Wind and Earthquake Analysis of Spire of Cimborio of Barcelona Cathedral.
Master’s Thesis

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Wind and Earthquake Analysis of Spire of Cimborio of Barcelona Cathedral.

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University: Technical University of Catalonia (UPC), Spain
Date: 23rd July, 2009
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DEDICATION

To the current efforts of restoration of Spire of Cimborio of Barcelona Cathedral,

To my dear Prof. Pere Roca,

To my Parents,

To my Wife,
Wind and Earthquake Analysis of Spire of Cimborio of Barcelona Cathedral

Erasmus Mundus Programme

ADVANCED MASTERS IN STRUCTURAL ANALYSIS OF MONUMENTS AND HISTORICAL CONSTRUCTIONS
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Wind and Earthquake Analysis of Spire of Cimborio of Barcelona Cathedral
ABSTRACT

Wind and Earthquake Analysis of Spire of Cimborio of Barcelona Cathedral

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Barcelona Cathedral is one of the most important monuments not only in Spain but also all over the world. The construction of the Gothic cathedral started in 1298 under King Jaume II and in 1460 the main building was completed. The two architects Josep Oriol Mestres and August Font i Carreras completed the construction of the gothic façade in 1889 and the central spire in 1913, following the same design previously proposed by the French architect Charles Galters in 1408.

The central spire reaches a height of 90 m over ground level which makes it very vulnerable when subjected to lateral loads like wind and earthquakes. Being finished at the beginning of the 20th century (when the concept of reinforced concrete was being widely spread) gave the builders the chance to centrally reinforce all masonry beams of the spire with steel ties and nowadays these steel ties are facing very severe problems due to corrosion.

A complete project for restoration of the spire is being executed nowadays in which a complete dismantling and reconstruction will be carried out. The steel ties will be replaced with titanium ones in order to eliminate the corrosion problem.

In order to understand wind and seismic performance of the spire and the role and strength contributions of the steel ties, the different applied loads on the spire which are self weight, wind loads and earthquake loads have been estimated, then a numerical model of the spire has been created and analyzed using the finite element program DIANA. First a linear elastic analysis under the effect of spire self weight then a combination of spire self weight and wind loads and finally a combination of spire self weight and earthquake loads. The high tensile stresses in masonry beams under the effect of the combination of spire self weight and wind loads and the combination of spire self weight and earthquake loads meant that linear elastic analysis wasn't enough to describe the structure behavior and a nonlinear analysis was essential.

A nonlinear analysis under the effect of spire self weight (using three different constitutive models to describe masonry nonlinear behavior) was investigated and it revealed an elevated safety margin as the spire can carry more than ten times its self weight.

Then to investigate the seismic performance of the spire a nonlinear static pushover analysis (using two different constitutive models) has been carried out. As a conclusion of this study the steel ties are highly needed to carry the tensile stresses resulted from seismic actions and the spire would be able to resist a maximum base shear of 420 KN (16% of the spire self weight).
RESUMEN

Análisis del Efecto del Viento y de un Terremoto sobre la Aguja del Cimborio de la Catedral de Barcelona

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La Catedral de Barcelona es uno de los monumentos más importantes no sólo en España sino también en todo el mundo. La construcción de la Catedral gótica empezó en 1298 bajo el reinado de Jaume II y en 1460 se completó la construcción del edificio principal. Los dos arquitectos Joseph Oriol Mestres y August Font y Carreras completaron la construcción de la fachada gótica en 1889 y de la aguja central en 1913, siguiendo el mismo diseño propuesto anteriormente por el arquitecto francés Charles Galters en 1408.

La aguja central alcanza una altura de 90 m sobre el nivel del suelo la cual la hace muy vulnerable cuando está sujeta a cargas laterales tales como el viento y los terremotos. Terminada a principios del siglo 20 (cuando el concepto del cemento reforzado estaba siendo ampliamente extendido) dio a los constructores la oportunidad de reforzar toda la mampostería de las vigas de la aguja con tirantes de acero y hoy en día estos tirantes de acero se enfrentan a serios problemas debido a la corrosión.

Un proyecto completo de restauración de la aguja está siendo ejecutado hoy en día, en el que se está llevando a cabo su desmantelamiento y reconstrucción. Los tirantes de acero serán reemplazados por otros de titanio para eliminar el problema de la corrosión.

Para entender la actuación del viento y de los movimientos sísmicos sobre la aguja y el papel que desempeñan la fuerza sobre los tirantes de acero, las diferentes cargas aplicadas por el propio peso de la aguja, las cargas debidas al viento y a un terremoto se ha hecho una estimación, por lo que se ha creado un modelo numérico de la aguja y se ha analizado usando el programa de elementos finitos DIANA. Primero se ha realizado un análisis de elasticidad lineal del efecto del propio peso de la aguja, luego una combinación del propio peso de la aguja y las cargas debidas al viento y finalmente una combinación del propio peso de la aguja y las cargas debidas a un terremoto. Las elevadas fuerzas de tensión sobre la mampostería de las vigas debidas a la combinación del propio peso de la aguja junto con las cargas debidas al viento y del propio peso dela aguja junto a las cargas debidas a un terremoto muestran que un análisis de elasticidad lineal no es suficiente para describir el comportamiento de la estructura por lo que es necesario realizar un análisis no lineal.

Se investigó mediante un análisis no lineal el efecto del propio peso de la aguja (usando tres modelos diferentes constitutivos para describir el comportamiento no lineal de la mampostería) y reveló un elevado margen de seguridad en el que la aguja puede aguantar más de diez veces su propio peso.

Luego para investigar la actuación sísmica sobre la aguja se ha llevado a cabo un análisis no lineal de derribo estático (usando dos modelos constitutivos diferentes). Como conclusión de este estudio lo tirantes de acero son muy necesarios para aguantar las fuerzas de tensión que resultan de acciones sísmicas de modo que la aguja sería capaz de resistir un máximo de corte en la base (base shear) de 420 KN (16% del propio peso de la aguja).
عنوان البحث: التحليل الإنشائي للبرج الأوسط لكاتدرائية برشلونة تحت تأثير أحمال الرياح والزلزال

الباحث: أحمد اليممى على محمد

المشرف: برا روكا فابريرجاد

ملخص البحث

تعد كاتدرائية برشلونة من أهم الآثار ليس فقط على مستوى إسبانيا ولكن أيضا على مستوى العالم. بدأ بناء الكاتدرائية القوطية في عام 1298 م في عهد الملك (جوما الثاني) و في عام 1460 م تم الانتهاء من المبنى الرئيسي و في عام 1488 م تم الانتهاء من المباني الإضافية. بناء الواجهة القوطية و في عام 1913 م. أما بناي البارلوج الأوسط من نوع من التصميمات التي أقربها المعماري الفرنسي (تشارلز جارنترز) منذ عام 1408 م.

يعرض البحج الأوسط لمشاهد سريعة عند تعرضه للأعمال العرضية مثل أحمال الرياح والزلزال نظراً لأن ارتفاعه يصل إلى 90 متراً فوق مستوى سطح الأرض و نظرًا لأنه بناء هذا البارلوج في بداية القرن العشرين (عندما كانت مبادئ الخرسانة المسلحة أخذة في الانتشار على مستوى العالم). فقد مكن ذلك بناؤهم من وضع تصميم مركزى جيد عن شدات من الصلب في جميع كميات الرياح و تتعرض هذه الشدات في وقتنا الحالي لمشكلات خطيرة نتيجة تعرضها للصدأ.

يتم الآن تنفيذ مشروع تتكون للفج الأوسط و سيتم فيه افتتاحية ترجمة البارلوج بأكمله و كذلك سيتم استبدال أجهزة الصلبة بسلاك مجموعة من التشاميس للتصبح عليه من مشكلة السدأ.

لفهم كيفية تصرف البارلوج تحت تأثير أحمال الرياح والزلزال و كذلك دور الشدات البيانية في مقاومة الإجهاد تم تقديم الأحمال المختلفة المؤثرة على البارلوج وهي وزنه الذاتي و أحمال الرياح وأحمال الزلزال. ثم تم عمل مسح ليتم لتحليل هيئة باستخدام طريقة العناصر المحددة باستخدام برنامج (سيمبن). خلايا البداية تم عمل تحليل خطي مرن تحت تأثير الوزن الذاتي وحده ثم تحت تأثير تجميع الوزن الذاتي و أحمال الرياح وأحمال الزلزال. و نظرًا لوجود الإجهاد متناوبة في الكميات الحربية تحت تأثير تجميع الوزن الذاتي و أحمال الرياح و كذلك تحت تأثير تجميع الوزن الذاتي و أحمال الزلزال فإن التحليل الإنشائي المرن الخطي لم يكن كافياً لوصف سلاك المشابع و كانت هناك حاجة لعمل تحليل إنشائي غير خطي.

تم عمل تحليل إنشائي غير خطي تحت تأثير الوزن الذاتي (تم اختبار ثلاثة نماذج كوبونية مختلفة لوصف السلاك غير الخطي للحجر) و أثبتت النتائج أن البارلوج أم عن تحت تأثير عشرة أمتار وزنه الذاتي.

لفهم السلاك الزلزالي للبارلوج تم عمل تحليل استاتجري غير خطي بعمومطية بوساوية (تم اختبار نماذج كوبونية مختلف فوصف السلاك غير الخطي للحجر) و كنتيجة لهذه الدراسة تم توضيح الحاجة الماسة لوجود الشدات لحمل إجهاد الصلبة الناتجة من أحمال الزلزال وأن البارلوج قادر على تحمل قوة قص أقصى عند الأساس تبلغ 420 كيلونيوترون (16% من وزنه الذاتي).
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Chapter 1

History and Description of Barcelona Cathedral and Its Spire

1-1 Introduction

This chapter will discuss the historical, architectural, structural and geometrical aspects of Barcelona cathedral and its spire of cimborio.

1-2 History of Construction of Barcelona Cathedral

In the center of the Barri Gòtic (Gothic district), the heart of Barcelona, is the gothic cathedral, known as La Seu. The cathedral is officially named Cathedral de la Santa Creu i Santa Eulalia (Cathedral of the Holy Crucifix and of Saint Eulalia), after Barcelona's patron saint Eulalia [http://www.aviewoncities.com, visited on April 10, 2009].

Figure 1-1: Barcelona Cathedral Location (From Google Earth)

The most distant origins of the Cathedral of Barcelona correspond to a basilica with three naves which was destroyed by Al-Mansur 925. The remains of this basilica can be seen in the City History Museum. Around 1046, a new cathedral was commenced at the initiative of Bishop Guislabert. We have few references to this building: it is believed to have occupied a part of the Gothic building, but some of its Romanesque elements remain [http://www.bcn.es, visited on April 10, 2009]. It was replaced by a Roman cathedral, built between 1046 and 1058.

A Roman chapel, the Capella de Santa Llucia, was added between 1257 and 1268. It was later incorporated in the cloister next to the cathedral. 30 Years later, in 1298, construction of the gothic cathedral started under King Jaume II, known as 'the Just'. During the construction of the gothic cathedral, the existing roman building was demolished except for the Santa Llucia chapel. As usual, the apse was constructed first, being finished in 1327, while the construction of the entire nave continued until 1417. In 1422 work stopped, leaving the cimborio unfinished and a provisional wall closure as a facade. The central nave of Barcelona Cathedral, built during years 1298
to 1422 [Computer analysis of a Gothic cathedral by Pere Roca and Climent Molins, 2000].

Due to civil wars and the Black Death which hit the city several times, the construction only progressed slowly. It took until 1460 before the main building was completed. The gothic facade was finished much later, in 1889 and the last part, the central spire, was completed in 1913. The design of both the facade and the spire were based on the original design from 1408 by the French architect Charles Galters. Even though it took 453 years the original design was followed by the architects Josep Oriol Mestres and August Font i Carreras [http://www.barcelonayellow.com, visited on April 4, 2009].

Figure 1-2: Barcelona Cathedral (http://www.catedralbcn.org)
1-3 Architectural and Structural Aspects of Barcelona Cathedral

![Plan of Barcelona Cathedral](http://z.about.com/d/architecture)

Figure 1-3: Plan of Barcelona Cathedral (http://z.about.com/d/architecture)

1-3-1 Introduction

A blending of medieval and Renaissance styles, Barcelona's cathedral features large bell towers covered in Gothic pinnacles, high Gothic arches, a handsomely sculptured choir and many side chapels with rich altarpieces. The interior was recently cleaned. Especially notable is the Cappella de Sant Benet behind the altar, with a magnificent 15th century interpretation of the crucifixion by Bernat Matorell [http://www.sacred-destinations.com, visited on April, 4 2009]. The church is 93 m long and 40 m wide. The octagonal clock towers reach a height of more than 50m. The spire of the central tower reaches a height of 90 m [http://www.barcelonayellow.com, visited on April, 4 2009].

1-3-2 The Plan

The plan of the temple shows a very unconventional lay-out in which the cimborio is not erected over the crossing but over the first bay of the nave close to the façade, while the crossing is delimited by the two majestic clock towers, more commonly part of the façade. A similar distribution can be observed in very few cases, such as the German cathedrals of Ulm and Friburg [Computer analysis of a Gothic cathedral by Pere Roca and Climent Molins, 2000].

1-3-4 The Entrance
The entrance arch is a set of concentric curves within one another, giving the structure an appearance that is remarkable even among Gothic churches. Though the design is used elsewhere, it is hardly ever done with such care and grace [http://ezinearticles.com, visited on April, 4 2009].

The construction of Barcelona Cathedral was detained at the beginning of 15th century, after the nave was completed, leaving the main façade and the cimborio unfinished for more than five centuries. The cimborio stood during this time with only the low spandrels of the tambour constructed and a provisory traditional tiled roof mounted in place of the upper vaulting. The definitive cimborio was designed and constructed at the beginning of 20th century by the cathedral architect August Font,
who envisaged and actually built a 50 m high spire of stone tracery. No major damage on the structure has been observed which can be related to the construction of this cimborio.

The cimborio of Barcelona Cathedral is not built over the crossing piers but, unconventionally, over the bay closest to the main façade. In that bay, the medieval builders’ laid-out more robust piers, arches and buttresses -with larger cross-sections compared to those in rest of the nave- so that the additional weight and thrust could be adequately resisted. However, it seems that the original intention was to erect a two-storey flat-roofed cimborio, with no spire, similar to that of Valencia Cathedral. The cimborio actually built reaches a height of 90 m and has a total estimated dead load of 19 MN.

Since it is significantly more actual capacity of the structure to adequately carry it. Moreover, the arches where the cimborio had to be built showed significant deformation prior to its construction. For these reasons, architect August Font actually erected the cimborio on a new system of robust stone arches, markedly pointed, built over the original ones, while leaving these uncharged. No architectural measure was taken to alleviate the work of the piers in supporting the total load, or that of the arches and vaults of the aisles and close nave bays in buttressing the cimborio. However, no external signs are apparent of any structural disorder but for some repaired cracks which can be recognized in the original arches [Computer analysis of a Gothic cathedral by Pere Roca and Climent Molins, 2000].

![Figure 1-7: The Cimborio Supported on New Arches [From Pere Roca]](image)

1-3-6 The Interior

The interior is equally impressive. The visitor is struck on entering by the amazing Woodcarvings on every wall. A side chapel holds a cross-removed from a galleon that participated in the Battle of Lepanto in 1571. The Christ form on the cross is leaning to the right. As stated by a Spanish tale, the movement of the Christ form was to avoid a cannonball during that battle. Other chapels within the massive cathedral tell similarly extraordinary and appealing stories.
The Cappella de Sant Benet back of the cathedral altar houses a crucifix from the 15th century. This work of art is not instantly visible to the informal spectator, but those who want to see the whole lot, their effort will be well rewarded [http://ezinearticles.com, visited on April, 4 2009].

One side chapel is dedicated to "Christ of Lepanto", and contains a cross from a ship that fought at the Battle of Lepanto (1571). The body of the cross is shifted to the right. Catalan legend says that the body swerved to avoid being hit by a cannonball. This is believed to have been a sign from God that the Ottomans would be defeated [http://en.wikipedia.org visited on April, 10 2009].

1-3-7 Naves and Side Aisles

The central nave of Barcelona Cathedral, built during years 1298 to 1422, spans 12.80 m and has a maximum high of 25.6 m. The span of the side aisles is equal to one half the span of the nave. The building includes three naves (the nave and two aisles) although, as a consequence of its particular design, it appears to enclose two additional aisles. This particular effect is caused by the inclusion of the buttresses in the interior space and their use to laterally confine the side chapels. The space between the buttresses is vaulted at an intermediate level, to form a tribune across the side aisles. The upper vaulting between the buttresses stands at the same height as the aisle vaulting and gives the impression of being an extension of the former.

The existing flying arches are not structural but pure draining devices as can be understood by observing their very reduced section and inadequate geometry. In fact, the thrust caused by the vaults and arches of the main nave are transferred to the buttresses by the vaults and arches of the lateral naves, which are strategically placed to undertake the structural role of actual flying arches. This structural feature of Barcelona Cathedral can be observed only in a few other Gothic churches (as the Basilica of Sta. Maria del Mar, also in Barcelona) [http://ezinearticles.com, visited on April, 4 2009].
1-3-8 The Crypt

The creepy and the kingly mix in the Crypt of Saint Eulàlia, with its red interior, burning torches, towering pillars, stone angels, and immense altar. This historic site is where Saint Eulàlia’s remains are kept, guarded by the holy icons surrounding the tomb.

Figure 1-10: Crypt containing the tomb of Santa Eulalia

1-3-9 The Cloister
Probably the best part of the cathedral is the 14th century cloister, which the historian Cirici called "the loveliest oasis in Barcelona." Its vaulted galleries overlook a lush garden filled with orange, medlar and palm trees and a mossy central pond. Underneath the well-worn slabs of its stone floor are the tombs of key members of the Barri Gòtic’s ancient guilds [http://www.sacred-destinations.com/spain/barcelona-cathedral-la visited on April, 4 2009].

There are always 13 geese in its central courtyard. Each goose represents one year in the life of the martyr Santa Eulalia, a young girl tortured to death in the 4th century by the Romans for her religion. The cloister also contains a small museum with liturgist artifacts [http://www.aviewoncities.com visited on April, 10 2009]. The birds have their own swimming pond, surrounded by gargoyles, dragon, and frog statues [http://travel.excite.co.uk, visited on April, 9 2009].

1-4 The Spire

1-4-1 Introduction

A spire is a tapering conical or pyramidal structure on the top of a building, particularly a church tower. Etymologically, the word is derived from Anglo-Saxon, so it is related to "spear," rather than the Romance languages and "spirit."

Symbolically, spires have two functions. The first is to proclaim a martial power. A spire, with its reminiscence of the spear point, gives the impression of strength. The second is to reach up toward the skies. The celestial and hopeful gesture of the spire is one reason for its association with religious buildings. A spire on a church or cathedral is not just a symbol of piety, but is often seen as a symbol of the wealth and prestige of the order, or patron who commissioned the building.

As an architectural ornament, spires are most consistently found on Christian churches, where they replace the steeple. Although any denomination may choose to use a spire instead of a steeple, the lack of a cross on the structure is more common in Roman Catholic and other pre-Reformation churches.
The battlements of cathedrals featured multiple spires in the Gothic style (in imitation of the secular military fortress).

Currently, the largest spire to be part of the architecture of another building is the spire mounted on the recently completed Q1 residential tower on the Gold Coast in Australia.

![Figure 1-13: The spire of Salisbury Cathedral in England, 123 meters tall](http://en.wikipedia.org)

Spires are also common and notable as solo structures. After contact with Egyptian architecture and the mania for Egyptian artifacts in the west in the 19th century, towers in the shape of obelisks enjoyed a vogue. When original obelisks could be imported, such as Cleopatra's needles in London and New York City, they were, but spires as memorial structures were popular in funerary architecture and public monuments (e.g. the Washington monument in Washington, DC) into the 20th century. In the Modernist movements of the 20th century, office towers in the form of free-standing spires also began to be built.

Some famous buildings, such as the Space Needle in Seattle, Washington, use the spire as a testimony of civic power and hope; in the case of this example, it is also a reference to Seattle's participation in aerospace. A 1,776-foot (541-m) "Freedom Tower" is a projected feature of the 9/11 Memorial in New York City, and is to be topped by a spire [http://en.wikipedia.org, visited on April 10, 2009]

1-4-2 Gothic Spires

A spire declared the presence of the gothic church at a distance and advertised its connection to heaven. The tall, slender pyramidal twelfth century spire on the south tower Chartres Cathedral is one of the earliest spires. Openwork spires were an astounding architectural innovation, beginning with the early fourteenth-century spire at Freiburg cathedral, in which the pierced stonework was held together by iron cramps. The openwork spire, according to Robert Bork, represents a "radical but logical extension of the Gothic tendency towards skeletal structure." The organic skeletons of Antoni Gaudi's phenomenal spires at the Sagrada Familia in Barcelona represent an outgrowth of this Gothic tendency. Designed and begun by Gaudi in 1884, they were not completed until the 20th century.
In England, "spire" immediately brings to mind Salisbury Cathedral. Its 403-foot (123-m) spire, built between 1320 and 1380, is the tallest of the period anywhere in the world, and in its way is as remarkable as the Coliseum in Rome or the Parthenon in Athens. A similar but slightly smaller spire was built at Leighton Buzzard in Bedfordshire, England, which indicates the popularity of the spire spreading across the country during this period. We will never know the true popularity of the medieval spire, as many more collapsed within a few years of building than ever survived to be recorded. In the United Kingdom spires generally tend to be reserved for ecclesiastical building, with the exception to this rule being the spire at Burghley House, built for Elizabeth I's Lord Chancellor in 1585.

In the early Renaissance the spire was not restricted to the United Kingdom: the fashion spread across Europe. In Antwerp the 123 m spire was the tallest structure in the Low Countries for over five centuries. Between 1221 and 1457 richly decorated open spires were built for the Cathedral of Burgos in Spain, while at Ulm Cathedral in Germany the 529-foot (161-m) spire was built in the imported French Gothic style between 1377 and 1417.

Interestingly, the Italians never really embraced the spire as an architectural feature, preferring the classical styles. The gothic style was a feature of Germanic northern Europe and was never to the Italian taste, and the few gothic buildings in Italy always seem incongruous.

The blend of the classical styles with a spire occurred much later. In 1822, in London, John Nash built All Souls’ Church, Langham Place, a circular classical temple, with Ionic columns surmounted by a spire supported by Corinthian columns. Whether this is a happy marriage of styles or a rough admixture is a question of individual taste.

During the 19th century the Gothic revival knew no bounds. With advances in technology, steel production, and building techniques the spire enjoyed an unprecedented surge through architecture, Cologne Cathedral's famous spires, designed centuries earlier, were finally completed in this era.

Figure 1-14: The spires of Antoni Gaudi’s Sagrada Familia church, still under construction in Barcelona (http://www.greatbuildings.com)
Spires have never really fallen out of fashion. In the twentieth century reinforced concrete offered new possibilities for openwork spires [http://en.wikipedia.org, visited on April 10, 2009]

![Image of the Spire of Dublin](http://en.wikipedia.org)

Figure 1-15: The Spire of Dublin is an example of the use of spires in a modern context (http://en.wikipedia.org)

1-4-3 Architectural and Geometrical Description of Barcelona Cathedral Spire

The spire of Barcelona cathedral reaches a height of 90 above the ground level. It consists at base from 8 arches forming an octagonal of 67 cm width and 2.36 meters typical side length as shown in figure 00-00, and then the eight columns at corners are rising up to the top of the spire until they are attached together forming a base for a stone sculpture which has a height of 5 meters and a weight of 62.5 KN. To prevent the buckling of stone masonry columns and to increase the rigidity of the spire an

![Plan View of the Spire](image)

Figure 1-16: Plan View of the Spire at Level 43.73 (from Pere Roca)
octagonal closed masonry beam is repeated at different levels along the spire height. These beams in average have a width of 25 cm and a depth of 60 cm as shown in figure 00-00.

Figure 1-17: Elevation of the Spire Showing Columns, Beams and Sculpture (from Pere Roca)
Chapter 2

Damages in Spire and Current Restoration Project

2-1 Introduction

This chapter will discuss in brief the damages of the spire of cimborio of Barcelona Cathedral and its reasons then the current restoration project of the spire.

2-2 Damages and Its Reasons of the Spire and the Pinnacles

The spire and its pinnacles are suffering from obvious damage evidences which can be categorized under the following titles:

1-Cracks due to corrosion of steel ties
2-Partial collapse of some parts of pinnacles
3-Discoloration

2-2-1 Cracks Due to Corrosion of Steel Ties

The steel ties inside the masonry beams of the spire are suffering from sever corrosion as shown in figure 2-1.

Due to corrosion products the steel ties increase in volume (the volumetric increase may reach ten times the original volume) which causes pressure on the surrounding stone masonry and leads it to crack as shown in figure 2-2 or furthermore to fall down as shown in figure 2-3.

Figure 2-1: Corroded Steel Ties Extracted from Spire
Big parts of some of the stone masonry beams are about to fail due to corrosion products and they are temporary kept in position by some ropes until they will be completely dismantled and reconstructed as shown in figure 2-4.
2-2-2 Partial Collapse of Some Parts of Pinnacles

The construction technique used in pinnacles is to attach the last top part of the pinnacle to the rest of its structure by making a hole in both parts and connecting them using a vertical connector as shown in figure 2-5. This technique of connection proved to be inefficient especially under the effect of lateral loads and it wasn't able to prevent the collapses occurred as shown in figure 2-6. Another way of connecting was used in which the mortar was replaced with epoxy resin, also this way wasn't efficient because of the inaccurate application of epoxy resin and again another collapse occurred as shown in figure 2-7.
2-2-3 Salt Crystallization and Discoloration

Due to many factors like rains, pollution and sea spray salts and dust transfer to the spire and pinnacles cause the stone to acquire an altered appearance in the form of discoloration and salt deposits as shown in figure 2-8. Changes in relative humidity make salts crystallize, hydrate and dissolve; this repetitive cycle leads to the destruction of the material due to the forces resulting from the crystallization of the salts in the pores of the material [SAHC, SA6, Lecture 6 by Rui Miguel and Carlos Alves].
2-3 Current Restoration Project

Nowadays a complete restoration project of the spire is being carried out in which the spire and its pinnacles will be completely dismantled and reconstructed. As a first step in any restoration work a complete documentation should be carried out so a complete laser scanning of the cimborio and the spire have been carried out and figure 2-11 shows an example of these scans.
In order to execute restoration works complete scaffolding starts from the roof level and extend to the last top part of the spire is used as shown in figure 2-12.

Another complete steel structure has been constructed inside the spire as shown in figure 2-13 (a), (b) and (c). This steel structure will be used as scaffolding for the reconstruction process.
Figure 2-13 (a): Inside the Steel Structure (Looking Up)

Figure 2-13 (b): The Columns of the Steel Structure and Its Connections with the Spire
The different parts of the spire and the pinnacles are being numbered then being cut at the locations of mortar joints using an electrical saw; finally they will be stored on the roof of the cathedral as shown in figure 2-14 (a) and (b) in order to reconstruct them again after cleaning processes.
In the reconstruction process and to eliminate the current problem of steel ties corrosion, all ties will be from titanium.
Chapter 3

Description of Numerical Model and Properties of Materials

3-1 Introduction

This chapter will discuss the description of the used numerical model regarding the types of used elements, the mesh size, the ID's given to beams and columns of the spire to be easily referred to, the approximation of cross sections. Also it will discuss the used properties of materials and the used constitutive models to describe the nonlinear behavior of masonry and steel.

3-2 Description of Finite Element Model

3-2-1 Beam Elements Used in Modeling of Beams and Columns of Spire:

The beam elements used are Class II. The class-II beam elements in DIANA are numerically integrated over their cross section and along their axis therefore these elements may be used in geometrical and physical nonlinear analysis. The class-II beam elements are based on the Bernoulli theory which does not take shear deformation into account and assumes that the cross-sections remain plane and perpendicular to the slope of the beam axis.

The type of Class II beam elements used is L13BE which have the following characteristics: The L13BE element [Fig. 3-1] is a two-node, three-dimensional class-II beam element. Basic variables are the translations $u_x$, $u_y$ and $u_z$ and the rotations $\varphi_x$, $\varphi_y$ and $\varphi_z$ in the nodes. An additional variable is the elongation $\Delta u_x$ [DIANA 9.1 user's Manuel, Element Library, P.86 and 87].

![Figure 3-1:L13BE](image)

3-2-2 Flat Shells Used in Modeling of Arches and Last Attached Part of Columns:

The arches were modeled using flat shell elements which have the following characteristics: Flat shell elements basically are a combination of plane stress elements and plate bending elements. But unlike the plane stress elements, the basic variables are forces rather than Cauchy stresses. Flat shell elements must fulfill the following conditions with respect to shape and loading [Fig. 3-2].
Flat shell elements are characterized by the following facts. The normals of the element plane remain straight after the deformation, but by definition, they do not have to be perpendicular to the element plane. The displacement perpendicular to the plane does not vary in the direction of the thickness.

The flat shell elements in DIANA basically are combinations of a plane stress element and a plate bending element and there is no coupling between membrane and bending behavior. Generally the membrane behavior is conform its corresponding plane stress element except the primary stresses which are defined in terms of moments and forces rather than Cauchy stresses. [DIANA 9.1 user's Manuel, Element Library, P.233 and 234].

The used type of shell elements used to model arches is T15SF which has the following characteristics:

The T15SF element [Fig. 3-3] is a three-node triangular isoparametric flat shell element. The plate bending is according to the Mindlin–Reissner theory with an adapted transverse shear interpolation.
The Q20SF element [Fig. 3-4] is a four-node quadrilateral isoparametric flat shell element. The plate bending is according to the Mindlin–Reissner theory with an adapted transverse shear interpolation and is conform the Q12PL plate bending element. The membrane behavior is conform the Q8MEM plane stress element. [DIANA 9.1 user's Manuel, Element Library, P.252 and 253].

3-2-3 Mesh Description

The meshing of the finite element model is based on maximum element size of 0.1 m and this yield in having a model consisting of 4011 nodes and 5055 elements as shown in figure 3-5.
The used algorithm used for meshing the arches is Delaunay algorithm as shown in figure 3-6 and for the last attached part of columns mapped algorithm is used as shown in figure 3-7.

Figure 3-6: Delaunay Algorithm  
Figure 3-7: Mapped Algorithm

3-3 The Used Identification of Beams and Columns

To facilitate the process of referring to any element of the spire the beams and columns were given some ID's as shown in figure 3-8 and figure 3-9.

Figure 3-8: Beams ID's
3-4 Approximation of cross sections of Beams and Columns

The used cross sections of beams and columns are an approximation of the real cross sections as following:
### ACTUAL SECTIONS OF BEAMS

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<tr>
<th>Sec. Name</th>
<th>LEVEL</th>
<th>Shape</th>
<th>PROPERTIES OF AREA</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>A (m²)</td>
</tr>
<tr>
<td>B1</td>
<td>45.765</td>
<td>0.6474</td>
<td>0.0419</td>
</tr>
<tr>
<td>B2</td>
<td>47.220</td>
<td>0.0418</td>
<td>0.0002</td>
</tr>
<tr>
<td>B3</td>
<td>50.272</td>
<td>0.1051</td>
<td>0.0017</td>
</tr>
<tr>
<td>B4</td>
<td>52.490</td>
<td>0.1028</td>
<td>0.0016</td>
</tr>
<tr>
<td>B5</td>
<td>54.811</td>
<td>0.1137</td>
<td>0.0022</td>
</tr>
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<td>57.100</td>
<td>0.1237</td>
<td>0.0027</td>
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<tr>
<td>B9</td>
<td>63.677</td>
<td>0.0366</td>
<td>0.0001</td>
</tr>
</tbody>
</table>

Table 3-1: Actual Properties of Area of Beams

### APPROXIMATED SECTIONS OF BEAMS

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<th>LEVEL</th>
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<td>B4</td>
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</tr>
<tr>
<td>B6</td>
<td>57.100</td>
<td>0.60</td>
<td>0.204</td>
</tr>
<tr>
<td>B7</td>
<td>59.230</td>
<td>0.55</td>
<td>0.204</td>
</tr>
<tr>
<td>B8</td>
<td>61.560</td>
<td>0.69</td>
<td>0.203</td>
</tr>
<tr>
<td>B9</td>
<td>63.677</td>
<td>0.25</td>
<td>0.14</td>
</tr>
</tbody>
</table>

Table 3-2: Approximated Properties of Area of Beams
## APPROXIMATED SECTIONS OF COLUMNS

<table>
<thead>
<tr>
<th>Sec. Name</th>
<th>LEVEL</th>
<th>SECTION</th>
<th>PROPERTIES OF AREA</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>FROM</td>
<td>TO</td>
<td>$A \ (m^2)$</td>
</tr>
<tr>
<td>C1</td>
<td>42.181</td>
<td>43.730</td>
<td>0.7022</td>
</tr>
<tr>
<td>C2</td>
<td>45.765</td>
<td>50.272</td>
<td>0.2778</td>
</tr>
<tr>
<td>C3</td>
<td>50.272</td>
<td>52.490</td>
<td>0.2494</td>
</tr>
<tr>
<td>C4</td>
<td>52.490</td>
<td>54.811</td>
<td>0.2494</td>
</tr>
<tr>
<td>C5</td>
<td>54.811</td>
<td>57.100</td>
<td>0.3461</td>
</tr>
<tr>
<td>C6</td>
<td>57.100</td>
<td>59.230</td>
<td>0.2778</td>
</tr>
<tr>
<td>C7</td>
<td>59.230</td>
<td>61.560</td>
<td>0.2313</td>
</tr>
<tr>
<td>C8</td>
<td>61.560</td>
<td>63.677</td>
<td>0.2313</td>
</tr>
<tr>
<td>C9</td>
<td>63.677</td>
<td>66.861</td>
<td>0.2313</td>
</tr>
<tr>
<td>BA1</td>
<td>70.445</td>
<td>72.568</td>
<td>1.6148</td>
</tr>
<tr>
<td>BA2</td>
<td>72.568</td>
<td>72.853</td>
<td>5.3733</td>
</tr>
<tr>
<td>BA3</td>
<td>72.853</td>
<td>73.542</td>
<td>0.2507</td>
</tr>
<tr>
<td>BA4</td>
<td>73.542</td>
<td>73.748</td>
<td>2.6302</td>
</tr>
<tr>
<td>BA5</td>
<td>73.748</td>
<td>74.595</td>
<td>1.8146</td>
</tr>
<tr>
<td>BA6</td>
<td>74.595</td>
<td>75.000</td>
<td>1.3998</td>
</tr>
</tbody>
</table>

Table 3-3: Approximated Properties of Area of Columns
### Table 3-4: Actual Properties of Area of Columns

<table>
<thead>
<tr>
<th>Sec. Name</th>
<th>Level From</th>
<th>Level To</th>
<th>Properties of Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>42.181</td>
<td>43.730</td>
<td>A (m²)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.7042</td>
</tr>
<tr>
<td>C1</td>
<td>43.730</td>
<td>45.765</td>
<td></td>
</tr>
<tr>
<td>C1</td>
<td>AVERAGE PROPERTIES</td>
<td></td>
<td>0.7368</td>
</tr>
<tr>
<td>C2</td>
<td>45.765</td>
<td>46.660</td>
<td></td>
</tr>
<tr>
<td>C2</td>
<td>46.660</td>
<td>48.820</td>
<td></td>
</tr>
<tr>
<td>C2</td>
<td>48.820</td>
<td>50.272</td>
<td></td>
</tr>
<tr>
<td>C2</td>
<td>AVERAGE PROPERTIES</td>
<td></td>
<td>0.2761</td>
</tr>
<tr>
<td>C3</td>
<td>50.272</td>
<td>52.490</td>
<td></td>
</tr>
<tr>
<td>C4</td>
<td>52.490</td>
<td>54.811</td>
<td></td>
</tr>
<tr>
<td>C5</td>
<td>54.811</td>
<td>57.100</td>
<td></td>
</tr>
<tr>
<td>C6</td>
<td>57.100</td>
<td>57.750</td>
<td></td>
</tr>
<tr>
<td>C6</td>
<td>57.750</td>
<td>59.230</td>
<td></td>
</tr>
<tr>
<td>C6</td>
<td>AVERAGE PROPERTIES</td>
<td></td>
<td>0.2776</td>
</tr>
<tr>
<td>C7</td>
<td>59.230</td>
<td>61.560</td>
<td></td>
</tr>
<tr>
<td>C8</td>
<td>61.560</td>
<td>63.677</td>
<td></td>
</tr>
<tr>
<td>C9</td>
<td>63.677</td>
<td>66.861</td>
<td></td>
</tr>
<tr>
<td>BA1</td>
<td>70.445</td>
<td>72.568</td>
<td></td>
</tr>
<tr>
<td>BA2</td>
<td>72.568</td>
<td>72.853</td>
<td></td>
</tr>
<tr>
<td>BA3</td>
<td>72.853</td>
<td>73.542</td>
<td></td>
</tr>
<tr>
<td>BA4</td>
<td>73.542</td>
<td>73.748</td>
<td></td>
</tr>
<tr>
<td>BA5</td>
<td>73.748</td>
<td>74.595</td>
<td></td>
</tr>
<tr>
<td>BA6</td>
<td>74.595</td>
<td>75.000</td>
<td></td>
</tr>
</tbody>
</table>

3-5 Properties of Materials

The used properties of materials can be summarized in table 3-5, the values for stone masonry are previously used by [Juan Murcia, Seismic Analysis of Santa Maria del]
Mar Church in Barcelona, Msc thesis, SAHC 2008] because the same type of stone is used in both Santan Maria del Mar church and Barcelona cathedral.

<table>
<thead>
<tr>
<th>The Property</th>
<th>The Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>2200 kg/m$^3$</td>
</tr>
<tr>
<td>Young’s Modulus of Elasticity (E)</td>
<td>12000 MPa</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.30</td>
</tr>
<tr>
<td>Compressive Strength ($f_c$)</td>
<td>8 MPa</td>
</tr>
<tr>
<td>Tensile Strength ($f_t$)</td>
<td>0.60 MPa</td>
</tr>
<tr>
<td>Fracture Energy in Tension ($G_{t}$)</td>
<td>0.1 Nmm/mm$^2$</td>
</tr>
<tr>
<td>Fracture Energy in Compression ($G_{c}$)</td>
<td>$G_{f,c} = G_{f,t} \cdot \left(\frac{f_c}{f_t}\right)^2 = 0.1 \left(\frac{8}{6.0}\right)^2 = 17.8$ N.mm/mm$^2$</td>
</tr>
<tr>
<td>Steel Yield Strength ($F_y$)</td>
<td>280 MPa</td>
</tr>
</tbody>
</table>

Table 3-5: The used Properties of Materials

3-6 Constitutive Models of Masonry and Steel Nonlinear Behavior

3-6-1 Ideal Plasticity Model

The first assumed constitutive model was ideal plasticity in both tension and compression to describe masonry physical nonlinearity as shown in figure 3-10 (a) and (b). The ideal plasticity in tension doesn’t represent the masonry nonlinear behavior, but this is a first approximation to get an idea about the model nonlinear behavior with less computational troubles.

![Ideal Plasticity in Tension](DIANA™ manual)  
![Ideal Plasticity in Compression](DIANA™ manual)

Figure 3-10: First Examined Constitutive Model

3-6-2 Softening Model

The second assumed constitutive model was linear softening in tension and parabolic softening in compression as shown in figure 3-11 (a) and (b).
Wind and Earthquake Analysis of Spire of Cimborio of Barcelona Cathedral

3-6-3 Smeared Crack Model in Tension with Druker-Prager Model in Compression

For the nonlinear analysis under the effect of self weight only a third constitutive model was examined. This model was smeared crack model with tension cut-off criteria in tension and Druker-Prager model in compression which considered being more realistic model to describe masonry nonlinear behavior compared with the two previously examined models.

3-6-4 Constitutive Model of Steel Nonlinear Behavior

In all cases, only one constitutive model was used to describe steel nonlinear behavior which is the Von Misses model as shown in figure 3-13.
Figure 3-13: Von Mises for Reinforcement (SAHC 2008-SA2 Course- Lecture 9 by Lourenco and Jirásek)
Chapter 4

Calculations of Wind and Earthquake Loads

4-1 Introduction

This chapter will discuss first the calculations of the wind loads affecting the spire according to (Eurocode 1, part 1-4: wind actions) and the (Egyptian Code for Loads 1993), then it will discuss the calculations of the earthquake loads affecting the spire according to (Eurocode 8: EC-1998-1:2003).

4-2 Calculations of Wind Loads

4-2-1 According to the European Code: (Eurocode 1: Actions on structures-General actions - Part 1-4: Wind actions):

The wind loads can be estimated as following:

1- \( q_b = \frac{1}{2} \rho v_b^2 \) .... Expression (4.10), Page (23).

Where:

- \( q_b \) is the basic velocity pressure
- \( \rho \) is the air density, which depends on the altitude, temperature and barometric pressure to be expected in the region during wind storms. As per note 2 page 23 the recommended value of \( \rho \) is 1.25 kg/m³.
- \( v_b \) is the basic wind velocity, defined as a function of wind direction and time of year at 10 m above ground of terrain category II. According to Spanish code \( v_b \) for Barcelona City is 29 m/s and this value is to be increased by 8% to account for a return period of 200 years, so the used value in calculations is 1.08 X 29 = 31.32 m/s.

2- \( q_p = c_e(z) \cdot q_b \) .... Expression (4.8), Page (22).

Where:

- \( q_p \) is the peak velocity pressure
- \( c_e(z) \) is the exposure factor
The exposure coefficient $c_e(z)$ depends on the terrain Category and the height $Z$ from ground level. The wind loads on spire can be considered as terrain II which is defined according to EC1 Annex A page 94 as Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights.

At each level of the spire $Z$ (m) the exposure coefficient was determined.

3- The wind force $F_w$ acting on a structure or a structural component may be determined directly by using:

$$ F_w = c_s c_d \cdot c_r q_p (z_e) \cdot A_{ref} \ldots \text{Expression (5.3), Page (25).} $$

Where:

$-c_s c_d$ is the structural factor. It's approximated for the spire to be 1 and this is the maximum value of $c_s c_d$. 

Figure 4-1: Illustrations of the exposure factor $c_e(z)$ for $c_o=1.0$, $k_i=1.0$ [Eurocode 1 Part 1-4: Page (23) Figure (4.2)]

Figure 4-2: Description of terrain category II [Eurocode 1 Part 1-4: Annex (A) page (94)]
- $c_t$ is the force coefficient for the structure or structural element and it depends on the cross section of the elements of the spire as following:

3-1 Beams and columns forming the spire are treated according to EC1 item (7.6) Structural elements with rectangular sections, so $c_t$ is calculated as

$$c_t = c_{t,0} \cdot \psi_r \cdot \psi_{\lambda} \ldots$$ Expression (7.1), Page (67).

Where:

- $c_{t,0}$ is the force coefficient of rectangular sections with sharp corners and without free-end flow as given by Figure (7.23) Page (67).

- $\psi_r$ is the reduction factor for square sections with rounded corners. $\psi_r$ depends on Reynolds number and can be obtained from Figure (7.24).

---

**Figure 4-3:** Force coefficients $c_{t,0}$ of rectangular sections with sharp corners and without free-end flow [Eurocode 1 Part 1-4: page (67) Figure (7.23)]

**Figure 4-4:** Reduction factor $\psi_r$ for a square cross-section with rounded corners [Eurocode 1 Part 1-4: Page (68) Figure (7.24)]
For Columns and because they have some rounded corners $\psi_r$ was taken approximately as 0.75 which is the average of the maximum and minimum values of $\psi_r$. Whereas for Beams $\psi_r$ was taken approximately as 0.90 because they have less rounded corners.

- $\psi_\lambda$ is the end-effect factor for elements with free-end flow, and it can be obtained from Figure (7.36) Page (84) after calculating the slenderness ratio $\lambda$ from table (7.16) Page (83).

<table>
<thead>
<tr>
<th>No.</th>
<th>Position of the structure, wind normal to the plane of the page</th>
<th>Effective slenderness $\lambda$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td><img src="image1" alt="Diagram" /></td>
<td>For polygonal, rectangular and sharp edged sections and lattice structures: for $\ell &gt; 50$, $\lambda = 1.4 \ell/b$ or $\lambda = 70$, whichever is smaller.</td>
</tr>
<tr>
<td>2</td>
<td><img src="image2" alt="Diagram" /></td>
<td>For $\ell &lt; 15$, $\lambda = 2 \ell/b$ or $\lambda = 70$, whichever is smaller. For circular cylinders: for $\ell &gt; 50$, $\lambda = 0.7 \ell/b$ or $\lambda = 70$, whichever is smaller. for $\ell &lt; 15$, $\lambda = \ell/b$ or $\lambda = 70$, whichever is smaller.</td>
</tr>
<tr>
<td>3</td>
<td><img src="image3" alt="Diagram" /></td>
<td>For intermediate values of $\ell$, linear interpolation should be used.</td>
</tr>
<tr>
<td>4</td>
<td><img src="image4" alt="Diagram" /></td>
<td>For $\ell &gt; 50$, $\lambda = 0.7 \ell/b$ or $\lambda = 70$, whichever is larger. for $\ell &lt; 15$, $\lambda = \ell/b$ or $\lambda = 70$, whichever is larger. For intermediate values of $\ell$, linear interpolation should be used.</td>
</tr>
</tbody>
</table>

Table 4-1: Recommended values of $\lambda$ for cylinders, polygonal sections, rectangular sections, sharp edged structural sections and lattice structures. [Eurocode 1 Part 1-4: Page (83) Table (7.16)]
Figure 4-5: Indicative values of the end-effect factor $\psi_\lambda$ as a function of solidity ratio $\varphi$ versus slenderness $\lambda$ [Eurocode 1 Part 1-4: Page (84) Figure (7.36)]

The solidity ratio $\varphi$ is given by:

$$\varphi = \frac{A}{A_c} \quad \text{Expression (7.28) Page (84)}.$$ 

[Diagram showing the definition of solidity ratio $\varphi$]

$A_c = \ell b$ 

The value of $\varphi$ is 1 for all beams and columns of spire.

- $A_{\text{ref}}$ is the reference area and it can be determined by

$$A_{\text{ref}} = \ell \cdot b \quad \text{Expression (7.11), Page (68)}.$$ 

Where

$\ell$ is the length of the structural element being considered.

$b$ is the perpendicular length to wind pressure.
3-2 The base of the sculpture which is at the top of the spire is treated according to Eurocode 1 item (7.8) Page (69): Structural elements with regular polygonal section, so $c_f$ is calculated as

$$c_f = c_{f,0} \cdot \psi_{\lambda} \ldots \ldots \text{Expression (7.14), Page (69)}.$$ 

- $c_{f,0}$ is the force coefficient of structural elements without free-end flow and it can be obtained from table Table (7.11) Page (70).

<table>
<thead>
<tr>
<th>Number of sides</th>
<th>Sections</th>
<th>Finish of surface and of corners</th>
<th>Reynolds number $Re^{[1]}$</th>
<th>$c_{f,0}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>Pentagon</td>
<td>all</td>
<td>All</td>
<td>1,90</td>
</tr>
<tr>
<td>6</td>
<td>Hexagon</td>
<td>all</td>
<td>All</td>
<td>1,60</td>
</tr>
<tr>
<td>8</td>
<td>Octagon</td>
<td>surface smooth $\frac{r}{b} &lt; 0.075$ (2)</td>
<td>$Re \leq 2.4 \cdot 10^5$</td>
<td>1,45</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$Re \geq 3 \cdot 10^5$</td>
<td>1,30</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>surface smooth $\frac{r}{b} \geq 0.075$ (2)</td>
<td>$Re \leq 2 \cdot 10^6$</td>
<td>1,30</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$Re \geq 7 \cdot 10^6$</td>
<td>1,10</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>Decagon</td>
<td>all</td>
<td>All</td>
<td>1,30</td>
</tr>
<tr>
<td>12</td>
<td>Dodecagon</td>
<td>surface smooth (3) corners rounded</td>
<td>$2 \cdot 10^5 &lt; Re &lt; 1.2 \cdot 10^6$</td>
<td>0,90</td>
</tr>
<tr>
<td></td>
<td></td>
<td>all others</td>
<td>$Re &gt; 4 \cdot 10^5$</td>
<td>1,30</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$Re &gt; 4 \cdot 10^5$</td>
<td>1,10</td>
</tr>
<tr>
<td>16-18</td>
<td>Hexdecagon</td>
<td>surface smooth (3) corners rounded</td>
<td>$Re \leq 2 \cdot 10^5$</td>
<td>treat as a circular cylinder, see (7.9)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$2 \cdot 10^5 \leq Re &lt; 1.2 \cdot 10^5$</td>
<td>0,70</td>
</tr>
</tbody>
</table>

Reynolds number with $\nu = \nu_m$ and $\nu_m$ given in 4.3, $Re$, is defined in 7.9

$r = \text{corner radius}$, $b = \text{diameter of circumscribed circumference}$, see Figure 7.26

From wind tunnel tests on sectional models with galvanised steel surface and a section with $b = 0.3 \text{ m}$ and corner radius of $0.08 \cdot b$

Table 4-2: Force coefficient $c_{f,0}$ for regular polygonal sections.

[Eurocode 1 Part 1-4: Page (70) Table (7.11)]

- $\psi_{\lambda}$ is the end-effect factor as previously described.

- $A_{\text{ref}}$ is the reference area and it can be determined by

$$A_{\text{ref}} = \ell \cdot b$$ \ldots \ldots \text{Expression (7.15), Page (71)}.

Where:

- $\ell$ is the length of the structural element being considered.
b is the diameter of circumscribed circumference as in the following figure.

![Regular polygonal section](image)

Figure 4-7: Regular polygonal section
[Eurocode 1 Part 1-4: Page (71) Figure (7.26)]

3-3 The sculpture is treated as a fixed flag so $c_f$ is calculated as

<table>
<thead>
<tr>
<th>Flags</th>
<th>$A_{ref}$</th>
<th>$c_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fixed Flags</td>
<td>$h \cdot l$</td>
<td>1.8</td>
</tr>
</tbody>
</table>

Table 4-3: Force coefficients $c_f$ for flags
[Eurocode 1 Part 1-4: Page (82) part of Table (7.15)]

Finally, the whole previously described calculations can be summarized in the following tables using a spread sheet.
\[ q_b = \frac{1}{2} \cdot \rho \cdot V_b^2 \]

\( q_b \) is the basic velocity pressure

\( V_b \) (Barcelona) = 29 m/s

Return period = 200 years

Magnification Factor = 1.08

\( V_0 = 31.32 \) m/s
\( \rho = 1.25 \) Kg/m\(^3\)

\( q_b = 613.09 \) Kg/m.s\(^2\)

\( \frac{q_b}{c_e} = \left\lfloor 1 + 7 \cdot l_r(z) \right\rfloor \cdot \frac{1}{2} \cdot \rho \cdot v_m^2(z) = c_e(z) \cdot q_b \)

Table 4-4 (part 1): Wind Loads on Columns by EC

<table>
<thead>
<tr>
<th>Col. ID</th>
<th>Level (m)</th>
<th>( Z )</th>
<th>( C_e )</th>
<th>( q_p ) (Kg/m.s(^2) = N/m(^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>From To</td>
<td></td>
<td></td>
<td>Terrain category II</td>
</tr>
<tr>
<td>C1</td>
<td>45.33 48.91</td>
<td>47.12</td>
<td>3.42</td>
<td>2096.76</td>
</tr>
<tr>
<td>C2</td>
<td>48.91 53.42</td>
<td>51.17</td>
<td>3.48</td>
<td>2133.55</td>
</tr>
<tr>
<td>C3</td>
<td>53.42 55.64</td>
<td>54.53</td>
<td>3.55</td>
<td>2176.47</td>
</tr>
<tr>
<td>C4</td>
<td>55.64 57.96</td>
<td>56.80</td>
<td>3.58</td>
<td>2194.86</td>
</tr>
<tr>
<td>C5</td>
<td>57.96 60.25</td>
<td>59.11</td>
<td>3.60</td>
<td>2207.12</td>
</tr>
<tr>
<td>C6</td>
<td>60.25 62.38</td>
<td>61.31</td>
<td>3.62</td>
<td>2219.38</td>
</tr>
<tr>
<td>C7</td>
<td>62.38 64.71</td>
<td>63.55</td>
<td>3.68</td>
<td>2256.17</td>
</tr>
<tr>
<td>C8</td>
<td>64.71 66.83</td>
<td>65.77</td>
<td>3.70</td>
<td>2268.43</td>
</tr>
<tr>
<td>C9</td>
<td>66.83 70.01</td>
<td>68.42</td>
<td>3.71</td>
<td>2274.56</td>
</tr>
</tbody>
</table>

(Kg/m.s\(^2\) is equal to N/m\(^2\))

(\( \cdot \) is the multiplication operator)

Return period and Magnification Factor are According to National Annex of Spanish Code
\[ F_{w} = c_s c_d \cdot c_r \cdot q_s(z_s) \cdot A_{ref} \]

\[ C_s C_d = \frac{1}{1} \]

\[ c_r = c_{r,0} \cdot \psi_r \cdot \psi_d \]

**EC1 Eqn. 5.3 Page 25**

**EC1 Eqn. 7.10 Page 67**

<table>
<thead>
<tr>
<th>Col. ID</th>
<th>Level (m)</th>
<th>Column Height (m)</th>
<th>L (m)</th>
<th>S (m)</th>
<th>L/S</th>
<th>S/L</th>
<th>C_{r,0} (L/S)</th>
<th>C_{r,0} (S/L)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>45.33</td>
<td>48.91</td>
<td>3.61</td>
<td>1.37</td>
<td>0.82</td>
<td>1.66</td>
<td>0.60</td>
<td>1.83</td>
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<tr>
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<td>53.42</td>
<td>4.54</td>
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<td>0.70</td>
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**Table 4-4 (part 2): Follow Wind Loads on Columns by EC**

Notes
L = Long dimension of Column
S = Short dimension of Column
b = Perpendicular length to Wind
<table>
<thead>
<tr>
<th>(r/b)</th>
<th>(\psi_r)</th>
<th>(\psi_\lambda)</th>
<th>(\lambda)</th>
<th>(\varphi)</th>
<th>(\psi_\lambda)</th>
<th>(C_t) (L/S)</th>
<th>(C_t) (S/L)</th>
<th>(A_{ref}) (m²)</th>
<th>(F_w) (KN)</th>
<th>(F_w) (KN/m)</th>
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<td>0.97</td>
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<td>2.09</td>
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<td>5.15</td>
</tr>
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</table>

Table 4-4 (part 3): Wind Loads on Columns by EC

Notes
L = Long dimension of Column
S = Short dimension of Column
b = Perpendicular length to Wind
\[ q_b = \frac{1}{2} \cdot \rho \cdot V_b^2 \]

EC1 Eqn. 4.10 Page 23

\( q_b \) is the basic velocity pressure

\[ V_b \text{ (Barcelona)} = 29 \quad \text{m/s} \]

Return period = 200 years

Magnification Factor = 1.08

\[ V_b = 31.32 \quad \text{m/s} \]

\[ \rho = 1.25 \quad \text{Kg/m}^3 \]

\[ q_b = 613.089 \quad \text{Kg/m.s}^2 \]

(Kg/m.s^2 is equal to N/m^2)

\[ q_p(z) = \left[1 + 7 \cdot f_v(z)\right] \cdot \frac{1}{2} \cdot \rho \cdot V_m^2(z) = c_v(z) \cdot q_b \]

EC1 Eqn. 4.8 Page 22

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Z (m)</th>
<th>( C_v )</th>
<th>( q_p \text{ (Kg/m.s}^2 = \text{N/m}^2) )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Terrain category II</td>
<td>Terrain category II</td>
</tr>
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<td>2133.55</td>
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<tr>
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<td>2213.25</td>
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<tr>
<td>B7</td>
<td>62.38</td>
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<td>2225.51</td>
</tr>
<tr>
<td>B8</td>
<td>64.71</td>
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</tr>
<tr>
<td>B9</td>
<td>66.83</td>
<td>3.69</td>
<td>2262.30</td>
</tr>
<tr>
<td>Arches</td>
<td>45.33</td>
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<td>2084.50</td>
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<tr>
<td>Wall</td>
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<td>3.72</td>
<td>2280.69</td>
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Wall = Last Top Attached Part of Columns

Table 4-5 (part 1): Wind Loads on Beams, Arches and Wall by EC
Table 4-5 (part 2): Wind Loads on Beams, Arches and Wall by EC

Table:

<table>
<thead>
<tr>
<th>Beams ID</th>
<th>Z (m)</th>
<th>Span (face to face)</th>
<th>b (Depth)</th>
<th>d (Width)</th>
<th>d/b</th>
<th>$C_{t0} (d/b)$</th>
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</thead>
<tbody>
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</tr>
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<td>0.20</td>
<td>0.39</td>
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<tr>
<td>Arches</td>
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<td>0.67</td>
<td>0.69</td>
<td>2.39</td>
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<tr>
<td>Wall</td>
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<td>0.42</td>
<td>0.42</td>
<td>2.00</td>
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Equations:

$$F_x = c_s c_d c_r q_s (z_h)^2 A_{ef}$$

EC Eqn. 5.3 Page 25

$$c_r = c_{r0} \cdot \psi_r \cdot \gamma_k$$

EC Eqn. 7.10 Page 67
Table 4-5 (part 3): Wind Loads on Beams, Arches and Wall by EC

<table>
<thead>
<tr>
<th>r/b</th>
<th>( \psi_r )</th>
<th>( \lambda )</th>
<th>( \varphi )</th>
<th>( \psi_{\lambda} )</th>
<th>( C_r )</th>
<th>( A_{\text{ref}}(m^2) )</th>
<th>( F_w(t) )</th>
</tr>
</thead>
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<td>0.1</td>
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<td>4.7</td>
<td>1</td>
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<td>0.1</td>
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</table>

Table 4-5 (part 3): Wind Loads on Beams, Arches and Wall by EC
\[ q_b = \frac{1}{2} \cdot \rho \cdot v_b^2 \]  

EC1 Eqn. 4.10 Page 23

\( q_b \) is the basic velocity pressure

\[ V_b \text{(Barcelona)} = 29 \text{ m/s} \]

Return period = 200 years

Magnification Factor = 1.08

\[ V_b = 31.32 \text{ m/s} \]

\[ \rho = 1.25 \text{ Kg/m}^3 \]

\[ q_b = 613.089 \text{ Kg/m.s}^2 \]  

(Kg/m.s^2 is equal to N/m^2)

\[ q_p(z) = [1 + 7 \cdot l_v(z)] \cdot \frac{1}{2} \cdot \rho \cdot v_m^2(z) = q_e(z) \cdot q_b \]  

EC1 Eqn. 4.8 Page 22

\[ v = \sqrt{\frac{2 \cdot q_p}{\rho}} \]  

EC1 Note 2 Page 73

<table>
<thead>
<tr>
<th>Item</th>
<th>Level (m)</th>
<th>Z</th>
<th>C_e</th>
<th>q_p (Kg/m.s^2 = N/m^2)</th>
<th>v</th>
</tr>
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<td>From</td>
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Table 4-6 (part 1): Wind Loads on Sculpture and Its Base by EC
\[ F_w = c_s c_d \cdot c_r \cdot q_f(z_e) \cdot A_{ef} \]

\[ c_s c_d = 1 \]

\[ c_r = c_{r0} \cdot \psi_{\lambda} \]

\[ \nu = 1.50E-05 \text{ m}^2/\text{s} \]

---

### Table 4-6 (part 2): Wind Loads on Sculpture and Its Base by EC

<table>
<thead>
<tr>
<th>Item</th>
<th>Level (m)</th>
<th>Z</th>
<th>( \lambda )</th>
<th>( \psi_{\lambda} )</th>
<th>( \text{Re} = \frac{b \cdot v(z_e)}{\nu} )</th>
<th>( C_{t0} )</th>
<th>( C_t )</th>
<th>( A_{ref} (m^2) )</th>
<th>( F_w (N) )</th>
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</thead>
<tbody>
<tr>
<td><strong>Base of Sculpture</strong></td>
<td>73.60 75.72 74.66</td>
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<td>0.68</td>
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<td>1 CONSIDERED IT LIKE A FLAG ( EC page 82 )</td>
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Note: The table above is a simplified representation of the data from the document. The full table includes additional columns and rows not shown here. The equation \( F_w \) is calculated using the given parameters, and the unit \( N \) represents Newtons, which is the SI unit for force.
<table>
<thead>
<tr>
<th>Item</th>
<th>Level (m)</th>
<th>Height</th>
<th>$F_w$(t)</th>
<th>$F_w$(t/m)</th>
<th>$F_w$(KN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base of Sculpture</td>
<td>From</td>
<td>To</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>73.60</td>
<td>75.72</td>
<td>2.12</td>
<td>0.20</td>
<td>0.10</td>
<td>0.96</td>
</tr>
<tr>
<td>75.72</td>
<td>76.00</td>
<td>0.28</td>
<td>0.85</td>
<td>2.98</td>
<td>29.82</td>
</tr>
<tr>
<td>76.00</td>
<td>76.69</td>
<td>0.69</td>
<td>0.60</td>
<td>0.87</td>
<td>8.69</td>
</tr>
<tr>
<td>76.69</td>
<td>76.90</td>
<td>0.21</td>
<td>0.49</td>
<td>2.37</td>
<td>23.67</td>
</tr>
<tr>
<td>76.90</td>
<td>77.74</td>
<td>0.85</td>
<td>0.29</td>
<td>0.35</td>
<td>3.47</td>
</tr>
<tr>
<td>77.74</td>
<td>78.15</td>
<td>0.41</td>
<td>0.19</td>
<td>0.48</td>
<td>4.77</td>
</tr>
<tr>
<td>Sculpture</td>
<td>78.15</td>
<td>83.15</td>
<td>5.00</td>
<td>2.25</td>
<td>Force 2.25 t</td>
</tr>
</tbody>
</table>

Table 4-6 (part 3): Wind Loads on Sculpture and Its Base by EC
4-2-2 According to the Egyptian Code for Loads, 1993:

This code was used as a method of validation of calculations. The wind loads were calculated as following:

\[ P_e = C_e \cdot K \cdot q \]

where

- \( P_e \) = The static external wind pressure acting perpendicular to the unit area of the external surfaces of the building
- \( C_e \) = A factor depends on the shape and size of the structure
- \( K \) = Coefficient depending on the height of building \( Z \) (ms) which is measured from the ground level.

\( q \) = The pressure of wind per square meter and it depends on the location of the building (I assumed that Barcelona has wind pressure 1KN/m² which is a little bit higher than Alexandria city which is 0.80 KN/m²)

<table>
<thead>
<tr>
<th>Col. ID</th>
<th>Level (m)</th>
<th>Area sub. To Wind (m²)</th>
<th>C_e</th>
<th>K</th>
<th>q (kN/m²)</th>
<th>P_e (kN/m²)</th>
<th>Wind Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>45.33</td>
<td>48.91</td>
<td>3.61</td>
<td>1.37</td>
<td>0.82</td>
<td>4.93</td>
<td>2.96</td>
</tr>
<tr>
<td>C2</td>
<td>48.91</td>
<td>53.42</td>
<td>4.54</td>
<td>0.73</td>
<td>0.70</td>
<td>3.29</td>
<td>3.19</td>
</tr>
<tr>
<td>C3</td>
<td>53.42</td>
<td>55.64</td>
<td>2.23</td>
<td>0.68</td>
<td>0.64</td>
<td>1.52</td>
<td>1.43</td>
</tr>
<tr>
<td>C4</td>
<td>55.64</td>
<td>57.96</td>
<td>2.34</td>
<td>0.68</td>
<td>0.64</td>
<td>1.59</td>
<td>1.50</td>
</tr>
<tr>
<td>C5</td>
<td>57.96</td>
<td>60.25</td>
<td>2.31</td>
<td>1.17</td>
<td>0.69</td>
<td>2.70</td>
<td>1.59</td>
</tr>
<tr>
<td>C6</td>
<td>60.25</td>
<td>62.38</td>
<td>2.15</td>
<td>0.80</td>
<td>0.69</td>
<td>1.72</td>
<td>1.48</td>
</tr>
<tr>
<td>C7</td>
<td>62.38</td>
<td>64.71</td>
<td>2.35</td>
<td>0.67</td>
<td>0.65</td>
<td>1.57</td>
<td>1.53</td>
</tr>
<tr>
<td>C8</td>
<td>64.71</td>
<td>66.83</td>
<td>2.13</td>
<td>0.67</td>
<td>0.65</td>
<td>1.43</td>
<td>1.39</td>
</tr>
<tr>
<td>C9</td>
<td>66.83</td>
<td>70.01</td>
<td>3.21</td>
<td>0.67</td>
<td>0.65</td>
<td>2.15</td>
<td>2.09</td>
</tr>
</tbody>
</table>

Table 4-7: Wind Loads on Columns by Egyptian Code

Notes
- Col. = Columns
- Press. = Pressure
- Suc. = Suction
- L = Long dimension of Column
- S = Short dimension of Column
<table>
<thead>
<tr>
<th>Level (m)</th>
<th>Area Subjected To Wind (m²)</th>
<th>C_e</th>
<th>K</th>
<th>q (kN/m²)</th>
<th>P_e (kN/m²)</th>
<th>Wind Load (KN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>depth</td>
<td>Span (face to face)</td>
<td>Area</td>
<td>Pressure</td>
<td>Suction</td>
<td>Pressure</td>
</tr>
<tr>
<td>48.91</td>
<td>1.08</td>
<td>2.5384</td>
<td>2.74</td>
<td>0.8</td>
<td>0.5</td>
<td>1.5</td>
</tr>
<tr>
<td>50.37</td>
<td>0.26</td>
<td>1.97</td>
<td>0.51</td>
<td>0.8</td>
<td>0.5</td>
<td>1.7</td>
</tr>
<tr>
<td>53.42</td>
<td>0.52</td>
<td>1.71</td>
<td>0.89</td>
<td>0.8</td>
<td>0.5</td>
<td>1.7</td>
</tr>
<tr>
<td>55.64</td>
<td>0.49</td>
<td>1.53</td>
<td>0.75</td>
<td>0.8</td>
<td>0.5</td>
<td>1.7</td>
</tr>
<tr>
<td>57.96</td>
<td>0.56</td>
<td>1.33</td>
<td>0.74</td>
<td>0.8</td>
<td>0.5</td>
<td>1.7</td>
</tr>
<tr>
<td>60.25</td>
<td>0.60</td>
<td>1.14</td>
<td>0.68</td>
<td>0.8</td>
<td>0.5</td>
<td>1.7</td>
</tr>
<tr>
<td>62.38</td>
<td>0.55</td>
<td>0.96</td>
<td>0.53</td>
<td>0.8</td>
<td>0.5</td>
<td>1.7</td>
</tr>
<tr>
<td>64.71</td>
<td>0.69</td>
<td>0.77</td>
<td>0.53</td>
<td>0.8</td>
<td>0.5</td>
<td>1.7</td>
</tr>
<tr>
<td>66.83</td>
<td>0.25</td>
<td>0.59</td>
<td>0.15</td>
<td>0.8</td>
<td>0.5</td>
<td>1.7</td>
</tr>
</tbody>
</table>

**Table 4-8: Wind Loads on Beams by Egyptian Code**

<table>
<thead>
<tr>
<th>Item</th>
<th>Level (m)</th>
<th>Area Subjected To Wind (m²)</th>
<th>Ce</th>
<th>K</th>
<th>q (kN/m²)</th>
<th>P_e (kN/m²)</th>
<th>Wind Load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>From</td>
<td>To</td>
<td>Press.</td>
<td>Suc.</td>
<td>Press.</td>
<td>Suc.</td>
<td>Press. + Suc.</td>
</tr>
<tr>
<td>Arches</td>
<td>47.98</td>
<td>48.91</td>
<td>0.86</td>
<td>1.00</td>
<td>0.86</td>
<td>0.8</td>
<td>1.5</td>
</tr>
<tr>
<td>Wall</td>
<td>70.01</td>
<td>73.60</td>
<td>3.58</td>
<td>1.59</td>
<td>5.70</td>
<td>0.8</td>
<td>1.7</td>
</tr>
<tr>
<td>Base of</td>
<td>73.60</td>
<td>75.72</td>
<td>2.12</td>
<td>1.63</td>
<td>3.46</td>
<td>0.8</td>
<td>1.7</td>
</tr>
<tr>
<td>Sculpture</td>
<td>75.72</td>
<td>76.00</td>
<td>0.28</td>
<td>2.54</td>
<td>0.72</td>
<td>0.8</td>
<td>1.7</td>
</tr>
<tr>
<td></td>
<td>76.00</td>
<td>76.69</td>
<td>0.69</td>
<td>2.10</td>
<td>1.44</td>
<td>0.8</td>
<td>1.7</td>
</tr>
<tr>
<td>Base of</td>
<td>76.69</td>
<td>76.90</td>
<td>0.21</td>
<td>1.79</td>
<td>0.37</td>
<td>0.8</td>
<td>1.7</td>
</tr>
<tr>
<td>Sculpture</td>
<td>76.90</td>
<td>77.74</td>
<td>0.85</td>
<td>1.47</td>
<td>1.24</td>
<td>0.8</td>
<td>1.7</td>
</tr>
<tr>
<td></td>
<td>77.74</td>
<td>78.15</td>
<td>0.41</td>
<td>1.34</td>
<td>0.54</td>
<td>0.8</td>
<td>1.7</td>
</tr>
<tr>
<td>Sculpture</td>
<td>78.15</td>
<td>83.15</td>
<td>5.00</td>
<td>1.06</td>
<td>5.30</td>
<td>0.8</td>
<td>1.7</td>
</tr>
</tbody>
</table>

**Table 4-9: Wind Loads on Arches, Wall, Sculpture and Base of Sculpture by Egyptian Code**
4-3 Calculations of Earthquake Loads according to (Eurocode 8 : EC-1998-1:2003)

4-3-1 Lateral Force Method of Analysis [Eurocode Item 4.3.3.2 Page (42)]

4-3-1-1 Check the Applicability of This Method to The Spire

This method can be applied if the following two conditions are satisfied:

a) The fundamental period of vibration $T_1$ in the two main directions is smaller than the following values:

$$T_1 \leq 4 \cdot T_C$$

2 Sec.

Where $T_C$ is taken from table 3.2 Page (24)

<table>
<thead>
<tr>
<th>Ground type</th>
<th>$S$</th>
<th>$T_B$ (s)</th>
<th>$T_C$ (s)</th>
<th>$T_D$ (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1.0</td>
<td>0.15</td>
<td>0.4</td>
<td>2.0</td>
</tr>
<tr>
<td>B</td>
<td>1.2</td>
<td>0.15</td>
<td>0.5</td>
<td>2.0</td>
</tr>
<tr>
<td>C</td>
<td>1.15</td>
<td>0.20</td>
<td>0.6</td>
<td>2.0</td>
</tr>
<tr>
<td>D</td>
<td>1.35</td>
<td>0.20</td>
<td>0.8</td>
<td>2.0</td>
</tr>
<tr>
<td>E</td>
<td>1.4</td>
<td>0.15</td>
<td>0.5</td>
<td>2.0</td>
</tr>
</tbody>
</table>

Table 4-10: Values of the parameters describing the recommended Type 1 elastic response spectra: [Eurocode 8: Table 3.2- Page (24)]

The soil of Barcelona Cathedral is type B so $T_C = 0.5$ sec. From Modal analysis the fundamental period of vibration of the spire (without reinforcement) is 0.49 sec. So the spire is satisfying the first condition.

b) The spire should meet the criteria for regularity in elevation given in item (4.2.3.3) Page (36) which is checked in the following table.

<table>
<thead>
<tr>
<th>The Condition</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>All lateral load resisting systems, such as cores, structural walls, or frames, shall run without interruption from their foundations to the top of the building or, if setbacks at different heights are present, to the top of the relevant zone of the building.</td>
<td>Satisfied</td>
</tr>
<tr>
<td>Both the lateral stiffness and the mass of the individual storeys shall remain constant or reduce gradually, without abrupt changes, from the base to the top of a particular building.</td>
<td>Unsatisfied</td>
</tr>
<tr>
<td>In framed buildings the ratio of the actual storey resistance to the resistance required by the analysis should not vary disproportionately between adjacent storeys.</td>
<td>Satisfied</td>
</tr>
<tr>
<td>When setbacks are present, the following additional conditions apply:</td>
<td></td>
</tr>
</tbody>
</table>
For gradual setbacks preserving axial symmetry, the setback at any floor shall be not greater than 20% of the previous plan dimension in the direction of the setback. **Satisfied**

For a single setback within the lower 15% of the total height of the main structural system, the setback shall be not greater than 50% of the previous plan dimension. In this case the structure of the base zone within the vertically projected perimeter of the upper storeys should be designed to resist at least 75% of the horizontal shear forces that would develop in that zone in a similar building without the base enlargement. **Inapplicable**

If the setbacks do not preserve symmetry, in each face the sum of the setbacks at all storeys shall be not greater than 30% of the plan dimension at the ground floor above the foundation or above the top of a rigid basement, and the individual setbacks shall be not greater than 10% of the previous plan dimension. **Inapplicable**

<table>
<thead>
<tr>
<th>Table 4-11: Check the criteria for regularity in elevation: [Eurocode 8: Item (4.2.3.3) - Page (36)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>From the previous two conditions we can conclude that applying lateral force method to the spire is an accepted approximation.</td>
</tr>
</tbody>
</table>

4-3-1-2 Calculation of Base Shear

\[ F_b = S_d(T_1) \cdot \lambda \]  
**Expression (4.5) Page (43)**

Where

- \( F_b \) is the base shear force

- \( S_d(T_1) \) is the ordinate of the design spectrum (from item 3.2.2.5 pages (27, 28) at period \( T_1 \), where \( T_1 \) is the fundamental period of vibration of the building for lateral motion in the direction considered.

\[
0 \leq T \leq T_B : S_d(T) = a_g \cdot S \cdot \left[ \frac{2}{3} + \frac{T}{T_B} \cdot \left( \frac{2.5}{q} - \frac{2}{8} \right) \right]
\]
For the spire $T_1$ is 0.49 sec which is satisfying the condition of $T_B \leq T_1 \leq T_C$ so $S_d$ is calculated as: $S_d(T) = a_g \cdot S \cdot \frac{2.5}{q}$

Where

- $a_g = 0.04$ g for Barcelona City for return period of 50 years but for return period of 200 years $a_g = \gamma_1$. $a_g$ [Eurocode 8: Item 3.2.1, Point (3), Page (21)] and $\gamma_1$ is equal to 1.40 according to [Eurocode 8 :Item 4.2.5, Points (4,5P), Page (39)].

- $S = 1.20$ according to: [Eurocode 8: Table (3.2) - Page (24)]

- $q = 1.5$ according to: [Eurocode 8: Table (9.1) - Page (182)]

Finally: $S_d(T) = (1.40 \times 0.04 \times 9.81) \times (1.20) \times \frac{2.5}{1.50} = 1.099 \text{ m/s}^2$

$m$ is the total mass of the building, above the foundation or above the top of a rigid basement, for the spire:

$m = \frac{262.929 \text{ (ton)} \times 1000 \text{ (KN)} \times 9.81 \text{ (N)}}{9.81} = 262929 \text{ Kg}$

Where 262.929 ton is the total weight of the spire which includes the columns, the beams, the sculpture and the cantilever sculptures of span 0.5 m and it was calculated as following:
## Calculation of The Weight of The Spire

**NOTES**

<table>
<thead>
<tr>
<th>level</th>
<th>column</th>
<th>total weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>From</td>
<td>To</td>
<td>height (m)</td>
</tr>
<tr>
<td>8 columns</td>
<td>42.18</td>
<td>45.76</td>
</tr>
<tr>
<td>16 arches</td>
<td>42.18</td>
<td>45.76</td>
</tr>
<tr>
<td>8 columns</td>
<td>45.76</td>
<td>50.27</td>
</tr>
<tr>
<td>8 columns</td>
<td>50.27</td>
<td>52.49</td>
</tr>
<tr>
<td>8 columns</td>
<td>52.49</td>
<td>54.81</td>
</tr>
<tr>
<td>8 columns</td>
<td>54.81</td>
<td>57.10</td>
</tr>
<tr>
<td>8 columns</td>
<td>57.10</td>
<td>59.23</td>
</tr>
<tr>
<td>8 columns</td>
<td>59.23</td>
<td>61.56</td>
</tr>
<tr>
<td>8 columns</td>
<td>61.56</td>
<td>63.68</td>
</tr>
<tr>
<td>8 columns</td>
<td>63.68</td>
<td>66.86</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Wall</th>
<th>base of sculpture</th>
<th>total weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>66.86</td>
<td>67.58</td>
<td>0.72</td>
</tr>
<tr>
<td>67.58</td>
<td>68.29</td>
<td>0.72</td>
</tr>
<tr>
<td>68.29</td>
<td>69.01</td>
<td>0.72</td>
</tr>
<tr>
<td>69.01</td>
<td>69.73</td>
<td>0.72</td>
</tr>
<tr>
<td>69.73</td>
<td>70.45</td>
<td>0.72</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Base of Sculpture</th>
<th>beam</th>
<th>total weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>70.45</td>
<td>71.06</td>
<td>2.62</td>
</tr>
<tr>
<td>71.06</td>
<td>72.57</td>
<td>0.51</td>
</tr>
<tr>
<td>72.57</td>
<td>72.85</td>
<td>0.28</td>
</tr>
<tr>
<td>72.85</td>
<td>73.54</td>
<td>0.69</td>
</tr>
<tr>
<td>73.54</td>
<td>74.35</td>
<td>0.21</td>
</tr>
<tr>
<td>73.75</td>
<td>74.59</td>
<td>0.85</td>
</tr>
<tr>
<td>74.59</td>
<td>75.00</td>
<td>0.41</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Level</th>
<th>beam</th>
<th>total weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>span (m)</td>
<td>Area (m²)</td>
<td>volume (m³)</td>
</tr>
<tr>
<td>45.76</td>
<td>2.54</td>
<td>0.65</td>
</tr>
<tr>
<td>47.22</td>
<td>2.44</td>
<td>0.04</td>
</tr>
<tr>
<td>49.02</td>
<td>1.82</td>
<td>0.04</td>
</tr>
<tr>
<td>50.27</td>
<td>2.18</td>
<td>0.11</td>
</tr>
<tr>
<td>52.49</td>
<td>2.00</td>
<td>0.10</td>
</tr>
<tr>
<td>53.78</td>
<td>1.42</td>
<td>0.03</td>
</tr>
<tr>
<td>54.81</td>
<td>1.81</td>
<td>0.11</td>
</tr>
<tr>
<td>57.10</td>
<td>1.62</td>
<td>0.12</td>
</tr>
<tr>
<td>58.09</td>
<td>1.06</td>
<td>0.04</td>
</tr>
<tr>
<td>59.23</td>
<td>1.44</td>
<td>0.12</td>
</tr>
<tr>
<td>60.45</td>
<td>0.86</td>
<td>0.04</td>
</tr>
<tr>
<td>61.56</td>
<td>1.25</td>
<td>0.14</td>
</tr>
<tr>
<td>62.52</td>
<td>0.69</td>
<td>0.04</td>
</tr>
<tr>
<td>63.68</td>
<td>1.07</td>
<td>0.04</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Other (the cantilever sculptures of span 0.5 m)</th>
<th>Area (m²)</th>
<th>Thickness (m)</th>
<th>volume (m³)</th>
<th>weight (tons)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
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<td>0.05</td>
<td>0.10</td>
<td>8.20</td>
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<table>
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<tr>
<th>The Sculpture</th>
<th>Area (m²)</th>
<th>Thickness (m)</th>
<th>volume (m³)</th>
<th>weight (tons)</th>
</tr>
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<tbody>
<tr>
<td>2.67</td>
<td>1.06</td>
<td>2.83</td>
<td>6.23</td>
<td></td>
</tr>
</tbody>
</table>

**Total Weight of Spire = 192.61+55.89+8.20+6.229 = 262.929 tons**

Table 4-11: Calculations of Spire Weight
- $\lambda$ is the correction factor, the value of which is equal to: $\lambda = 0.85$ if $T_1 < 2T_C$ and the building has more than two storeys, or $\lambda = 1.0$ otherwise. In the case of the spire $\lambda=1$. 

- Finally the $F_b$ for the spire is 

$$F_b = \frac{1.099 \text{ (m/s}^2\text{)} \times 262929 \text{(Kg)} \times 1}{1000 \text{ (KN)} \times 9.81 \text{ (ton)}} \cong 30 \text{ (tons)}$$
Calculations and Distribution of Earthquake Forces on Spire

F_b = 30 tons Base Shear

<table>
<thead>
<tr>
<th>Column Levels</th>
<th>Weight (ton)</th>
<th>Weight dist. (mass)</th>
<th>Height (Height)</th>
<th>Lateral Load (ton)</th>
</tr>
</thead>
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<tr>
<td>45.33 to 48.91</td>
<td>49.56</td>
<td>47.22</td>
<td>24.78</td>
<td>48.91</td>
</tr>
<tr>
<td>45.33 to 48.91</td>
<td>23.10</td>
<td>47.22</td>
<td>11.55</td>
<td>48.91</td>
</tr>
<tr>
<td>48.91 to 50.37</td>
<td>7.08</td>
<td>48.91</td>
<td>3.54</td>
<td>50.37</td>
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<tr>
<td>50.37 to 53.42</td>
<td>14.85</td>
<td>50.37</td>
<td>7.42</td>
<td>53.42</td>
</tr>
<tr>
<td>53.42 to 56.64</td>
<td>9.63</td>
<td>53.42</td>
<td>4.82</td>
<td>56.64</td>
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<td>55.64 to 57.96</td>
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<td>55.64</td>
<td>5.05</td>
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</tr>
<tr>
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<td>13.38</td>
<td>57.96</td>
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<td>60.25</td>
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<td>60.25 to 62.38</td>
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<td>6.74</td>
<td>70.01</td>
</tr>
<tr>
<td>70.01 to 70.49</td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

Table 4-12: Calculations and Distribution of Earthquake Forces on Spire
Chapter 5

Results of Linear Elastic Analysis and Modal Analysis

5-1 Introduction

This chapter will discuss the obtained results for linear elastic analysis under the effect of self weight, combination of self weight and earthquake loads and combination of self weight and wind loads. Also, the results of modal analysis will be presented.

5-2 Results of Linear Elastic Analysis under Self Weight:

5-2-1 Deformed Shapes

As expected there is no deformation of any significant magnitude (the maximum value is of order of $10^{-4}$ m which is almost zero) in the horizontal directions (X and Y directions) with and without reinforcement. The only significant deformation is the vertical deformation (Z direction). The maximum value is 1.08 mm at the last top point of the spire (this point is subjected directly to the concentrated load of the sculpture at top). The reinforcement has no effect on the global deformation of the spire.

![Figure 5-1: Total Deformations Without Reinforcement (m)](image1)

![Figure 5-2: Total Deformations With Reinforcement (m)](image2)

5-2-2 Reactions Values

There is no difference in values of reactions of the two models, in the two models the resultant of horizontal reactions (X and Y directions) are equal to zero. The resultant of vertical reactions = $0.355\times 10^6 \times 8 = 2.84\times 10^6$ N, which is the summation of the weight...
of the spire which equals to 2.63E6 N, and the weight of the eight pinnacles on the
eight corners of the arch at base which equals to 0.19E6 N. The weight of
reinforcement is very low to make a difference in the value of vertical reactions.

Figure 5-3: Horizontal Reaction
(X-direction) Without Reinforcement (N)

Figure 5-4: Horizontal Reaction
(X-direction) With Reinforcement (N)

Figure 5-5: Horizontal Reaction
(Y-direction) Without Reinforcement (N)

Figure 5-6: Horizontal Reaction
(Y-direction) With Reinforcement (N)

Figure 5-7: Vertical Reaction
(Z-direction) Without Reinforcement (N)

Figure 5-8: Vertical Reaction
(Z-direction) With Reinforcement (N)
5-2-3  Straining Actions and Normal Stresses

5-2-3-1 Straining Actions and Normal Stresses of Columns

The following three figures show the three local axes to which all straining actions and stresses are related.

Figure 5-9: Local x-axis (in Red)  Figure 5-10: Local y-axis (in Blue)

Figure 5-11: Local z-axis (in Violet)

The reinforcement has no significant effect on the straining actions and hence on normal stresses of columns; the following figures are plot of straining actions and normal stresses of column C1 - as an example - and the same values of straining actions and normal stresses can be noticed with and without reinforcement.
Figure 5-12: N.F.D (Nx) (Newton) of Columns C1 without reinforcement

Figure 5-13: N.F.D (Nx) (Newton) of Columns C1 with reinforcement
Figure 5-14: B.M.D (My) (Newton.m) of Columns C1 without reinforcement

Figure 5-15: B.M.D (My) (Newton.m) of Columns C1 with reinforcement
Figure 5-16: S.F.D (Qz) (Newton) of Columns C1 without reinforcement

Figure 5-17: S.F.D (Qz) (Newton) of Columns C1 with reinforcement
5-2-3-2 Straining Actions and Normal Stresses of Beams

The following three figures show the three local axes to which all straining actions and Stresses are related.
The reinforcement adds axial stiffness to beams cross sections, so there is an increase in the values of axial forces in the model with reinforcement. The axial force carried by masonry in the model with reinforcement is slightly less than the axial force carried by masonry in the model without reinforcement due to the share of reinforcement, and as a result the tensile stresses at top fibers in the model without reinforcement are slightly less than the tensile stresses at top fibers in the model with reinforcement and on the contrary the compressive stresses in the model without reinforcement are slightly more than the compressive stresses in the model with reinforcement as shown in figures 5-23 and 5-24. The reinforcement has no significant effect on the other straining actions. The following figures are for the straining actions and stresses of beam B2 as an example.
Figure 5-23: N.F.D (Nx) (Newton) of Beam B2 without reinforcement

Figure 5-24: N.F.D (Nx) (Newton) of Beam B2 with reinforcement
Figure 5-25: B.M.D (My) (Newton.m) of Beam B2 without reinforcement

Figure 5-26: B.M.D (My) (Newton.m) of Beam B2 with reinforcement
Wind and Earthquake Analysis of Spire of Cimborio of Barcelona Cathedral

Figure 5-27: S.F.D (Qz) (Newton) of Beam B2 without reinforcement

Figure 5-28: S.F.D (Qz) (Newton) of Beam B2 with reinforcement
Wind and Earthquake Analysis of Spire of Cimborio of Barcelona Cathedral

Figure 5-29: Normal Stress (N/m²) of Beam B2 without reinforcement

Figure 5-30: Normal Stress (N/m²) of Beam B2 with reinforcement
5-2-3-3 Check on Maximum Normal Stresses

Under only self weight of the spire the compressive normal stresses as expected are very low compared with the compressive strength. The maximum compressive stress is 0.74 MPa in columns C3 which represents only 9.3% of the compressive strength (8 MPa), also the tensile strength (0.6 MPa) is not exceeded at any of the elements as shown in as shown in figures 00-00 and 00-00. The reinforcement has no effect on the normal stresses.
Wind and Earthquake Analysis of Spire of Cimborio of Barcelona Cathedral

Figure 5-32: Normal Stresses (N/m²) in Spire without Reinforcement

Figure 5-33: Normal Stresses (N/m²) in Spire with Reinforcement
5-3 Results of Linear Elastic Analysis under (Self Weight+ Earthquake):

5-3-1 Deformed Shapes

The spire is symmetric about the two perpendicular axes X and Y, so the earthquake in X-direction only will be discussed and the same conclusions can be applied in the other direction, Y.

The maximum horizontal displacement is at the last top point of the spire with a value of 1.1 cm. The reinforcement has no effect on the global deformation of the spire (the same result like the case of self weight only).

Figure 5-34: Total Deformations Without Reinforcement (m)
Figure 5-35: Total Deformations With Reinforcement (m)
5-3-2 Reactions Values

Figure 5-36: Horizontal Reaction (X-direction) Without Reinforcement

Figure 5-37: Horizontal Reaction (X-direction) With Reinforcement

Figure 5-38: Horizontal Reaction (Y-direction) Without Reinforcement

Figure 5-39: Horizontal Reaction (Y-direction) With Reinforcement

Figure 5-40: Vertical Reaction (Z-direction) Without Reinforcement

Figure 5-41: Vertical Reaction (Z-direction) With Reinforcement
There is no difference in values of reactions of the two models, in the two models the resultant of horizontal reactions (Y-direction) is equal to zero. The resultant of the horizontal reactions (X-direction) is equal to 0.3E6 N which is equal to the applied base shear. Again and like the case of self weight, the resultant of vertical reactions = 0.355E6 X 8 = 2.84E6 N.

5-3-3 Straining Actions and Normal Stresses

5-3-3-1 Straining Actions and Normal Stresses of Columns

Like the case of analysis under self weight only, the reinforcement has no significant effect on the straining actions and hence on normal stresses of columns; the following figures are plot of straining actions and normal stresses of column C3 -as an example- and the same values of straining actions and normal stresses can be noticed with and without reinforcement.

Figure 5-41: N.F.D (Nx) (Newton) of Columns C3 without reinforcement
Figure 5-42: N.F.D (Nx) (Newton) of Columns C3 with reinforcement

Figure 5-43: B.M.D (My) (Newton.m) of Columns C3 without reinforcement
Figure 5-44: B.M.D (My) (Newton.m) of Columns C3 with reinforcement

Figure 5-45: B.M.D (Mz) (Newton.m) of Columns C3 without reinforcement
Wind and Earthquake Analysis of Spire of Cimborio of Barcelona Cathedral

Figure 5-46: B.M.D (Mz) (Newton.m) of Columns C3 with reinforcement

Figure 5-47: S.F.D (Qy) (Newton) of Columns C3 without reinforcement
Figure 5-48: S.F.D (Qy) (Newton) of Columns C3 with reinforcement

Figure 5-49: S.F.D (Qz) (Newton) of Columns C3 without reinforcement
Figure 5-50: S.F.D (Qz) (Newton) of Columns C3 with reinforcement

Figure 5-51: Normal Stress (N/m²) of Column C3 without reinforcement
5-3-3-2 Straining Actions and Normal Stresses of Beams

The same conclusions about reinforcement role in beams in the case of analysis under self weight can be shown in the case of (self weight) as shown in figures 5-53 and 5-54. The following figures are for the straining actions and stresses of beam B3 as an example.
Figure 5-53: N.F.D (Nx) (Newton) of Beam B3 without reinforcement

Figure 5-54: N.F.D (Nx) (Newton) of Beam B3 with reinforcement
Wind and Earthquake Analysis of Spire of Cimborio of Barcelona Cathedral

Figure 5-55: B.M.D (My) (Newton.m) of Beam B3 without reinforcement

Figure 5-56: B.M.D (My) (Newton.m) of Beam B3 with reinforcement
Figure 5-57: S.F.D (Qz) (Newton) of Beam B3 without reinforcement

Figure 5-58: S.F.D (Qz) (Newton) of Beam B3 with reinforcement
Figure 5-59: Normal Stress (N/m²) of Beam B3 without reinforcement

Figure 5-60: Normal Stress (N/m²) of Beam B3 with reinforcement
5-3-3-3 Check on Maximum Normal Stresses

Under the application of (self weight + Earthquake) and for both cases with and without reinforcement, column C2 is subjected to the maximum compressive normal stress of 2.3 MPa which represents only 29% of the compressive strength (8 MPa) as shown in figures 00-00 and 00-00.

Figure 5-61: Normal Stresses in Columns without Reinforcement

Figure 5-62: Normal Stresses in Columns with Reinforcement
All beams are subjected to tensile stresses greater than the tensile strength, which means that these beams are cracked and linear elastic analysis is not enough to identify the actual stresses distributions as shown in figures 5-63 and 5-64.

Figure 5-63: Tensile Stresses Greater Than Masonry Tensile Strength in Beams without Reinforcement (Blue Zones)

Figure 5-64: Tensile Stresses Greater Than Masonry Tensile Strength in Beams with Reinforcement (Blue Zones)
5-4 Results of Linear Elastic Analysis under (Self Weight+ Wind):

5-4-1 Deformed Shapes

The wind in X-direction only will be discussed and the same conclusions can be applied in the other direction, Y (like the case of self weight + earthquake).

The maximum horizontal displacement is at the last top point of the spire with a value of approximately 2 cm (approximately twice the case of self weight + earthquake, because the wind load is approximately twice the earthquake load). Like the two previously discussed cases (self weight and combination of self weight + earthquake) the reinforcement has no effect on the global deformation of the spire.
5-4-2 Reactions Values

Figure 5-67: Horizontal Reaction (X-direction) Without Reinforcement (N)

Figure 5-68: Horizontal Reaction (X-direction) With Reinforcement (N)

Figure 5-69: Horizontal Reaction (Y-direction) Without Reinforcement (N)

Figure 5-70: Horizontal Reaction (Y-direction) With Reinforcement (N)

Figure 5-71: Vertical Reaction (Z-direction) Without Reinforcement (N)

Figure 5-72: Vertical Reaction (Z-direction) With Reinforcement (N)
Like the two previous mentioned cases there is no difference in values of reactions of the two models, in the two models the resultant of horizontal reactions (Y-direction) is equal to zero. The resultant of the horizontal reactions (X-direction) is equal to 0.6E6 N which is equal to the applied base shear. Again and like the two previous cases (self weight and combination of self weight + earthquake) the resultant of vertical reactions = 0.355E6 X 8 = 2.84E6 N.

5-4-3 Straining Actions and Normal Stresses

5-4-3-1 Straining Actions and Normal Stresses of Columns

Like the case of analysis under self weight only the reinforcement has no significant effect on the straining actions and hence on normal stresses of columns; the following figures are plot of straining actions and normal stresses of column C6 -as an example- and the same values of straining actions and normal stresses can be noticed with and without reinforcement.

Figure 5-73: N.F.D (Nx) (Newton) of Columns C6 without reinforcement
Figure 5-74: N.F.D (Nx) (Newton) of Columns C6 with reinforcement

Figure 5-75: B.M.D (My) (Newton.m) of Columns C6 without reinforcement
Wind and Earthquake Analysis of Spire of Cimborio of Barcelona Cathedral

Figure 5-76: B.M.D (My) (Newton.m) of Columns C6 with reinforcement

Figure 5-77: B.M.D (Mz) (Newton.m) of Columns C6 without reinforcement
Figure 5-78: B.M.D (Mz) (Newton.m) of Columns C6 with reinforcement

Figure 5-79: S.F.D (Qy) (Newton) of Columns C6 without reinforcement
Wind and Earthquake Analysis of Spire of Cimborio of Barcelona Cathedral

Figure 5-80: S.F.D (Qy) (Newton) of Columns C6 with reinforcement

Figure 5-81: S.F.D (Qz) (Newton) of Columns C6 without reinforcement
Wind and Earthquake Analysis of Spire of Cimborio of Barcelona Cathedral

Figure 5-82: S.F.D (Qz) (Newton) of Columns C6 with reinforcement

Figure 5-83: Normal Stress (N/m²) of Column C6 without reinforcement
5-4-3-2 Straining Actions and Normal Stresses of Beams

The same conclusions about reinforcement role in beams in the case of analysis under self weight can be shown in the case of (self weight + Earthquake) as shown in figures 5-85 and 5-86. The following figures are for the straining actions and stresses of beam B6 as an example.
Figure 5-85: N.F.D (Nx) (Newton) of Beam B6 without reinforcement

Figure 5-86: N.F.D (Nx) (Newton) of Beam B6 with reinforcement
Wind and Earthquake Analysis of Spire of Cimborio of Barcelona Cathedral

Figure 5-87: B.M.D (My) (Newton.m) of Beam B6 without reinforcement

Figure 5-88: B.M.D (My) (Newton.m) of Beam B6 with reinforcement
Wind and Earthquake Analysis of Spire of Cimborio of Barcelona Cathedral

Figure 5-89: S.F.D (Qz) (Newton) of Beam B6 without reinforcement

Figure 5-90: S.F.D (Qz) (Newton) of Beam B6 with reinforcement
Wind and Earthquake Analysis of Spire of Cimborio of Barcelona Cathedral

Figure 5-91: Normal Stress (N/m²) of Beam B6 without reinforcement

Figure 5-92: Normal Stress (N/m²) of Beam B6 with reinforcement
5-4-3-3 Check on Maximum Normal Stresses

Under the application of (self weight + Wind) and for both cases with and without reinforcement, column C2 is subjected to the maximum compressive normal stress of 4.2 MPa which represents only 53% of the compressive strength (8 MPa) (this value is approximately twice the case of self weight + earthquake because the wind load is approximately twice the earthquake load) as shown in figures 5-93 and 5-94.

**Figure 5-93: Normal Stresses in Columns without Reinforcement**

**Figure 5-94: Normal Stresses in Columns with Reinforcement**
All beams are subjected to tensile stresses greater than the tensile strength, which means that (like the case of self weight + Earthquake) these beams are cracked and linear elastic analysis is not enough to identify the actual stresses distributions as shown in figures 5-95 and 5-96.

Figure 5-95: Tensile Stresses Greater Than Masonry Tensile Strength in Beams without Reinforcement (red Zones)

Figure 5-96: Tensile Stresses Greater Than Masonry Tensile Strength in Beams with Reinforcement (red Zones)
5-5 Results of Modal Analysis

A modal analysis has been carried out and the first twenty modes of vibration have been obtained. The cumulative mass participation of these twenty modes of vibration is 95.33%. The frequencies of mode 1 and mode 2 are 2.18 Hz and they have the largest mass participation factor of 48.2% in x-direction and y-directions respectively as shown in figure 00-00. Mode shapes 13 (x-direction) and 12 (y-direction) are the second largest mode shapes with mass participation factor of 13% as shown in figure 00-00.
The reinforcement has no effect on the mode shapes because it contributes only with an increase in the axial stiffness and no effect on the flexural stiffness.

Tables 5-1 and 5-2 summarize the frequencies and mass participation factors for different modes in x-direction and y-directions respectively:

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency (HZ)</th>
<th>Percentage (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.18</td>
<td>48.21</td>
</tr>
<tr>
<td>3</td>
<td>4.46</td>
<td>10.62</td>
</tr>
<tr>
<td>8</td>
<td>9.19</td>
<td>4.32</td>
</tr>
<tr>
<td>10</td>
<td>11.08</td>
<td>10.16</td>
</tr>
<tr>
<td>13</td>
<td>13.56</td>
<td>13.28</td>
</tr>
</tbody>
</table>

Table 5-1: Important Mode Shapes in X-direction
<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency (HZ)</th>
<th>Percentage (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>2.18</td>
<td>48.21</td>
</tr>
<tr>
<td>4</td>
<td>4.46</td>
<td>10.62</td>
</tr>
<tr>
<td>9</td>
<td>9.19</td>
<td>4.32</td>
</tr>
<tr>
<td>11</td>
<td>11.08</td>
<td>10.16</td>
</tr>
<tr>
<td>12</td>
<td>13.56</td>
<td>13.28</td>
</tr>
</tbody>
</table>

Table 5-2: Important Mode Shapes in Y-direction
Chapter 6

Results of Non-Linear Analysis

6-1 Introduction

This chapter will discuss first the nonlinear analysis under self weight then the pushover static nonlinear analysis. In both analyses geometrical and physical nonlinearity are considered. The material properties and constitutive models have been already defined in chapter 3.

6-2 Results of Non-Linear Analysis under Self Weight:

6-2-1 Ideal Plasticity model

The used method of solution is Newton with maximum number of iteration of 50 taking into account both Arc length control and line search. The load was applied incrementally as 20 steps each step equals to 0.5 of the self weight then 120 steps each step equals to 0.25 of the self weight. The convergence criteria are based on both displacement and force with tolerance of 0.01.

The maximum load factor reached was 10.90 as shown in figure 00-00 which represents the deflection of the last top point of the spire with the increasing in load increments.

The collapse occurred in compression in columns C3 as the applied compressive stresses are larger than the compressive strength of 8 MPa, as shown in figure 00-00. The collapse occurred in tension in beams B2 and B3 as the applied tensile stresses are larger than the tensile strength of 0.6 MPa, as shown in figure 00-00.

![Figure 6-1: Deflection of Last Top Point with Load Increasing (Ideal Plasticity model)](image-url)

The collapse occurred in compression in columns C3 as the applied compressive stresses are larger than the compressive strength of 8 MPa, as shown in figure 6-2.
The collapse occurred in tension in beams B2 and B3 as the applied tensile stresses are larger than the tensile strength of 0.6 MPa, as shown in figure 6-3.

Figure 6-2: Collapse Mechanism in Compression (Ideal Plasticity model)

Figure 6-3: Collapse Mechanism in Tension (Ideal Plasticity model)
6-2-2 Softening Model

The used method of solution is Newton with maximum number of iteration of 50 taking into account both Arc length control and line search. The load was applied incrementally as 20 steps each step equals to 0.5 of the self weight then 40 steps each step equals to 0.25 of the self weight. The convergence criteria are based on both displacement and force with tolerance of 0.01.

The maximum load factor reached was 10.90 (the same as ideal plasticity model) as shown in figure 6-4 which represents the deflection of the last top point of the spire with the increasing in load increments.

![Figure 6-4: Deflection of Last Top Point with Load Increasing (Softening Model)](image)

The collapse occurred in compression in columns C3 as the applied compressive stresses are larger than the compressive strength of 8 MPa, as shown in figure 6-5. The collapse occurred in tension in beams B2 and B3 as the applied tensile stresses are larger than the tensile strength of 0.6 MPa, as shown in figure 6-6.
Figure 6-5: Collapse Mechanism in Compression (Softening Model)

Figure 6-6: Collapse Mechanism in Tension (Softening Model)
6-2-3 Smear Crack Model in Tension and Druker-Prager Model in Compression

The used method of solution is Newton with maximum number of iteration of 50 taking into account both Arc length control and line search. The load was applied incrementally as 20 steps each step equals to 0.5 of the self weight then 40 steps each step equals to 0.05 of the self weight. The convergence criteria are based on both displacement and force with tolerance of 0.01.

The maximum load factor reached was 10.70 (slightly less than the two previous examined models) as shown in figure 6-7 which represents the deflection of the last top point of the spire with the increasing in load increments. As expected it wasn't possible to capture clearly the nonlinear behavior of the model.

![Figure 6-7: Deflection of Last Top Point with Load Increasing (Drucker-Prager with Smear Crack Model)](image)

The collapse occurred in compression in columns C3 as the applied compressive stresses are larger than the compressive strength of 8 MPa, as shown in figure 6-8. The collapse occurred in tension in beams B2 and B3 as the applied tensile stresses are larger than the tensile strength of 0.6 MPa, as shown in figure 6-9.
Wind and Earthquake Analysis of Spire of Cimborio of Barcelona Cathedral

Figure 6-8: Collapse Mechanism in Compression (Drucker-Prager with Smeared Crack Model)

Figure 6-9: Collapse Mechanism in Tension (Drucker-Prager with Smeared Crack Model)
6-3 Results of Nonlinear Static Analysis of Seismic Actions

6-3-1 Introduction

A pushover analysis has been done in order to assess the behavior of the structure under a seismic load. This is a simplified analysis as it is the equivalent nonlinear static analysis of one macro element of the structure. An accurate analysis would be a nonlinear dynamic analysis in the time-domain, introducing a time-acceleration spectrum. However, this simplified analysis is accepted to assess the seismic performance of the structure [Juan Murcia, Seismic Analysis of Santa Maria del Mar Church in Barcelona, Msc thesis, SAHC 2008].

6-3-2 Ideal Plasticity Model

6-3-2-1 Analysis Procedure:

The applied action is a prescription of increasing displacements in X-direction according to the first natural mode shape of vibration, as it is the dominant vibration mode of the structure with mass participation factor of 49%. A more realistic prescription of displacements would be the combination of the first mode shape (participation factor of 49%), third mode shape (participation factor of 11%), tenth mode shape (participation factor of 10%) and thirteenth mode shape (participation factor of 13%) using the SRSS method of combination.

A phased analysis has been carried out. In the first phase the self weight of the spire was applied in two load increments, each of 50% of the self weight of the spire. In the second phased the prescribe displacements were applied at control points at each level. From level 44.07 to level 70.45 the eight corners of each level were chosen as the control points. From level 70.45 to level 75.00 the point of change of cross section of the base of the sculpture was chosen to be the control point as shown in figure 6-10.
In the second phase the displacements were applied as 10% equal increments (each 10% displacement is called a step) until non convergence occurred at 200% of the displacements (step 21).

6-3-2-2 Analysis Results:

The obtained capacity curve is plotted in figure 6-11. This curve represents the relationship between the lateral displacement of the last top point of the spire, at the end of each step, on the horizontal axis versus the summation of X-reaction (the base shear), at the same step, on the vertical axis.
Figure 6-11: Capacity Curve of the Spire (Ideal Plasticity Model)

From the curve it can be noticed that, the spire can carry at failure a maximum base shear of 0.228 of its self weight (600 KN) at which a maximum top displacement of 5 cm occurs. The tensile damage, positions of hinges, at the last step is shown in figure 6-12.

Figure 6-12: Tensile Damage at Last Step (Ideal Plasticity Model)
The capacity curve was idealized based on equal area criteria and the resulting curve is shown in figure 6-13.

![Figure 6-13: Idealization of Capacity Curve (Ideal Plasticity Model)](image)

6-3-2-3 The Role of Reinforcement

The reinforcement is highly needed to resist the earthquake loads as the results showed that in the cracked zones of beams (at these zones the normal stresses exceed the tensile strength of masonry 0.6 MPa) a composed section is formed in which the reinforcement is resisting tension and the masonry is resisting compression. The first reinforcement to work in tension was the reinforcement of beam B9 at step 3 at which only 20% of the prescribed displacements have been applied as shown in figure 6-14 and 6-15, this result makes sense because the linear elastic analysis showed that this reinforcement is the highest tensioned reinforcement as shown in figure 6-16. A plot of normal stresses in all reinforcements at the last step is shown in figure 6-17 and 6-18, from this figure it can be shown that all reinforcements are highly needed to resist very high values of tensile stresses as the maximum tensile stress reaches a value of 272 MPa which represents 97% of the yield strength of reinforcement (280 MPa).
Figure 6-14: Normal Stresses in Reinforcement of B2, B4, B6 and B8 at Step 3

Figure 6-15: Normal Stresses in Reinforcement of B3, B5, B7 and B9 at Step 3
Figure 6-16: Normal Stresses of Linear Elastic Analysis in All Reinforcement

Figure 6-17: Normal Stresses in Reinforcement of B2, B4, B6 and B8 at Last Step
6-3-3 Softening Model

6-3-3-1 Analysis Assumptions and Procedure

The same loading procedure used in the first pushover analysis with ideal plasticity model was used for the softening model also but for this model non convergence occurred at only 140% of the displacements (step 15).

6-3-3-2 Analysis Results

Again the obtained capacity curve is plotted in figure 6-19. Comparing this curve with the curve of ideal plasticity model we find that there is a reduction of 30% \( \frac{0.228 - 0.16}{0.228} \) in the maximum load at failure and an increase in the maximum displacement of 144% \( \frac{12.2 - 5.0}{5.0} \). These results are reasonable because this model is more fragile than the first one and more representative for masonry behavior. Again the tensile damage, positions of hinges, at the last step is shown in figure 6-20.
Wind and Earthquake Analysis of Spire of Cimborio of Barcelona Cathedral

Figure 6-19: Capacity Curve of the Spire (Softening Model)

Figure 6-20: Tensile Damage at Last Step (Softening Model)
The failure in the softening model occurs just after a small part of the span start to crack, not like the plasticity model in which nearly 1/3 of the span is damaged at failure.

The capacity curve was idealized based on equal area criteria and the idealized curve with the original one are shown in figure 6-21.

![Figure 6-21: Idealization of Capacity Curve (Softening Model)](image)

6-3-3-3 The Role of Reinforcement

The same role of reinforcement for ideal plasticity model is confirmed in the softening model, the reinforcement is highly needed to resist the earthquake loads. Like the ideal plasticity model the first reinforcement to work in tension was the reinforcement of beam B9 at step 3 at which only 20% of the prescribed displacements have been applied as shown in figure 6-22 and 6-23. A plot of normal stresses in all reinforcements at the last step is shown in figure 6-24 and 6-25, from this figure it can be shown that reinforcements are resisting higher values of tensile stresses compared to ideal plasticity model, which makes sense, as the highest tensile stress in this case is 287 MPa.
Wind and Earthquake Analysis of Spire of Cimborio of Barcelona Cathedral

Figure 6-22: Normal Stresses in Reinforcement of B3, B5, B7 and B9 at Step 3

Figure 6-23: Normal Stresses in Reinforcement of B3, B5, B7 and B9 at Step 3
Figure 6-24: Normal Stresses in Reinforcement of B2, B4, B6 and B8 at Last Step

Figure 6-25: Normal Stresses in Reinforcement of B3, B5, B7 and B9 at Last Step
Chapter 7

Conclusions and Future Works

7-1 Introduction

This chapter will discuss the final conclusions derived from the work done and also will propose some future work in the analysis of the spire.

7-2 Conclusions

The spire of Barcelona Cathedral was built at the beginning of the 20th century, at that time the reinforced concrete was starting to be the prevailing construction material and the concept of using steel reinforcement was well established, so it was a good decision from the two architects Josep Oriol Mestres and August Font i Carreras to reinforce the stone masonry beams with central steel ties.

Due to the coastal weather of Barcelona city which is full of rains and also because of sea spray and pollution products the steel ties are suffering from sever corrosion which caused not only wide visible cracks in masonry beams but also caused large parts of beams to be detached and it's now temporary kept in position by some ropes.

As a result of steel ties corrosion it was essential to carry out the current restoration project in which the spire will be dismantled and reconstructed using this time titanium ties instead of steel ties to eliminate the corrosion problem.

The height of the spire which reaches 90 m above ground level makes it very sensitive to lateral loads of wind and earthquake and hence makes it subjected to high values of tensile stresses. From the linear elastic analysis only it wasn't possible to clarify the strength contribution of steel ties because of the high tensile stresses found in masonry beams which are not found in reality and another distribution of stresses actually exist in the spire. But from this analysis it was possible to approximately compare the two cases of reinforced and unreinforced spire and to see the effect of steel ties on deformation, reactions, straining actions and stresses distribution in the linear stage only and no significant differences found between the two cases.

Analyzing the spire in the nonlinear stage first under its self weight (assuming more than one constitutive model) gave a load factor more than 10, which is very satisfactory, and indicated the possible compressive collapse which may start from column C3 and the possible tensile collapse which may start from beams B2 and B3.

The nonlinear static pushover analysis (using seismic action in the form of prescribed displacements at control points at different levels) showed that the steel ties are start to work in tension at low values of seismic actions (almost at only 30% of the applied displacements) and showed that the tensile stresses at the maximum base shear are near the yield strength.

This analysis also showed that the spire can carry 16% of its weight as a lateral load which is also considered satisfactory compared with the actual applied earthquake loads (calculated by the lateral force method) which is 10% of its weight.
7-3 Future Works

The following are some of the future works that can be done to have more detailed information about the spire seismic behavior.

The seismic actions in pushover analysis can be in the form of loads proportional to the mass, loads proportional to the mass and first mode of vibration and loads proportional to the mass and combination of the dominant modes of vibration (modes 1, 3, 8, 10 and 13).

A dynamic non linear analysis in time domain can give more detailed information about the seismic behavior of the spire.

The Drucker-Prager model in compression with smeared cracks model in tension can be examined with the two previously mentioned analyses.

Sensitivity analysis can be carried out to compare some reasonable assumed values of fracture energy in tension and tensile strength.

Making use of the obtained capacity curves, the capacity spectrum method can be applied and compared with the nonlinear analysis results.
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