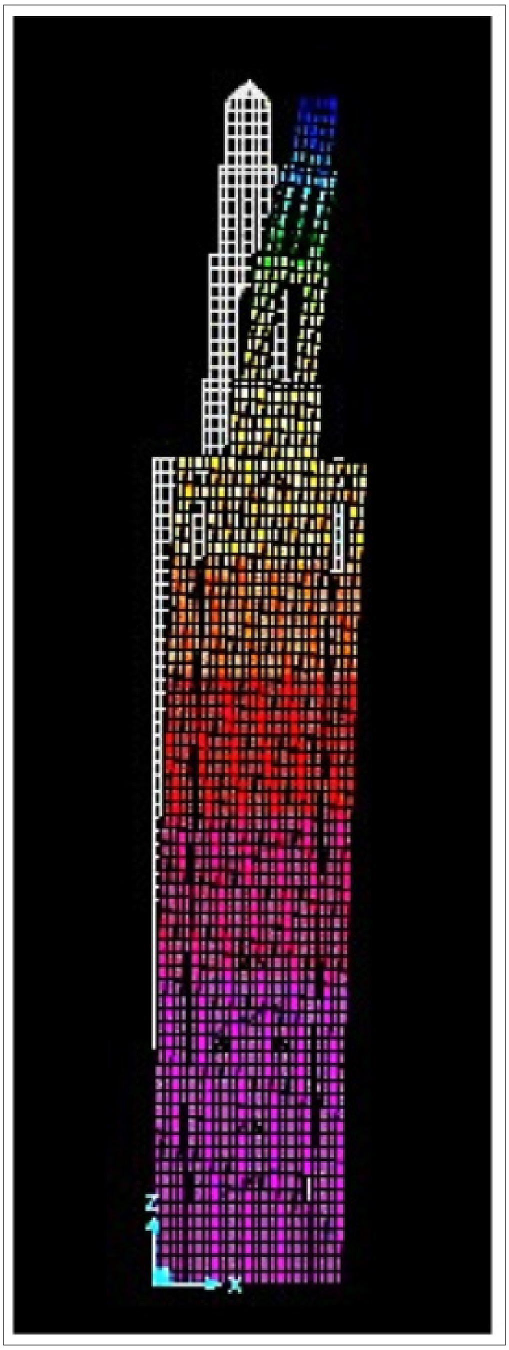

Figure 25: Deformed by the Action of the Wind of the Whole of the Bell Tower (Old Part And New Part), on the Model Developed by Doctor Cobreros Vime [23].

Earthquake Resistance Analysis
For the seismic analysis, modal spectral analysis will be used. Since the tower is fully compressed due to its own weight, we do not have cracking problems that would invalidate a linear model. Using a model with the Sap 2000/15 program, it was possible to obtain the envelope of deformed values and stresses. Doctor Cobreros Vime modified the sign of some data and ignored the shape of the deformed one. Thus, only the absolute value of the displacements was taken into account.

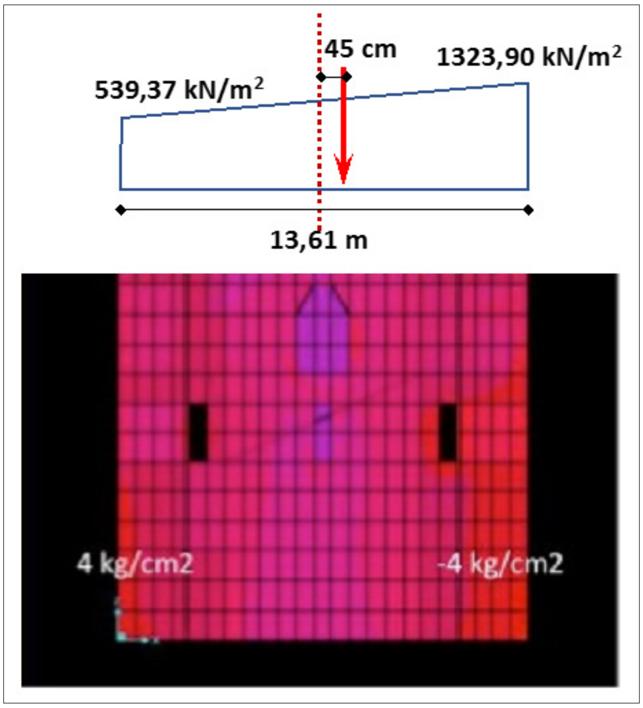

Figure 26: Stress Scheme at the Base and Eccentricity of the Resultant of the Loads. within the Central Nucleus, on the Model Developed by Doctor Cobreros Vime [23].

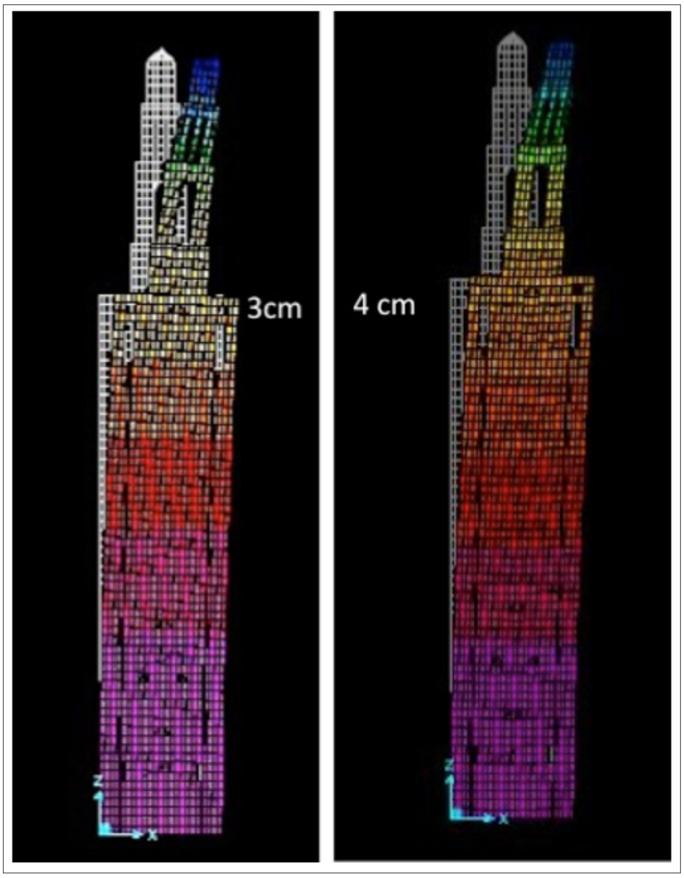

Figure 27: Deformed Before the Wind Load 1 Kn/M^2 and Deformed before the Earthquake, for a Basic Seismic Acceleration 0.07, on the Model Developed by Doctor Cobreros Vime [23].

The conclusion is that the earthquake affects something more than the wind to the tower [24]. In modern skyscrapers it normally happens the other way around, even in areas of high seismicity from 200 meters high and slenderness 3. The slenderness decreases the effect of the earthquake, the mass increases it. Because the Giralda bell tower is a massive tower, the earthquake effect is slightly bigger (Figure 27, right) than the wind effect (Figure 27, left), but both considerably less than the self-weight load. The risk of collapse due to an earthquake is greater than that due to wind, but the latter is more than offset by the own weight of the bell tower.

Conclusion

The Giralda Tower, as we can see it today in Seville, is the result of a Mudejar tower built in the 12th century and increased during the 16th century.

The analyzes carried out on the foundation of the bell tower have made it possible to verify that the foundation makes it possible for the tower to remain standing. However, an improvement of the ground in the foundation area was necessary to resist the pressure that the bell tower transmits to the ground.

The Giralda tower has resisted and resists very well the loads of its own weight, wind and earthquake. In spite of being a masonry structure, the brick perfectly resists the efforts to which it is subjected. The center of gravity of the double wall structure moves very little with the wind and earthquake, thanks to the high self-weight of the bell tower and its perimeter distribution.

The absence of fissures and cracks showed that the level of traction was low and that the compression was adequate. This aspect has been corroborated here numerically and by means of finite element models.

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