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DEPARTMENT OF CIVIL, ARCHITECTURAL AND ENVIRONMENTAL ENGINEERING

THE ANALYSIS OF THE WIND EFFECTS ON THE TALL STEEL RETICULAR STRUCTURE: CASE STUDY OF A 70m TALL TELECOMMUNICATION TOWER AT MAKEPE IN DOUALA

Thesis submitted in partial fulfilment of the requirement for the degree of Masters in Engineering

Curriculum: Civil Engineering

Presented by:

MBALLA MEKONGO HENRI OLIVIER

Student Matricula number: **15TP20998**

Supervised by:

Prof. Carmelo MAJORANA

Co-supervised by:

Dr. Eng. Guillaume Hervé POH'SIE

Eng. Giuseppe CARDILLO

Academic year: 2019/2020

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DEDICATION

To my beloved mother, **NKODO MBODO CATHERINE**

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LIST OF ABBREVIATIONS AND SYMBOLS

C_t, global drag coefficient

q, dynamic pressure

S_p, surface area exposed to wind

S, surface area of panel

φ, surface area ratio

q_{10}, basic dynamic pressure at 10 m height

K_s, site effect

K_m, mask effect

δ, dimension effect

β, dynamic amplifying factor

K_h, height factor

T_d, drag force

H, height of the structure above the foundation.

θ is the global coefficient depending on the type of construction

ξ, response coefficient

T, period of fundamental mode of oscillation

τ pulsation coefficient

FH, faisceau hertzien

WCDMA, wideband code division multiple access

GSM, global system for mobile

S_{eq}, effective area S_{eq}

C_{ti}, drag coefficient as a function of its azimuth

S_{eq}, equivalent surface area

F_{eq}, equivalent force on antennas

SLS, serviceability limit state

ULS, ultimate limit state

A, area of cross-section of the members

N_{Ed}, axial force

N_{Rd}, resisting axial force

f_{yk}, characteristic steel yield strength

f_{yd}, design steel yield strength

$N_{b,Rd}$, buckling resistance force

χ, the buckling factor

N_{cr}, elastic critical force for the relevant buckling mode

E, young's modulus

I, moment of inertia

l_0, buckling length

F_{Ed}, shear force

F_{Rd}, resisting shear force

f_{ub}, ultimate tensile stresses of the bolt

A_b, gross area of the bolt

γ_{Mb}, partial safety factor for bolts

$\sigma_{0.2}$, proof strength of bolt

As, area of steel reinforcement

M_{ED}, bending moment

M_{RD}, resisting bending moment

$F_{b,Rd}$, bearing resistance of plate

f_u, is the ultimate tensile strength for the plate

d_0, is the hole diameter

d, diameter of bolt

t, thickness of plate

f, displacement

f_{cd}, design concrete strength

N_a, lifting force on the bolt,

L_s, anchor length,

σ_{sol}, allowable bearing capacity of the footing.

σ_{adm}, allowable bearing capacity of the soil.

Υ_C, specific weight of concrete

M_R, overturning moment

M_S, stabilising moment

ABSTRACT

The main objective of this work was to analyze the effects of wind on a tall telecommunication tower. The case study for this thesis was a 70m tall self-supporting steel tower of 4 legs made up of schifflerized angle sections of equal wings, each leg being 5m away from the other. The structure was modelled using the computer software SAP2000 v22. A preliminary design which consists in giving dimensions to each element of the structure according to constructive provisions and based on existing structures was made which permits us to start evaluating the wind loads on our structure since the wind loads are a function of the geometric characteristics of each element (member leg, the primary and secondary bracing elements) thus many iterations are made to obtain the actual wind loads on the sections finally considered. The wind loads were calculated following the NV65 rules. The solicitations on the steel elements were determined and the elements were designed following the Eurocode to avoid instability on the elements and the connections between them. The overall displacement of the tower was determined and the maximum displacement observed was 32cm at the top of the tower. The wind loads have an uplifting effect on the tower structure which has to be resisted. For this reason, we have designed anchor bolts of sufficient thickness and length that enter the ground level to overcome this lifting effect due to wind. The next step was to design the foundation of our structure and to do so we first design a column-base plate in order to connect the steel structure to the footing made up of concrete. The first type of foundation designed was a flexible plinth, the sections, the thickness and the depth of the footing were determined and a structural stability verification was carried. As result, we've seen that the structure was unstable and that the foundation used was not appropriate. The instability was due to a very big overturning moment acting on our structure, this overturning moment is caused by the overall action of wind on our tower. A foundation of higher weight had to be designed in order to overcome this instability hence increasing the stabilising moment on the tower. The foundation used was a raft footing which has been designed, the structural stability verification has been carried and it was observed that the structure was stable.

Keywords: Wind loads, reticular structures, telecommunication towers, stability.

RESUME

L'objectif principal de ce travail était d'analyser les effets du vent sur un pylône de télécommunication. Le cas étudié dans le cadre de notre travail est un pylône autostable en acier de 70 m de haut dotée de 4 pieds et faite de cornières schifflerisées d'ailes égales, chaque pied étant distant de 5 m de l'autre. La structure a été modélisée à l'aide du logiciel SAP2000 v22. Un avant-projet qui consiste à dimensionner chaque élément de la structure selon les dispositions constructives et sur la base des structures existantes a été réalisé ce qui nous permet de commencer à évaluer les charges de vent sur notre structure puisque les charges de vent sont fonction des caractéristiques géométriques de chaque élément (les membrures, les éléments de contreventement primaires et secondaires), ainsi de nombreuses itérations ont été effectuées pour obtenir les charges de vent réelles sur les sections finalement considérées. Les charges de vent ont été calculées selon les règles NV65. Les sollicitations sur les éléments en acier ont été déterminées et les éléments ont été dimensionnés selon l'Eurocode pour éviter l'instabilité et les connexions entre les éléments ont été déterminées. Le déplacement global du pylône a été déterminé et le déplacement maximum observé était de 32cm au sommet du pylône. Les charges du vent ont un effet de soulèvement sur la structure du pylône, auquel il faut résister. C'est pourquoi nous avons calculé des boulons d'ancrage d'une épaisseur et d'une longueur suffisantes qui pénètrent dans le sol pour résister à cet effet de soulèvement dû au vent. L'étape suivante a été de concevoir la fondation de notre structure et pour ce faire, nous avons d'abord utilisé une platine en pied de membrures afin de relier la structure en acier à la semelle en béton. Le premier type de fondation utilisé était une semelle flexible, les sections, l'épaisseur et la profondeur de la semelle ont été déterminées et une vérification de la stabilité structurelle a été effectuée. Le résultat que nous avons constaté est que la structure était instable et que la fondation utilisée n'était pas appropriée. L'instabilité était due à un très grand moment de renversement agissant sur notre structure, ce moment de renversement est causé par l'action globale du vent sur notre pylône. Une fondation de poids plus élevé a dû être conçue afin de surmonter cette instabilité en augmentant le moment stabilisant sur le pylône. La fondation utilisée est un radier plein qui a été dimensionné, la vérification de la stabilité structurelle a été effectuée et il a été observé que la structure était stable.

Mots clés : Charges de vent, structures réticulées, pylônes de télécommunication, stabilité.

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GENERAL INTRODUCTION

In the past decade, an increasing number of tall buildings and complex structures such as the Burj Khalifa in Dubai, the Bird's Nest Stadium in Beijing and the London Aquatic Center were built. These projects demand that the civil and structural engineers have the ability to handle the increasing difficulty in designing even more complicated projects such as tall buildings or structures with more complex geometries. The lateral loading due to wind or earthquake is the major factor that causes the design of high-rise structure to differ from those of low-to-medium-rise structure. Tall structures have become more efficient, lighter and consequently, more prone to deflect and even to sway under wind loads.

Nowadays, telecommunication is rapidly expanding all over the world. Almost every country can enjoy this technology. This rise has led to the constructions of many telecommunication towers and hence an increasing number of accidents due to the collapse of these structures. Structural failures in these structures especially those of great heights can cause considerable loss of human life and material damages and thus special care must be taken when designing and constructing them.

The main objective of this work is to analyze the wind effects on a tall steel telecommunication tower. However, this type of light and slender structure is sensitive to the effects of instability due to general climatic stresses. It is therefore necessary to ensure that the stability of these structures is maintained during their design. This requires a precise analysis and verification of the various instability problems.

In order to achieve this objective, the study is divided in three parts. The first part (Chapter 1) consists of a literature review on steel, a clever understanding of concepts of reticular structures, telecommunication towers and wind loads on structures. The second chapter, methodology to perform the structural analysis and design of the tower 'structure is presented. Finally, the results obtained for the analysis and design procedure are presented in the third chapter, analysis and interpretation of results followed by a general concluding statement.

CHAPTER 1: LITERATURE REVIEW

Introduction

This chapter aims to provide sufficient knowledge on fundamental concepts inherent to the problematic of the thesis with the help of the work already carried out by other authors on the subject. To achieve this objective, available literature on steel which is the main material used for this kind of structure is presented. In addition, we will look at the concept of reticular structures regarding the analysis methods and the kind of connections used as well as their common domain of application. Furthermore, we will focus on telecommunication towers, the different types that exist, the mounting methods and also look at some accidents that have occurred. Finally, we will discuss about some important aspects to be taken into account when considering the action of wind on structures.

1.1. Steel

By far the most widely used material for building the world's infrastructure and industries, it is used to fabricate everything from sewing needles to oil tankers. In addition, the tools required to build and manufacture such articles are also made of steel. A clear aspect of steel will be studied reviewing its raw materials, its obtention, properties, field of application, defects and the defects treatments.

1.1.1. Steel raw materials

Steel is an alloy consisting mostly of iron and less than 2% carbon, 1% manganese and small amounts of silicon, phosphorus, sulphur and oxygen. The three raw materials used in making pig iron (which is the raw material needed to make steel) are the processed iron ore, coke (residue left after heating coal in the absence of air, generally containing up to 90% carbon) and limestone ($CaCO_3$) or burnt lime.

1.1.2. Manufacture of steel

There are 5 different processes of making steel namely the Bessemer process, the Siemens-Martins process, Thomas process, the basic oxygen process and the electric-arc process. The most recent and used processes are the basic oxygen and the electric-arc processes.

1.1.2.1. Basic oxygen steelmaking

More than half the world's steel is produced in the basic oxygen process (BOP), which uses pure oxygen to convert a charge of liquid blast-furnace iron and scrap into steel. The basic oxygen furnace (BOF) is a refractory-lined, tiltable converter into which a vertically movable, water-cooled lance is inserted to blow oxygen through nozzles at supersonic velocity onto the charge. The use of pure oxygen at high flow rates results in such fast oxidation of the elements contained in blast-furnace iron that only about 20 minutes are required per heat i.e., to refine one charge. Converters vary in size and are operated for heats ranging from 30 to 360 tons (Wondris et al, 2019).

1.1.2.2. Electric-arc steelmaking

About one-quarter of the world's steel is produced by the electric-arc method, which uses high-current electric arcs to melt steel scrap and convert it into liquid steel of a specified chemical composition and temperature. External arc heating permits better thermal control than does the basic oxygen process, in which heating is accomplished by the exothermic oxidation of elements contained in the charge. This allows larger alloy additions to be made than are possible in basic oxygen steelmaking. However, electric-arc steelmaking is not as oxidizing, and slag-metal mixing is not as intense; therefore, electric-arc steels normally have carbon contents higher than 0.05 percent. In addition, they usually have a higher nitrogen content of 40 to 120 parts per million, compared with 30 to 50 parts per million in basic-oxygen steels. Nitrogen, which renders steel brittle, is absorbed by liquid steel from air in the high-temperature zone of the arc. The nitrogen content can be lowered by blowing other gases into the furnace, by heating with a short arc, and by applying a vigorous carbon monoxide boil or argon stir to the melt (Wondris et al, 2019).

1.1.3. Properties of steel

These properties can be distinguished in 3 groups which are physical properties, mechanical properties and chemical properties.

1.1.3.1. Physical properties of steel

Steel has an attractive outer appearance. It is silvery in colour with a shiny, lustrous outer surface. Others Physical properties are seen in Table 1.1.

Table 1.1.Physical properties of steel

Physical properties	Value
density, ρ (kg/m^3)	7,850
Specific Heat Capacity, c_p (J/(kg-K)	502.416
thermal conductivity, k (W/Km)	45
Coefficient of thermal expansion, α (/°C)	0.312 x 10-6
Resistivity at 20 °C; ρ (Ω•m)	4.60×10−7
Conductivity at 20 °C; σ (S/m)	2.17×106
Permeability (μ)-(H/m):	1.26×10−4
Relative permeability (μ / μ0):	100

1.1.3.2. Mechanical properties

One of the useful mechanical properties of steel, is its ability to change shape on the application of force to it, without resulting in a fracture. This property is known as ductility. Steel is equally malleable and can be deformed under compression. Others mechanical properties are seen in Table 1.2.

Table 1.2.Mechanical properties of steel

Mechanical properties	Value
Modulus of elasticity, E (GPA)	210
shear modulus, G (GPA)	81
Poisson's ratio, ν	0.3
Elongation percentage (%)	5-40

1.1.3.3. Chemical properties

The hardness of this alloy is high, reflecting its ability to resist strain. It is long-lasting and greatly resistant to external wear and tear. Hence it is considered a very durable material. Also, concerning the rust resistance, the addition of certain elements, makes some types of steel resistant to rust. Stainless steel for instance contains nickel, molybdenum and chromium which improve its ability to resist rust.

1.1.4. Typology of steels

Steel is graded as a way of classification and is often categorized into four groups namely carbon, alloy, stainless, and tool.

1.1.4.1. Carbon steel

Carbon steels are by far the most produced and used, accounting for about 90 percent of the world's steel production. They are usually grouped into high-carbon steels, with carbon above 0.5 percent; medium-carbon steels, with 0.2 to 0.49 percent carbon; low-carbon steels, with 0.05 to 0.19 percent carbon; extra-low-carbon steels, with 0.015 to 0.05 percent carbon; and ultralow-carbon steels, with less than 0.015 percent carbon. Carbon steels are also defined as having less than 1.65 percent manganese, 0.6 percent silicon, and 0.6 percent copper, with the total of these elements not exceeding 2 percent (Wondris et al, 2019).

1.1.4.2. Alloy steel

Low-alloy steels have up to 8 percent alloying elements; any higher concentration is considered to constitute a high-alloy steel. There are about 20 alloying elements besides carbon. These are manganese, silicon, aluminum, nickel, chromium, cobalt, molybdenum, vanadium, tungsten, titanium, niobium, zirconium, nitrogen, sulfur, copper, boron, lead, tellurium, and selenium. Several of these are often added simultaneously to achieve specific properties (Wondris et al, 2019).

1.1.4.3. Stainless steel

This outstanding group receives its stainless characteristics from an invisible, self-healing chromium oxide film that forms when chromium is added at concentrations greater than 10.5 percent. There are three major groups, the austenitic, the ferritic, and the martensitic. The best corrosion resistance is obtained in austenitic stainless steels. Their microstructures consist of very clean face centered cubic crystals in which all alloying elements are held in solid solution. These steels contain 16 to 26 percent chromium and up to 35 percent nickel, which, like manganese, is a strong austenizer. (Indeed, manganese is sometimes used instead of nickel.) Austenitic steels cannot be hardened by heat treatment; they are also nonmagnetic (Wondris et al, 2019).

1.1.4.4. Tool steel

Generally, they are very hard, wear-resistant, tough, inert to local overheating, and frequently engineered to particular service requirements. They also have to be dimensionally stable during hardening and tempering. They contain strong carbide formers such as tungsten, molybdenum, vanadium and chromium in different combinations and often cobalt or nickel to improve high-temperature performance (Wondris et al, 2019).

1.1.5. Field of application of steel

Steel is used in a variety of domains, some of which are; building and infrastructure, automotive, transport, energy, tools and machinery.

1.1.5.1. Buildings and Infrastructure

The possibilities for using steel in buildings and infrastructure are limitless. The most common applications are listed below:

- Structural sections provide a strong, stiff frame for the building and make up 25% of the steel use in buildings;
- Reinforcing bars add tensile strength and stiffness to concrete and make up 44% of steel use in buildings. Steel is used because it binds well to concrete, has a similar thermal expansion coefficient and is strong and relatively cost-effective;
- 31% is in sheet products such as roofing, purlins, internal walls, ceilings, cladding, and insulating panels for exterior walls;
- Steel is required for bridges, tunnels, rail track and in constructing buildings such as fuelling stations, train stations, ports and airports. About 60% of steel use in this application is as rebar and the rest is sections, plates and rail track.[1]

1.1.5.2. Automotive

The steel in a vehicle is distributed as follows, based on total vehicle curb mass:

- 40% is used in the body structure, panels, doors and trunk closures for high-strength and energy absorption in case of a crash;
- 23% is in the drive train, consisting of cast iron for the engine block and machinable carbon steel for the wear resistant gears;
- 12% is in the suspension, using rolled high-strength steel strip;
- The remainder is found in the wheels, tyres, fuel tank, steering and breaking systems.[2]

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[1] https://www.worldsteel.org/steel-by-topic/steel-markets/buildings-and-infrastructure.html
[2] https://www.worldsteel.org/steel-by-topic/steel-markets/automotive.html

1.1.5.3. Transport

Including automotive, around 16% of steel produced worldwide is used to meet society's transport needs. Steel is also essential to the related infrastructure: roads, bridges, ports, stations, airports and fueling. Some major applications include:

- Ship building traditionally uses structural steel plate to fabricate ship hulls. Modern steel plates have much higher tensile strengths than their predecessors, making them much better suited to the efficient construction of large container ships;
- Rail transport requires steel in the trains and for the rails and infrastructure. Steel makes up 15% of the mass of high-speed trains and is essential. The main steel components of these trains are bogies;
- For aeroplanes Steel is required for the engines and landing gear [3].

1.1.5.4. Tools and Machinery

Tools and machinery cover a wide range of equipment from small workshop tools to large factory-based robotic machinery and rolling mills. In 2017, tools and machinery represented approximately 15 % of global steel use.[4]

1.1.5.5. Energy

Steel is and will be critical for supplying the world with energy, whether based on fossil fuels, nuclear technology, or renewable sources like wind, solar or geothermal. Whatever the source, steel has a crucial role to play in producing and distributing energy as well as improving energy efficiency. Some of its applications are;

- Steel is used in nuclear and fossil fuel-based energy that is mining equipment, offshore oil platforms, equipment for oil and gas extraction and production, pipelines for the distribution of natural gas and oil, storage tanks, power plants;
- Steel is used for the production and distribution of electricity; transformers (magnetic steel core), generators and electric motors, power distribution pylons and steel-reinforced cables.[5]

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[3] https://www.worldsteel.org/steel-by-topic/steel-markets/transport.html
[4] https://www.worldsteel.org/steel-by-topic/steel-markets/tools-and-machinery.html
[5] https://www.worldsteel.org/steel-by-topic/steel-markets/energy.html

1.1.6. Defects of steel

Steel oxidizes in the air (atmospheric corrosion), especially if the air is very humid. They are also attacked by contact with more or less aggressive materials. lime parts are more easily attacked than forged parts.

Steel is attacked by plaster. It is therefore absolutely necessary to protect these metals with a protective layer. Before applying any protective layer, the metal must be free of all traces of rust and grease (Mbessa,2005).

1.1.7. Steel defect treatment

Steel will rust only if water and oxygen are both present and the rusting process is greatly accelerated by pollutants in the atmosphere, such as sulphur dioxide from the burning of oil, coal, or gas and chlorides from de-icing salts or marine atmospheres. To protect steel coatings can be applied such as paint coatings, metallic coatings and enamel coatings (NPL,2020).

1.1.7.1. Paint coatings

Conventionally, protective paint systems consist of primer, undercoat(s) and finish coats. However, there are now available single coat systems that combine primer and finish coats.

- The primer is applied directly onto the cleaned steel surface. Its purpose is to wet the surface and to provide good adhesion for subsequently applied coats. In the case of primers for steel surfaces, these are also usually required to provide corrosion inhibition.
- The undercoats (or intermediate coats) are applied to 'build' the total film thickness of the system. Generally, the thicker the coating the longer the life. This may involve the application of several coats. Undercoats are specially designed to enhance the overall protection and, when highly pigmented, decrease permeability to oxygen and water.
- The finish provides the required appearance and surface resistance of the system. Depending on the conditions of exposure, it must also provide the first line of defense against weather and sunlight, open exposure, condensation (as on the undersides of bridges), highly polluted atmospheres in chemical plant, impact and abrasion at floor or road level, and bacteria and fungi (NPL,2020).

1.1.7.2. Metallic coatings

The two most commonly used methods of applying metallic coatings to structural steel are hot-dip galvanizing and thermal (metal) spraying. In general, the corrosion protection afforded by metallic coatings is largely dependent upon the choice of coating metal and its thickness and is not greatly influenced by the method of application (NPL,2020).

1.1.7.3. Vitreous enamel coating

Enamel refers to a glassy, vitreous and usually opaque substance that is used in protective or decorative coating on metal, glass or ceramic ware. It is bonded to the steel substrate by thermal fusion. This coating is applied for the protection of steel products from surrounding environments. This coating provides not only an aesthetic exterior but also provides outstanding engineering properties, such as mechanical strength of the enameled surface, multiplicity and stability of colour, corrosion resistance, resistance to wear and abrasion, chemical and heat resistance, resistance to thermal shock and fire, hygiene and ease of cleaning.

1.2. Reticular truss structures

A truss is an assembly of straight or curved bars biarticulated at their ends, which forms a stable structure. For their lightness and strength, trusses are widely used to solve the problems of range, resistance and aesthetics. Trusses are used in bridges, floors, factories, large sports and conference halls, and domes, etc. The bars of a truss are joined by bolts or rivets through a plate called a "gusset". When the bars of a structure are on a single plane, the truss is said to be plane. In general, trusses are spatial and have a perfect (rigid) connection between their bars. In many cases, bridge or firm structures are considered to be plane structures, meaning they can be analyzed without influencing the accuracy of the results obtained **(**Khalfallah, 2018).

1.2.1. Hypothesis of truss analysis

The analysis of trusses is based on the following simplified assumptions:

- all the bars of the plane or space structures are biarticulated at their ends;
- the external forces are exclusively applied to the joints of the truss;
- the axis of each bar coincides with the line between the centers of two adjacent joints (Figure 1.1).

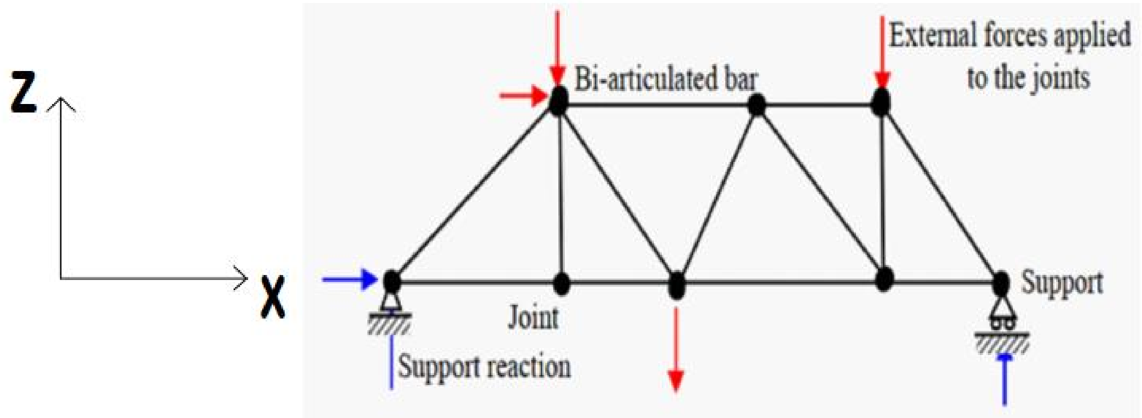


Figure 1.1. Model of a truss (khalfallah,2018)

In trusses, the loading system only generates forces on the structure's bars. Each internal force can be a traction, a compression or a zero force (Khalfallah, 2018).

1.2.2. Analysis methods of trusses

The primary purpose of truss analysis is to determine the internal forces on the bars. In the literature, there are several analysis methods; method of joint equilibrium, method of sections (Ritter's method), matrix method and graphic or cremona method. We will briefly discuss about the method of joint equilibrium and the method of sections (Khalfallah, 2018).

1.2.2.1. Method of joint equilibrium

In the method of joint equilibrium, the internal forces on the bars of a truss can be determined by considering the equilibrium of each joint. The equilibrium of the whole structure makes it possible to check the overall equilibrium. At each joint, the forces on the bars and the nodal forces and/or the support reactions must satisfy the equilibrium equations 1.1 and 1.2.

$$\sum F_X = 0 \tag{1.1}$$

$$\sum F_z = 0 \tag{1.2}$$

Equations 1.1 and 1.2 are checked at each joint. For this reason, it is recommended to start the analysis with the joint having a maximum of two bars. By successively applying the equilibrium equations at each joint, we can then determine the forces on the bars (Khalfallah, 2018).

1.2.2.2. Method of sections

The method of sections consists of cutting the bars by a section, whose internal forces are the unknowns of the problem. The section divides the truss into two portions each of which must satisfy the equilibrium conditions in equations 1.3, 1.4, and 1.5 (Khalfallah, 2018).

$$\sum F_X = 0 \tag{1.3}$$

$$\sum F_z = 0 \tag{1.4}$$

$$\sum M_i = 0 \tag{1.5}$$

1.2.3. Types of connections

Connections of structural steel members are of critical importance. An inadequate connection, which can be the "weak link" in a structure, has been the cause of numerous failures. Failure of structural members is rare, most structural failures are the result of poorly designed or detailed connections. Modern steel structures are connected by welding or bolting or by a combination of both.

1.2.3.1. Bolted connections

The most common method of joining one component to another in structural steelwork is bolting (Figure 1.2). Bolting may be carried out either in the shop or on site and has the advantage that the components can be separated easily should this become necessary for any reason. For site connections, however, bolting is virtually the universal medium of connection. The main function of the bolt is to transmit a force from one member to another.

Bolts have hexagonal heads and nuts, parallel shanks and threads cut or rolled into the shanks. They come in standard shank diameters of 12 mm, 16 mm, 20 mm and 24 mm in a large range of lengths and in various grades of strength. Bolts are designated by size, i.e., the nominal diameter of the shank and thread, and by length, i.e., the total length of the shank (including thread) up to the underside of the head. The bolt sizes mentioned above are designated M12, M16, M20 and M24 (M means metric). Here the British and the European Codes have common designations. The American designations are as follows (Bangash, 2000):

- ASTM -A307 Grade A and B (unfinished bolts) into 4 in. (6·4 mm to 100 mm);
- ASTM A502 -Ribbed type grades;

- ASTM A325 -Bearing type, same as ASTM A307 for sizes including 16 mm;
- ASTM A325-A490 -High-strength bolts 5/8 in. to 1 in. (16 mm to 25.4 mm).

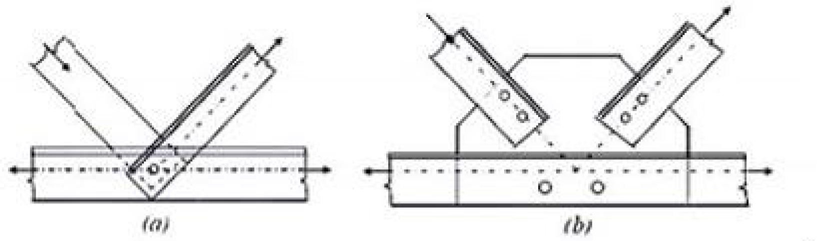


Figure 1.2. Bolted connections (Bangash, 2000)

1.2.3.2. Welded connections

Structural welding is a process whereby the parts to be connected are heated and fused, with supplementary molten metal added to the joint. For example, the tension member lap joint shown in Figure1.3 can be constructed by welding across the ends of both connected parts. A relatively small depth of material will become molten, and upon cooling, the structural steel and the weld metal will act as one continuous part where they are joined. The additional metal, sometimes referred to as filler metal, is deposited from a special electrode, which is part of an electrical circuit that includes the connected part, or base metal (Segui, 2013).

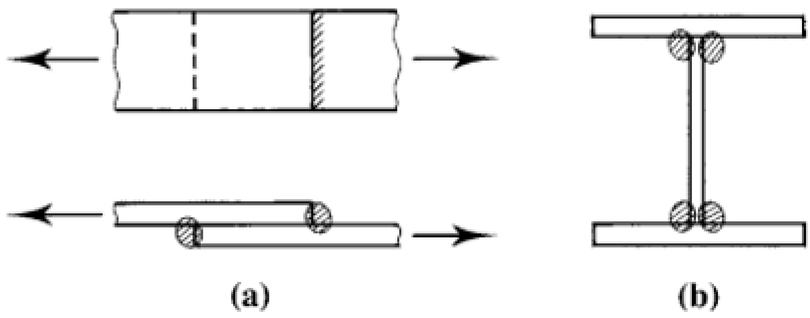


Figure 1.3. Welded connections (Segui, 2013).

The two most common types of welds are the fillet weld and the groove weld. fillet welds are defined as those placed in a corner formed by two parts in contact. Fillet welds can also be used in a tee joint. Groove welds are those deposited in a gap, or groove, between two parts to be connected. They are most frequently used for butt, tee, and corner joints. In most cases, one or both of the connected parts will have beveled edges, called prepared edges, although relatively thin material can be groove welded with no edge preparation (Segui, 2013).

1.2.4. Common applications of reticular truss structures

Reticular structures can be used in several types of structures, some of which are tower cranes, truss bridges, truss towers and roof trusses.

1.2.4.1. Tower cranes

Tower cranes (Figure1.4) are devices that are used for vertical and horizontal handling of the load. They can be used in every industry, most often in the civil engineering industry. The crane structure is a spatial truss structure which must be resistant to the load of the entire crane (base crane structure, load, cab, etc.) and side effects (operating shocks, wind, etc.). This results in requirements for the construction of the crane, which must be sufficiently rigid, strong, light, safe, yet cheap to manufacture and to transport easily (Monka, 2015). A working arm is long horizontal jib that is part of a crane carrying a load. There is a trolley connected to the jib that moves the load along the arm. To the trolley is attached a hook that includes motor to be the load lifting controlled. The engines and electronics of the crane, as well as large counterweight weights are located on the shorter arm. The motors that drive the rotary unit are located above the large unit transmission. The hook is placed on a long horizontal arm for lifting the load, which also includes its engine (Hric et al. 2019).

Figure 1.4. Tower crane (Skinner et al, 2005)

1.2.4.2. Truss bridge

Truss bridge, bridge with its load-bearing structures composed of a series of wooden or metal triangles, known as trusses. Given that a triangle cannot be distorted by stress, a truss gives a stable form capable of supporting considerable external loads over a large span. Trusses are popular for bridge (Figure 1.5) building because they use a relatively small amount of material for the amount of weight they can support. They commonly are used in covered bridges, railroad bridges, and military bridges. The individual pieces of a truss bridge intersect at truss joints, or panel points. The connected pieces forming the top and bottom of the truss are referred to respectively as the top and bottom chords. The sloping and vertical pieces connecting the chords are collectively referred to as the web of the truss.[6]

Figure 1.5. Truss bridge (Santosh, 2018)

1.2.4.3. Truss tower

Truss tower is a freestanding vertical framework tower. This construction is widely used in transmission towers carrying high voltage electric power lines, in radio masts and towers (a self-radiating tower or as a support for aerials) and in observation towers. Its advantage is good shear strength at a much lower weight than a tower of solid construction would have as well as lower wind resistance. In structural engineering the term lattice tower is used for a

[6] https://www.britannica.com/technology/truss-bridge#ref343099

freestanding structure, while a lattice mast is a guyed mast supported by guy lines. An example of a truss tower is the Eiffel tower in Paris (Figure1.6).

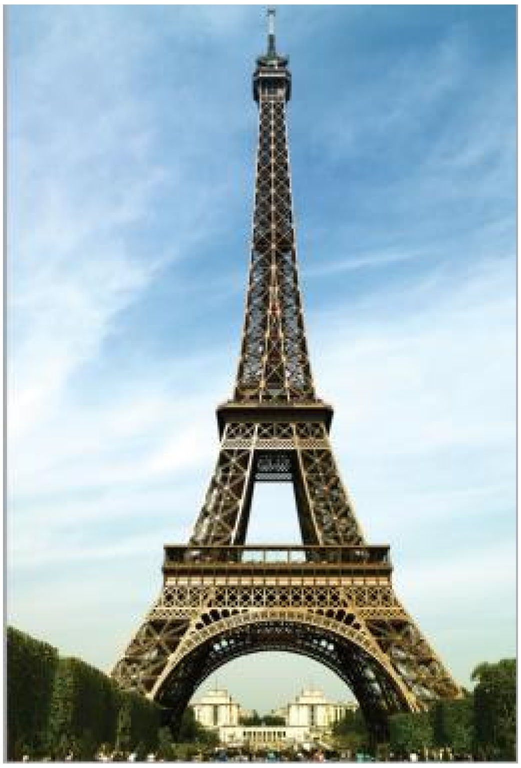


Figure 1.6.Eiffel tower Paris (Woog, 2014).

1.2.4.4. Roof truss

Roof trusses (Figure 1.7) are characterised by an economic use of construction materials (timber, steel). Composed of individual lightweight pieces, a truss can also provide considerable advantage in transport and assembly as compared to conventional roof structures. On the other hand, trusses are more labour-intensive and require connection devices. However, if a greater number of identical trusses can be manufactured, then considerable economies of scale can be achieved. The structural height of a truss is usually larger than the height of similar structures using solid beams. For roofs, however, this is usually no disadvantage as roofs must often depending on roof cover material used be higher at the ridge and lower at the eaves to facilitate roof drainage and ensure water tightness (Bieler et al, 1999).

Figure 1.7. Steel roof truss in car repair center.[7]

1.3. Telecommunication tower

Also known as cell towers are structures where electric communications equipment and antennae are mounted, allowing the surrounding area to use wireless communication devices like telephones and radios. Cell towers are usually built by a tower company or a wireless carrier when they expand their network coverage or capacity, providing a better reception signal in that area. In this chapter we will see the different types of cell towers and the different standards for the design of towers.

1.3.1. Types of cell towers

There are actually four different types of cell towers namely the self-supporting or lattice tower, the guyed tower, the monopole tower and the stealth tower.

1.3.1.1. Self- supporting tower

Self -supporting Towers may be either square or triangular in cross section. It usually has three or four sides with similar shaped bases. The tower members are made of schifflerized angles or tube sections suitably protected by hot-dip galvanizing. While it is usually more economical

[7]https://www.123rf.com/photo_122661201_steel-roof-truss-in-car-repair-center-steel-roof-frame-under-construction-the-interior-of-a-big-indu.html

to use a triangular cross section, there are situations where a square cross section is a better choice. The principal structural elements are the legs, the web bracing in each face, and if required for stability, horizontal diaphragm bracing. The legs are usually sloped (tapered) to provide adequate strength and stability as the height increases. The degree of slope is an option of the designer to suit the equipment supported, the required rigidity and the available land area. The slope is sometimes varied within a tower to maintain a desirable balance between the costs of leg members and bracing, or to reduce the foundation loads. Frequently the legs in the top section of the tower will be parallel to simplify the mounting of equipment.

There are several different configurations of bracing members for the individual truss panels. The choice is influenced by the width of the panel, the magnitude of the wind and ice loads imposed, the location of equipment and the required stability. Continuity in transferring the applied loads through the structure without significant eccentricity is essential regardless of the configuration used. A self- supporting tower (Figure 1.8) requires a nearly square plot of land with sides equal to 8 to 20 percent of its height. The advantage of the self - supporting tower is the relatively small land area required (Windle, 2013).

Figure 1.8. Self-supporting tower.[8]

[8] https://www.indiamart.com/proddetail/self-support-communication-tower-5673741373.html

1.3.1.2. Guyed towers

Guyed towers (Figure1.9) are almost always of triangular cross section although there are a few unique conditions for microwave antenna supports where a square cross section is advantageous. The tower members are made of schifflerized angles or tube sections suitably protected by hot-dip galvanizing and the guy wires are made of galvanized steel cable. The principal structural elements are the legs, the web bracing in each face, and the guy support systems. Except for sections at the tower base and locations where the width changes, the legs are parallel. The width of the tower is usually constant throughout the height of the tower with the exception of sections supporting antennas requiring a specific width of support structure. The base section is often tapered to a single point to provide a pivot support to eliminate large bending and torsional moments. The amount of land required for a guyed tower depends on the distance between the tower base and the guy anchors. This distance is preferably between 70 and 80 percent of the height which would require a rectangular plot having sides equal to 125 and 145 percent of the height. Because of the great flexibility in guyed tower design, it is possible to reduce the anchor distance to as little as 35 percent of height thereby requiring a much smaller land area. However, the cost of the tower increases as the anchor distance decreases (Windle, 2013).

Figure 1.9. Guyed tower.[9]

[9] https://millmanland.com/company-news/what-is-a-cell-tower-and-how-does-a-cell-tower-work/

1.3.1.3. Monopole towers

Monopole towers, which apply horizontal forces and overturning moments to the foundation, are generally cylindrical in shape and made of concrete, which allows them to be built using the slipform method. They have many advantages:

- They require much less maintenance than steel towers;
- They provide easy access to antennas;
- The interior can house radio equipment.

Figure 1.10. Monopole tower.[10]

1.3.1.4. Stealth towers

Stealth towers are a particular brand of concealed towers. Another manufacture of concealed towers in Larson Camouflage. Concealed towers are deployed to satisfy zoning regulations, and can range in size to accommodate their surroundings (Figure1.11). They are more

[10] http://www.steeltowerchn.com/communication-tower/monopole-communication-tower-design-and-analysis-solution/

expensive than other types of towers because they require additional material to create a "concealed appearance," yet at the same time, they provide less capacity to tenants than other towers do.

Figure 1.11. Stealth tower.[11]

1.3.2. Telecommunication site

Nowadays, with the expansion of the various telecommunication networks, the construction of new sites is indispensable. Indeed, it is a place intended for the installation of various telecommunication infrastructures. A telecommunication site is generally composed of two major elements that is the relay and the tower (Delmas, 2006).

1.3.2.1. Relay

The relay acts as an intermediary between the mobile phone and the network subsystem, which groups together all the elements of mobile management and communication routing. In this section we will look at the main characteristics of a GSM relay, as well as its composition. In a rather simplified way, the relay is composed of the antennas and the base transceiver station.

a. Antennas

The antennas are the most visible components of the network. They can be seen everywhere, often on high masts, on the roofs of buildings, against walls, inside buildings. They are often

[11] https://www.cabinetmagazine.org/issues/4/dodge.php

invisible because they are camouflaged for aesthetic reasons near buildings. Antennas have different characteristics which are: frequency of usage, directivity and azimuth.

i. Frequency of usage

The most important characteristic of an antenna, also known as an aerial, is the frequency band it supports that is the frequencies the antenna can transmit and receive. On GSM sites, there are antennas that transmit only in 900 MHz, only in 1800 MHz or dual-band 900 and 1800 MHz antennas. Figure 1.12 and 1.13 shows some types of antennas that are available in the market (Delmas, 2006).

Figure 1.12.GSM antenna.[12]

Figure 1.13.FH antenna.[13]

[12] https://fr.dreamstime.com/photographie-stock-antenne-r%C3%A9seau-image2656502
[13] https://www.hertzien.fr/antennes.htm

ii. Directivity

The second important characteristic is the directivity in the horizontal plane that is the direction(s) in which the antenna will transmit.

iii. Azimuth

Each antenna is directed in a direction determined by simulations, so that it covers the defined area. The main direction of propagation of the antenna that is the direction in which the antenna transmits at its highest power, is directed in the established azimuth. The azimuth is an angle that is counted in degrees, positively clockwise, starting from north (0°). In this way, the 90° azimuth corresponds to the east, the 180° azimuth to the south and the 270° azimuth to the west.

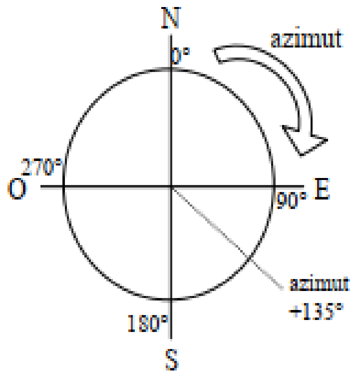


Figure 1.14. Representation of azimuths(Delmas, 2006).

1.3.2.2. Base transceiver station

The base transceiver station (Figure 1.15) is the first active electronic element of the GSM network, as seen by the mobile. It is the intermediate element that receives information, gives orders and executes them. It is housed in a shelter and is composed of a bay, an alimentation system, a control unit, communication cart and transmitter/receiver interface (Delmas, 2006).

Figure 1.15. Base transceiver station.[14]

1.3.3. Mounting procedures for tower supports

The towers are brought to the site in parts (bars or tubes). In the event of assembly in a location that is difficult to access, appropriate means are used to transport the parts. The tools needed for the assembly of the tower depend on the type of assembly used. In addition to small tools, there are masts, goats, winches, hoists, cables etc. The assembly process used depends on the characteristics of the towers (height, dimensions at the base, mass of the elements to be lifted), assembly facilities on the ground (assembly area), as well as the possibility of bringing lifting equipment to the site, and finally the means available to the company.

1.3.3.1. Mounting by rotating winch

This type of assembly is only feasible on flat and not very uneven ground. It is mainly used to erect towers of small dimensions (about 30 meters high) and limited weight (5 to 6 tons maximum), but also in rare cases to erect high metal towers. The fully assembled tower is placed in a horizontal position and fixed to the bases with hinges. The tower is then rotated around the bases using cables pulled by a winch. A mast can also be used to reduce the initial effort (which is then maximum). A guy line is used to hold the tower in place when it reaches

[14] https://www.wikiwand.com/en/Base_transceiver_station

the vertical position and to facilitate the attachment of the bolts. During the operation, the base of the tower is reinforced with wooden poles to prevent deformation.[15]

Figure 1.16.Rotating winch.[16]

1.3.3.2. Progressive mounting

Erection is carried out using a guyed lifting mast. The base of the mast is placed on the ground for the assembly of the lower sections and then on the erected sections for the lifting of the upper sections. The bar-by-bar assembly is no longer used. It has been replaced by the assembly of previously assembled units (sections, panels, beams, brackets, etc.). It is sometimes preferable to use a tall mast (over 50m) to avoid moving the mast too often.

Figure 1.17. Progressive mounting of a tower.[15]

[15] https://www.ingenieurs.com/documents/exposes/les-pylones-354.php
[16] https://www.drass.tech/projects/bell-winch-2/

1.3.3.3. Mixed Mounting

This is a combination of the two previous methods. At the beginning of the lift, two symmetrical panels (2 or 3 support sections) are rotated in opposite directions. These two panels are then joined together when they have reached their final position. The lifting process is then continued in a forward direction. [15]

1.3.3.4. Mounting by crane

This is a process that allows for rapid tower erection. However, their size makes them difficult to use in areas of low strength or difficult access.[15]

Figure 1.18. Mounting by crane.[17]

1.3.3.5. Mounting by helicopter

The helicopter can be used to install different panels of the tower in succession. An automatic system of temporary or permanent interlocking avoids any presence of personnel on the tower, thus increasing safety. The final assembly is carried out by the fitters as they go along, after the helicopter has left. A great deal of rigor in the preparation of the joints is necessary (positioning of tools, verification of joint dimensions, layout of the sections on the landing zone).[15]

[17] https://www.liebherr.com/fr/che/actualit%C3%A9s/news-communiqu%C3%A9s-de-presse/detail/la-grue-mobile-ltm-1300-6.2-de-kvn-monte-un-pyl%C3%B4ne-%C3%A9lectrique-news.html

Figure 1.19.Mounting by helicopter.[18]

1.3.4. Tower accidents

The rapid increase in population, electricity needs and telecommunication technology has led to a rise in the need of the construction of towers in the world. Due to this increase, the number of collapses has also increased leading to loss of life and infrastructure. Some cases of tower collapses will be presented below.

1.3.4.1. Tower collapse in Manitoba Canada

In Manitoba province in Canada, more than 600 electric towers were inspected after five of them collapsed in the province in 2017. According to Manitoba Hydro, these incidents were the result of human error. Two of the five towers collapsed because the guys wires were not securely anchored. Two others collapsed because the components were not installed in the correct order. The fifth incident occurred when the tower broke away from a foundation pin.

[18] https://www.gamaniak.com/video-13842-pose-precise-pylone-electrique-helicoptere.html

Figure 1.20. Electric tower collapse in Manitoba 2017.[19]

1.3.4.2. Tower collapse in Abidjan Ivory Coast

In June 2016, a tower of the company Telecom-HIS in Ivory Coast collapsed. According to a press release from the High Authority for Audiovisual Communication (HACA), the audiovisual regulatory body in Ivory Coast, the tower whose fall is at the root of this situation is located in the commune of Abobo, in the northern suburb of Abidjan. The 130-metre tower, located in Abobo-sagbê, collapsed following a violent wind that blew over the city of Abidjan.

Figure 1.21. Cell tower collapse in the commune of Abobo in Abidjan 2016.[20]

[19] https://ici.radio-canada.ca/nouvelle/1040611/effondrement-tombe-tour-hydro-manitoba-electricite
[20] https://ici.radio-canada.ca/nouvelle/1040611/effondrement-tombe-tour-hydro-manitoba-electricite

1.3.4.3. Tower collapse in Douala Cameroon

At the place known as the Crtv antenna in Logbessou on Friday 26 September 2014, the collapse of a cell tower happened. According to information from sources close to the investigation, four technicians were trying to repair some of the rusty elements of this tower when the accident happened, they died immediately. Some of the leg members were defective, the horizontal primary bracings of some panels were attacked by rust and they had to replace these elements completely. Speaking of the causes of the fall of the Crtv tower, no one knows exactly what happened but according to the technicians of the Cartel company who won the repair of the tower, some parts of this installation were rusty, this would have contributed to the fall of the tower and the death of the four technicians.

Figure 1.22.Crtv tower collapse in Douala Cameroon 2014.[21]

1.4. Wind loading on structures

The characteristics of wind pressures on a structure are a function of the characteristics of the approaching wind, the geometry of the structure under consideration, and the geometry and proximity of the structures upwind. The pressures are not steady, but highly fluctuating, partly as a result of the gustiness of the wind, but also because of local vortex shedding at the edges of the structures themselves. The fluctuating pressures can result in fatigue damage to

[21] http://hervevillard.over-blog.com/2014/09/douala-le-pylone-der-la-crtv-tue-quatre-ingenieurs.html

structures, and in dynamic excitation, if the structure happens to be dynamically wind sensitive. The pressures are also not uniformly distributed over the surface of the structure, but vary with position. Below we will see the different types of wind drift design and the design criteria in wind loading (Mendis et al. 2007).

1.4.1. Types of wind drift design

Typically for wind sensitive structures three basic wind effects need to be considered namely environmental wind studies, wind loads for façade and wind loads for structure (Mendis et al. 2007).

1.4.1.1. Environmental wind studies

It investigates the wind effects on the surrounding environment caused by erection of the structure (e.g., tall building). This study is particularly important to assess the impact of wind on pedestrians, motor vehicles and architectural features such as fountains, etc., which utilize public domain within the vicinity of the proposed structure (Mendis et al. 2007).

1.4.1.2. Wind loads for façade

This is to assess design wind pressures throughout the surface area of the structure for designing the cladding system. Due to the significant cost of typical façade systems in proportion to the overall cost of very tall buildings, engineers cannot afford the luxury of complexity of building shapes and dynamic characteristics of the wind and building structures, even the most advanced wind codes generally cannot accurately assess design loads. Wind tunnel testing to assess design loads for cladding, is now normal industry practice, with the aim of minimizing initial capital costs, and more significantly avoiding expensive maintenance costs associated with malfunctions due to leakage and/or structural failure (Mendis et al. 2007).

1.4.1.3. Wind loads for structure

This is to determine the design wind load for designing the lateral load resisting structural system of a structure to satisfy various design criteria.

1.4.2. Design criteria

In terms of designing a structure for lateral wind loads the following basic design criteria need to be satisfied:

- Stability against overturning, uplift and/or sliding of the structure as a whole;

- Strength of the structural components of the building is required to be sufficient to withstand imposed loading without failure during the life of the structure;
- Serviceability for example for buildings, where inter-storey and overall deflections are expected to remain within acceptable limits. Control of deflection and drift is imperative for tall buildings with the view to limiting damage and cracking of non-structural members such as the facade, internal partitions and ceilings (Mendis et al. 2007).

Conclusion

In this chapter, we started by introducing the concept of steel material. Steel is the main element constituting telecommunication towers and for this reason a clear understanding of this material is necessary. It's manufacturing processes, properties (physical, chemical and mechanical), typology, defects and treatments methods have been discussed above. The concept of reticular truss structure has been introduced where the hypothesis on which its analysis is done have been given, the different methods used to determine its solicitations have been introduced followed by its common field of application. We also discussed about telecommunication towers starting by its typology where we have seen that four different types exist. The elements present in a communication site were also discussed. The different methods used for the mounting process of towers were sighted and some incidents that occurred on these towers have been reported. Finally, we have discussed about wind loads on structures where the different types of wind drift design have been seen and the essentials criteria necessary when considering the action of wind were stated.

CHAPTER 2: METHODOLOGY

Introduction

The methodology is the part of the study that establishes the research procedure after the definition of the problem, so as to achieve the set objectives. In other words, it will be question of describing the different constitutive elements of our research. In this work, the first step consists in a site recognition through documentary research followed by the data collection that will enable the analysis and a modelling of the telecommunication tower. Thereafter, this chapter will focus on the description of the design procedures and the governing equations used by analytical and numerical procedures which are intended to be used for the performance assessment of the steel structure and the foundations. All this information will help us verify the tower is structurally stable that is determining if it will not collapse and fall.

2.1. Site recognition

Recognition of the site was done through research from available documents in order to know on one hand the general physical characteristics (geographical location, climate, hydrology and geology) as well as the economic activities of the region.

2.2. Data collection

The main type of data required for the purpose of this research is the architectural and structural data of the building.

2.2.1. Architectural data

These data collected gives information on the geometry of the building, the surface area and the specific use of the building.

2.2.2. Structural data

These data collected gives information on the section's properties of structural elements, the characteristic of materials used as well as the load applied on the structure.

2.3. Norms

The regulations and design standards used in the development of this work are:

- Eurocode 3 for the design of steel elements;
- NV 65 modified 99, for the determination of wind loads;

- Eurocode 2, for the design of the reinforced concrete foundation.

The present work therefore involves the intensive use of parts 1 and 3 of Eurocode 3. The modified NV 65 regulations, on the other hand, are intended to establish the values of climatic overloads and to give the methods for evaluating the corresponding forces on the whole of a construction or on its different parts.

2.4. Preliminary design of the structure

It is the first step in the design of a steel structure. Indeed, this step is necessary to start the technical study of a project. Generally, it is almost impossible to give the exact dimensions of the elements of a structure without having all the loads that are applied to it. However, the loads are a function of the geometric characteristics of each element. Therefore, the preliminary design consists in giving dimensions to each element of the structure according to constructive provisions and based on existing structures.

2.5. Evaluation of the loads on the structure

Different types of actions act on the structure. Regarding our study, we will focus on permanent, variable and wind loads.

2.5.1. Permanent loads

These are also known as static or dead loads. These are actions that act on the whole nominal life of the structure with a negligible variation of their intensity in time. We have the self-weight of the structural elements and the self-weight of the non-structural elements such as antennas and rest levels on the structure.

2.5.2. Variable loads

These are actions on structures with instantaneous values which can significantly vary in time. We have imposed loads and wind loads.

2.5.2.1. Imposed loads

These are loads arising from building occupancy and maintenance (roof elements). They include the loads induced by the normal use by persons, the furniture and moveable objects, vehicles etc.

2.5.2.2. Wind Loads

The wind is an action that can be applied from any direction for the calculation of constructions it is assumed that the wind has a horizontal average direction. The action of the wind on a structure and on each of its elements depends characteristics such as wind speed, dimensions and type of the structure, local configuration of the terrain (topography, region), permeability of the structure. Two actions are to be considered according to the surface exposed to the wind that is the overall action on the tower and the overall action on the antennas

a. Overall action on the tower

This action allows the calculation of the main elements ensuring the stability of the structure. The overall action of the wind blowing in a given direction on a construction is the geometric resultant of all the actions on the different walls. For towers, it is the horizontal component T_d which is the drag producing an overturning effect.

$$T_d = C_t * q * S_p \qquad (2.1)$$

Where:

C_t is the global drag coefficient;

q is the average value of the dynamic pressure;

S_p is the surface exposed to wind whatever the incidence.

i. Global drag coefficient C_t

Wind acts in two directions. One normal to the face of the tower and the other acting on the diagonal of the tower. The wind on the diagonal has two components in two different:

- Wind normal to a face where the coefficient C_t is obtained in Table 2.1;
- Wind along a diagonal, the previously determined coefficient is multiplied by a coefficient χ depending on the nature of the structure and given by Table 2.2.

Table 2.1. Global drag coefficient C_t.

	Truss bar slightly rounded	Truss Tube section
C_t	3.20 - 2φ	2.24 -1.1φ

Table 2.2. χ coefficient with respect to material type.

Material type	χ	
	Normal bar	Twinned bar
Steel	1+0.6φ	1.2
Concrete	1.2	1.2
Wood	1.2	1.3

Where, $\varphi = \frac{S_p}{S}$ and S is the surface area of the panel.

ii. Average dynamic pressure

The average dynamic pressure q is given by,

$$q = q_{10} * K_h * K_s * K_m * \delta * \beta \tag{2.2}$$

Where:

q_{10} the basic dynamic pressure at 10 m height;

K_s is the site effect;

K_m is the mask effect;

δ is the dimension effect;

β is the dynamic amplifying factor;

K_h is the height factor.

Each of these factors will be determined using the NV 65 norm.

1) Basic dynamic pressure q_{10}

By convention, the normal and extreme base dynamic pressures are those at a height of 10 m above the ground, for a normal site, without mask effect on an element whose largest dimension is equal to 0.50 m. Table 2.3 shows the different base dynamic pressure, wind speeds normal and extreme according to the zone. The ratio extreme dynamic pressure to normal dynamic pressure is 1.75.

Table 2.3. Basic dynamic pressure and wind speed with respect to zone.

	Normal base dynamic pressure(KN/m2)	Extreme base dynamic pressure(KN/m2)	Normal wind speed(m/s)	Extreme wind speed(m/s)
Zone 1	0.5	0.875	28.6	37.8
Zone 2	0.6	1.05	31.3	41.4
Zone 3	0.75	1.3125	35	46.3
Zone 4	0.9	1.575	38.2	50.7
Zone 5	1.2	2.1	44.2	58.5

Figure 2.1 shows the different zones and corresponding wind speed in the African continent.

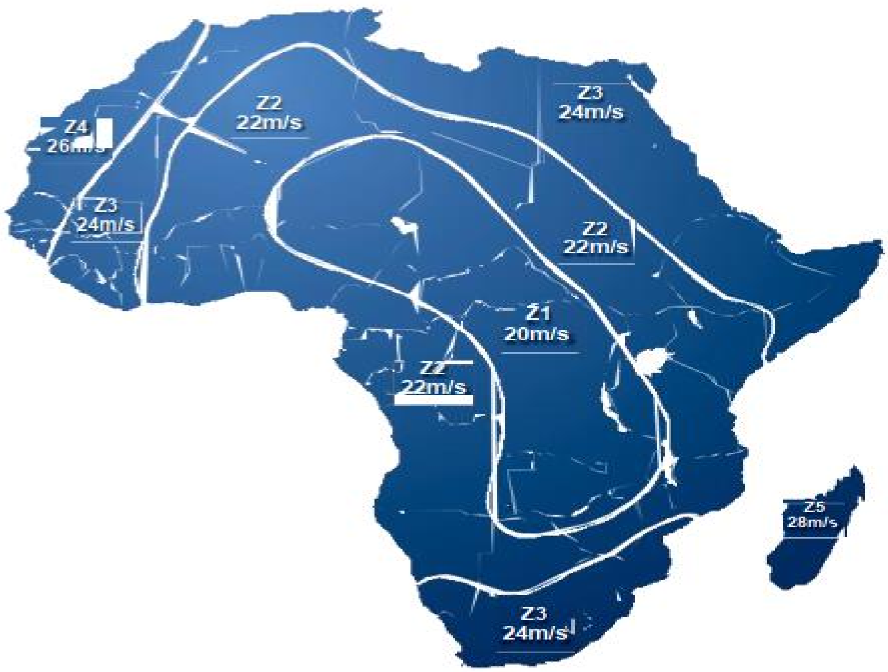


Figure 2.1. Africa wind map.

2) **Height factor K_h**

The height factor given by equation 2.3.

$$K_h = \frac{q_H}{q_{10}} = 2.5\,\frac{H + 18}{H + 60} \tag{2.3}$$

with q_H being the pressure at a specific height H.

3) **Site effect K_s**

The site effect coefficient is obtained in Table 2.4.

Table 2.4. Site effect coefficient K_s.

	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5
Protected site	0.8	0.8	0.8	0.8	
Normal site	1	1	1	1	1
Exposed site	1.35	1.3	1.25	1.2	1.2

The Rules consider three types of sites:

- Protected site. Example. Bottom of a basin bordered by hills all around and thus protected for all wind directions;
- Normal site. Example. A large plain or plateau that may have small differences in level, with a slope of less than 10 % (undulations, valleys);
- Exposed site. Examples. In the vicinity of the sea: the coastline in general (over a depth of about 6 km); the top of the cliffs; the islands or narrow peninsulas. Inland: narrow valleys where the wind rushes in; isolated or high mountains (for example Mount Saint-Vincent) and some passes.

4) **Mask effect K_m**

A masking effect occurs when a construction is partially or totally masked by other constructions with a high probability of duration. The masking effect can result:

- Either by an aggravation of the wind actions, when the construction located behind the mask is in a turbulent wake zone. In this case, it is not possible to formulate rules; only wind tunnel tests can give precise information;
- Or by a reduction of the wind actions in the other cases. The basic dynamic pressures can then be reduced by 25% subject to the subject to the rule;

In general, we do not take into account the masking effects due to other constructions partially or completely masking the studied construction. We then use K_m= 1.

5) **Dimension effect δ**

The regulation takes into account an effect of dimension, which is introduced by a coefficient δ taking into account the largest dimension offered to the wind of the element studied taking into account the largest dimension offered to the wind of the element studied. This coefficient allows to reduce the pressures, it takes into account the statistical distribution of pressures on a surface. The maximum pressure is located at the center, where the streamlines are deviated to 90 °, and it tends to decrease on the edges. It is therefore unlikely that the pressure will be the same everywhere on a large surface pressure everywhere, the wind being gusty. This justifies the decrease of the pressures with the increase of the dimensions. The dimension effect is obtained in figure 2.2**.**

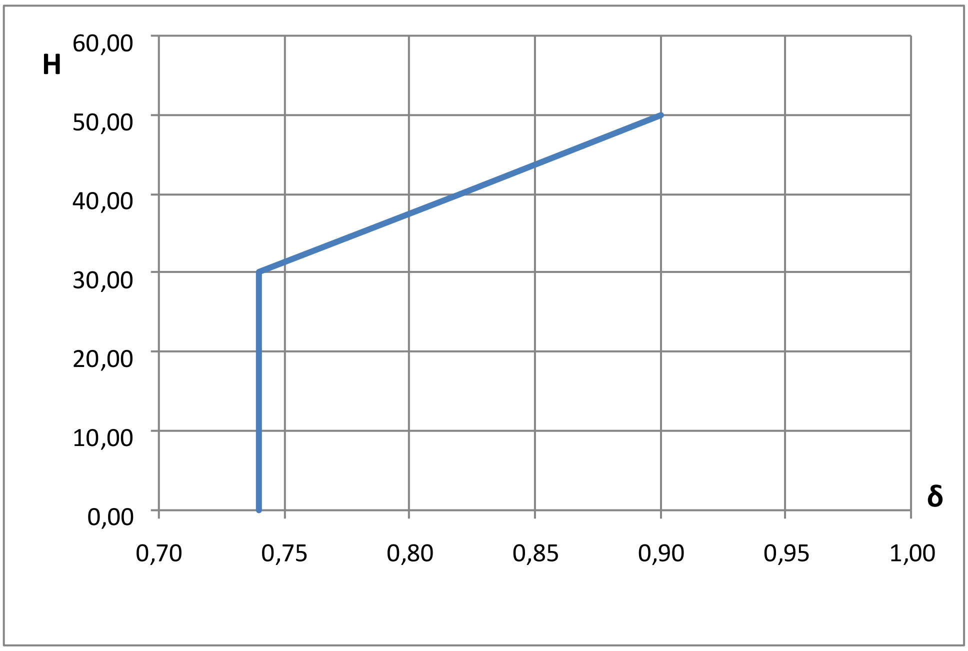


Figure 2.2.Dimension effect δ with respect to height.

6) **Dynamic amplification factor β**

In addition to the static effects defined above, there are dynamic effects that depend on the depend on the mechanical and aerodynamic characteristics of the construction. To take into account the effect of the actions parallel to the wind direction, the normal dynamic pressures the normal dynamic pressures used for the calculation of the overall action, are multiplied at each level by a coefficient of increase at least equal to the unit. This coefficient depends on whether the dynamic pressure is normal or extreme. Its value is obtained through the equation;

$$\beta_{normal} = \theta\ (1+ \xi\ \tau) \qquad (2.4)$$

ξ response coefficient, is given as a function of the period T of the fundamental mode of oscillation and for structures of various degrees of damping. It is obtained in figure 2.3.

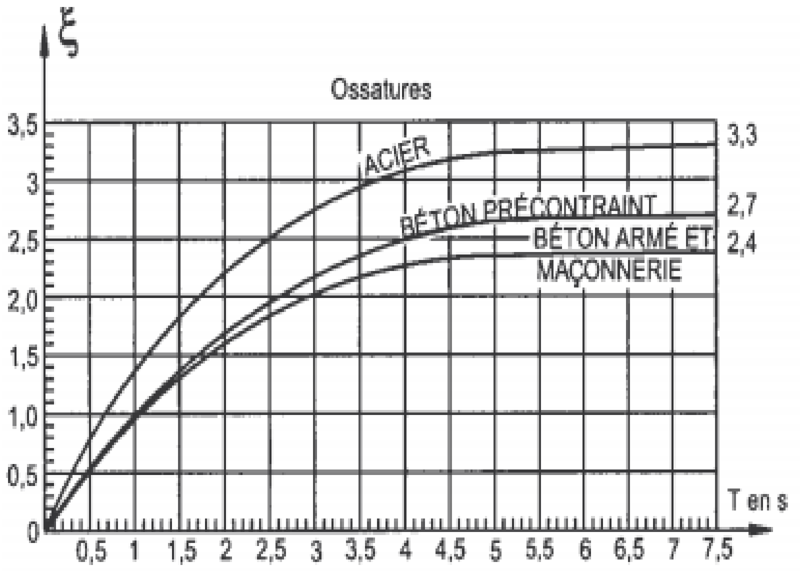


Figure 2.3. Response coefficient with respect to the fundamental period of oscillation of building.

The fundamental period of a building is an `inherent property who depend on the properties of the structure. Some empirical formulas permit to estimate this period. It's the case of the one defined by the EC8 1-2.

$$T = C_t H^{\frac{3}{4}} \tag{2.5}$$

Where:

C_t is a coefficient that depends on the moment resisting type of the structure;
H is the height of the structure above the foundation.

θ is the global coefficient depending on the type of construction. For our construction case:

θ = 0.7, H < 30m;

θ = 0.70 + 0.01 (H-30), 30m < H < 60m;

θ = 1, H > 60m.

τ pulsation coefficient, is determined at each level considered as a function of its height H above the ground by the functional scale in figure 2.4.

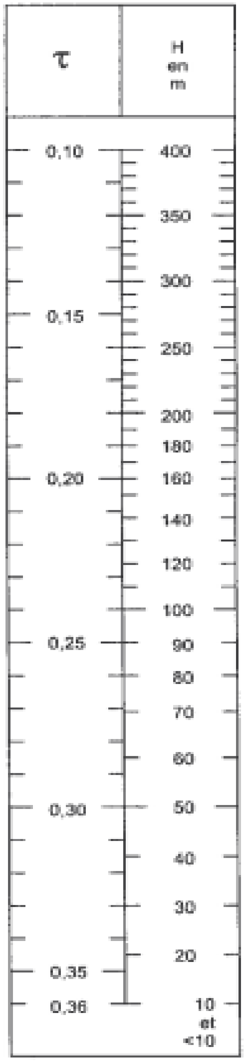


Figure 2.4. Pulsation factor τ with respect to height above ground level.

In case of extreme overload, the dynamic amplification factor is given by equation 2.6.

$$\beta_{extreme} = \beta_{normal}(\frac{\theta}{2} + 0.5) \tag{2.6}$$

b. Overall action on the antennas

Only FH, WCDMA and GSM antennas will be considered in the dimensioning of the towers. For each antenna the effective area S_{eq} has to be calculated by the following steps:

- Calculate its effective area S_{eff} according to the shape of the antenna);
- Locate its position P with respect to the tower base;
- Read the corresponding drag coefficient C_{ti} as a function of its azimuth;

- Determine the equivalent surface S_{eq} given by equation 2.7.

$$S_{eq} = C_{ti} S_{eff} \frac{P}{H} \tag{2.7}$$

By applying the same operation for all antennas, the sum of these areas is called the the tower's head area. This area corresponds to the area of a fictitious antenna placed on the tower's head. It is necessary to multiply the tower's head area by the dynamic pressure q_H

to get the force that will be applied horizontally to the masthead. If we note F_{eq} the force created by the antenna is determined by equation 2.8.

$$F_{eq} = q \sum S_{eq} \tag{2.8}$$

2.6. Load combinations

A load combination defines a set of values used for the verification of the structural reliability for a limit state under the simultaneous influence of different loads. A combination of actions defines a set of values used for the verification of the structural reliability for a limit state under the simultaneous influence of different actions. In the case of a building, they are defined by:

The fundamental combination, used for the Ultimate Limit State (ULS) associated with collapse or other similar forms of structural failure is given by equation 2.9.

$$\sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \gamma_p P + \gamma_{Q,1} Q_{k,1} + \sum_{i \geq 1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i} \tag{2.9}$$

For SLS the load combinations include Characteristic (rare) combination, frequent and quasi permanent combinations; these combinations are represented respectively equations 2.10, 2.11 and 2.12.

$$\sum_{j \geq 1} G_{k,j} + P + Q_{k,1} + \sum_{i \geq 1} \Psi_{0,i} Q_{k,i} \tag{2.10}$$

$$\sum_{j \geq 1} G_{k,j} + P + \Psi_{1,1} Q_{k,1} + \sum_{i \geq 1} \Psi_{2,i} Q_{k,i} \tag{2.11}$$

$$\sum_{j \geq 1} G_{k,j} + P + \Psi_{2,1} Q_{k,1} + \sum_{i \geq 1} \Psi_{2,i} Q_{k,i} \tag{2.12}$$

Where:

Gk,j is the characteristic value of the permanent action j;

$Qk,1$ is the characteristic value of the leading variable action 1;

Qk,i is the characteristic value of the accompanying variable action i;

$\Psi_{0,i}$ is the combination coefficient;

Γ is the safety factor for permanent and variable loads.

2.7. Analysis and design

The design of the elements of the structure consists in calculating them taking into account the forces and loads applied to the structure. This calculation will provide the profiles that can ensure both the strength and stability of the structure. Global analysis of the framework and determination of the stresses in the bars, verification of the sections and elements of the framework at the SLS and ULS, design and resistance of connections, foundation design will be discussed in this chapter accompanied with a description of the software used for the analysis.

2.7.1. Software used for the structural analysis and design of the structure

The structural model of the case study shall be modelled with SAP2000 v22. Modelling will consist of creating the appropriate material, section properties, loads and combinations. The steel elements shall be drawn according to plan and the supports conditions assigned to be fixed and pinned where concerned. The structure shall be loaded with respect to specific load patterns discussed in section 2.5.1 and 2.5.2. The load combinations will be defined prior to the analyses to satisfy the ULS and SLS conditions discussed in section 2.6.

2.7.2. Design of the steel structure

The structure will be designed according to the corresponding limit states in such a way to sustain all actions acting upon it during its intended life. This implies it will be designed having adequate structural stability (ultimate limit states) and remain fit for the use it is required (serviceability). Before any element is designed, it needs to be classified according to its capacity to develop plastic hinges and rotation deformations. We shall use the classification proposed by EC3 1-1.

2.7.2.1. Classification of the sections

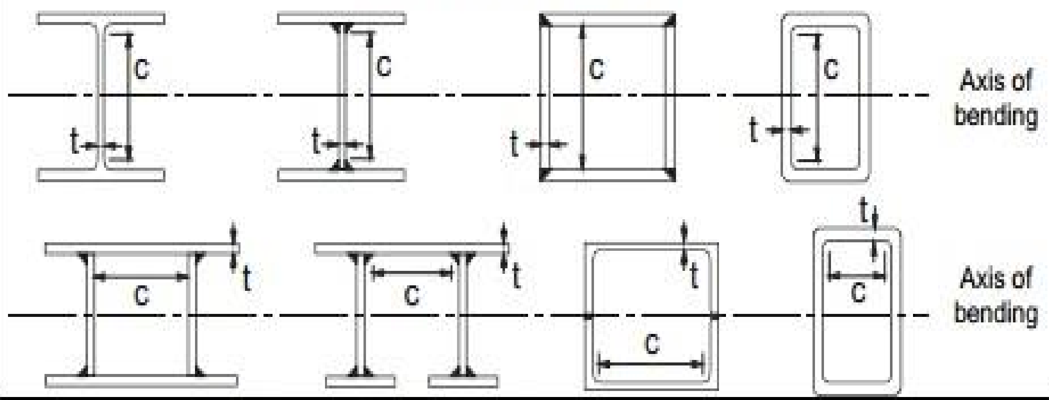
The sections of the members to be design are going to be classified as class 1, 2, 3, or 4. The norm defines these classes as follow:

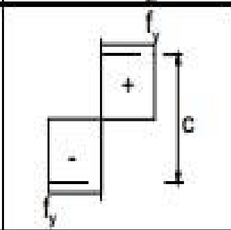
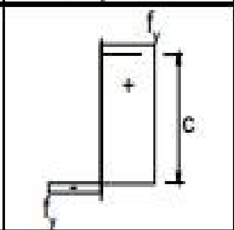
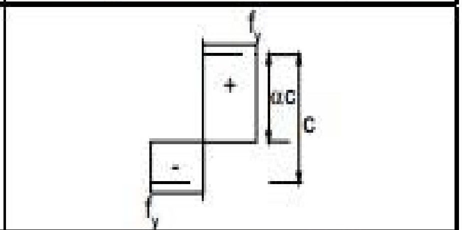
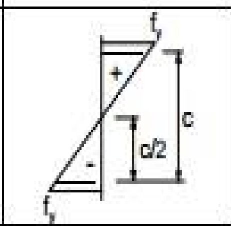
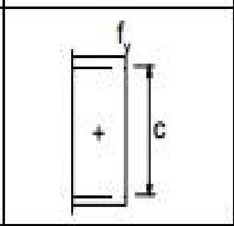
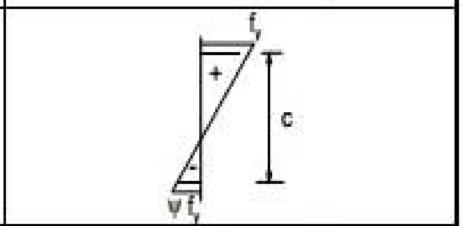
- Class 1 cross-sections are those which can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of the resistance;
- Class 2 cross-sections are those which can develop their plastic moment resistance, but have limited rotation capacity because of local buckling;
- Class 3 cross-sections are those in which the stress in the extreme compression fiber of the steel member assuming an elastic distribution of stresses can reach the yield strength, but local buckling is liable to prevent development of the plastic moment resistance;
- Class 4 cross-sections are those in which local buckling will occur before the attainment of yield stress in one or more parts of the cross-section.

The sections are going to be classified according to Table 2.5, Table 2.6 and Table 2.7. (source EC3 1-1, 2005).

Table 2.5. Maximum width-to-thickness ratios for internal compression parts.

Internal compression parts



Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \le 72\varepsilon$	$c/t \le 33\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \dfrac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \dfrac{36\varepsilon}{\alpha}$
2	$c/t \le 83\varepsilon$	$c/t \le 38\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \dfrac{456\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \dfrac{41{,}5\varepsilon}{\alpha}$
Stress distribution in parts (compression positive)			
3	$c/t \le 124\varepsilon$	$c/t \le 42\varepsilon$	when $\psi > -1$: $c/t \le \dfrac{42\varepsilon}{0{,}67 + 0{,}33\psi}$ when $\psi \le -1$*): $c/t \le 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

*) $\psi \le -1$ applies where either the compression stress $\sigma \le f_y$ or the tensile strain $\varepsilon_y > f_y/E$

Table 2.6. Maximum width-to-thickness ratios for compression parts (outstand flanges).

Outstand flanges

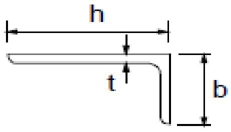
Class	Part subject to compression	Part subject to bending and compression: Tip in compression	Part subject to bending and compression: Tip in tension
Stress distribution in parts (compression positive)			
1	$c/t \le 9\varepsilon$	$c/t \le \frac{9\varepsilon}{\alpha}$	$c/t \le \frac{9\varepsilon}{\alpha\sqrt{\alpha}}$
2	$c/t \le 10\varepsilon$	$c/t \le \frac{10\varepsilon}{\alpha}$	$c/t \le \frac{10\varepsilon}{\alpha\sqrt{\alpha}}$
Stress distribution in parts (compression positive)			
3	$c/t \le 14\varepsilon$	$c/t \le 21\varepsilon\sqrt{k_\sigma}$ For k_σ see EN 1993-1-5	

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

Table 2.7. Maximum width to thickness ration for members in bending and/or compression.

Angles

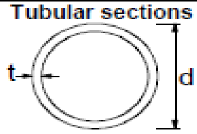
Refer also to "Outstand flanges" (see sheet 2 of 3)



Does not apply to angles in continuous contact with other components

Class	Section in compression
Stress distribution across section (compression positive)	
3	$h/t \le 15\varepsilon : \frac{b+h}{2t} \le 11{,}5\varepsilon$

Tubular sections



Class	Section in bending and/or compression
1	$d/t \le 50\varepsilon^2$
2	$d/t \le 70\varepsilon^2$
3	$d/t \le 90\varepsilon^2$ NOTE For $d/t > 90\varepsilon^2$ see EN 1993-1-6.

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71
	ε^2	1,00	0,85	0,66	0,56	0,51

2.7.2.2. Ultimate limit states design checks for steel members (ULS)

Ultimate limit states are those that relate to the failure of a structural member or a whole structure. Design verifications that relate to the safety of the people in and around the structure are ultimate limit state verifications. The design verifications will be done with respect to EC3 1-1, 2005.

a. Design of tension or compression members

The design for a tension or compression member must be such that the design actions are lower than the resisting forces and moments.

i. Tension member

The area of the section is given by equation 2.13. For a tension member, we will verify equation 2.14.

$$A_{nec}=\frac{\mathrm{N_{Ed}}\,\gamma_{\mathrm{M1}}}{\chi_{min} f_{yk}} \tag{2.13}$$

$$\frac{N_{t,Ed}}{N_{t,Rd}} \leq 1 \tag{2.14}$$

Where:

$\mathrm{N_{Ed}}$ is the design tension load;

$\mathrm{N_{t,Rd}}$ is the resisting tensile force of the element and is the minimum between $\mathrm{N_{pl,Rd}}$ and $\mathrm{N_{u,Rd}}$ given in equations (2.17) and (2.18).

$$\mathrm{N_{Rd}} = \frac{\mathrm{Af_y}}{\gamma_{\mathrm{M0}}} \tag{2.15}$$

$$\mathrm{N_{u,Rd}} = \frac{0.9\mathrm{A_{net}f_u}}{\gamma_{\mathrm{M2}}} \tag{2.16}$$

ii. Compression member

For a compression member, we will verify equation 2.17. The area of the section is obtained from equation 2.13.

$$\frac{\mathrm{N_{c,Ed}}}{\mathrm{N_{c,Rd}}} \leq 1 \tag{2.17}$$

Where $\mathrm{N_{c,Rd}}$ should be determined by equation (2.18).

$$\frac{N_{c,Ed}}{N_{c,Rd}} \leq 1 \tag{2.17}$$

For class 1, 2 and 3 sections,

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}} \tag{2.18}$$

The steel element shall also be checked for buckling. To do so, the design compressive force should be lower than the buckling resistance force (equation 2.19).

$$\frac{N_{c,Ed}}{N_{b,Rd}} \leq 1 \tag{2.19}$$

$N_{b,Rd}$ is the design buckling resistance of the compression member and is given by equation (2.20) and (2.21).

For class 1, 2 and 3 sections,

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}} \tag{2.20}$$

For class 4 cross section,

$$N_{b,Rd} = \frac{\chi A_{eff} f_y}{\gamma_{M1}} \tag{2.21}$$

χ is the buckling factor which reduces the resisting axial force of the whole element.

To determine χ, we first need to select the appropriate buckling curve (Figure 2.5) which is given according to Table 2.8 with respect to the section's characteristics and steel type.

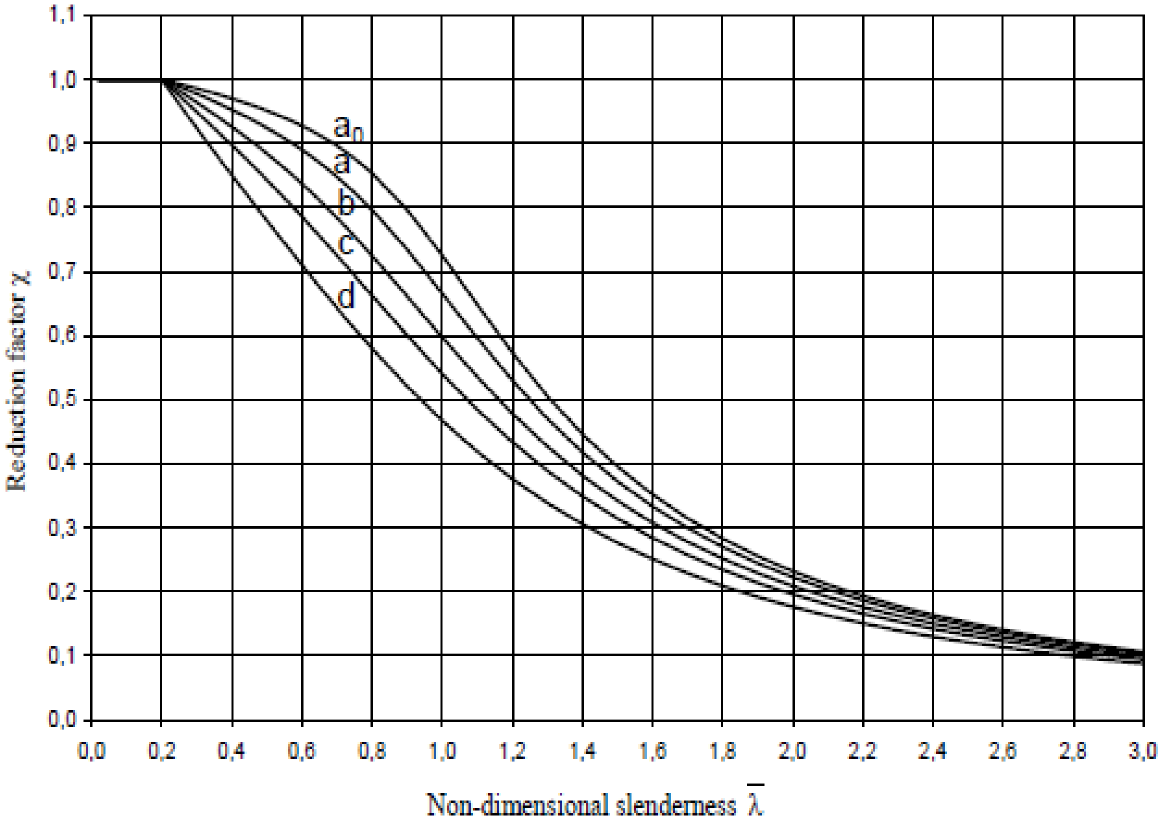
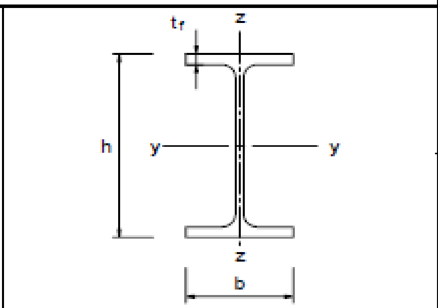
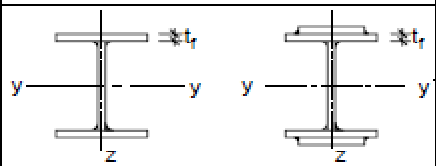
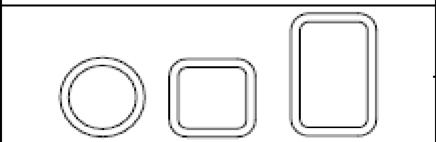
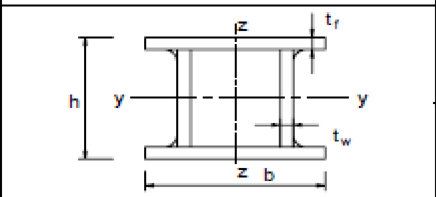
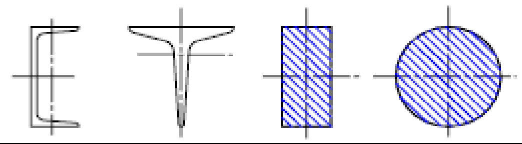
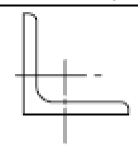


Figure 2.5. Buckling curves (source: EC3 1-1 , 2005).

Table 2.8. Selection of buckling curve for a cross section (source: EC3 1-1, 2005).

Cross section		Limits		Buckling about axis	Buckling curve S 235 S 275 S 355 S 420	Buckling curve S 460
Rolled sections		h/b > 1,2	$t_f \leq 40$ mm	y – y z – z	a b	a_0 a_0
		h/b > 1,2	40 mm < $t_f \leq 100$	y – y z – z	b c	a a
		h/b ≤ 1,2	$t_f \leq 100$ mm	y – y z – z	b c	a a
		h/b ≤ 1,2	$t_f > 100$ mm	y – y z – z	d d	c c
Welded I-sections		$t_f \leq 40$ mm		y – y z – z	b c	b c
		$t_f > 40$ mm		y – y z – z	c d	c d
Hollow sections		hot finished		any	a	a_0
		cold formed		any	c	c
Welded box sections		generally (except as below)		any	b	b
		thick welds: $a > 0{,}5t_f$ $b/t_f < 30$ $h/t_w < 30$		any	c	c
U-, T- and solid sections				any	c	c
L-sections				any	b	b

The non-dimensional slenderness, $\bar{\lambda}$ is computed using equation (2.24) and equation (2.25).

For class 1, 2 and 3 cross sections

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \frac{l_{cr}}{i}\left(\frac{1}{\lambda_1}\right) \quad (2.22)$$

For class 4 cross sections

$$\bar{\lambda} = \sqrt{\frac{A_{eff}f_{yk}}{N_{cr}}} = \frac{l_{cr}}{i}\frac{\sqrt{\frac{A_{eff}}{A}}}{\lambda_1} \quad (2.23)$$

Where:

l_{cr} is the buckling length in the buckling plane considered;

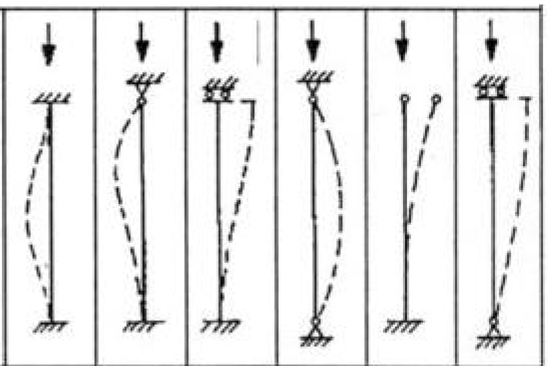
i is the radius of gyration about the relevant axis;

$\lambda_1 = 93.9\varepsilon$.

N_{cr} is the elastic critical force for the relevant buckling mode based on the gross cross-sectional properties and is given by equation (2.26).

$$N_{cr} = \frac{\pi^2 EI}{{l_0}^2} \tag{2.24}$$

Table 2.9. Effective length factors

Buckled shape of column						
Theoretical value	0,5	0,7	1,0	1,0	2,0	2,0

b. Design of the connections

In this structure, bolted connections were used the most.

i. Shear resistance for one bolt

The design resistance for a bolt in shear is given by equation (2.25).

$$F_{v,Rd} = \frac{\alpha_v \, f_{ub} \, A_b}{\gamma_{Mb}} \tag{2.25}$$

Where:

$$\alpha_v = \begin{cases} 0{,}6 \text{ for classes } 4.6, 5.6 \text{ and } 8.8 \\ 0{,}5 \text{ for classes } 4.8, 5.8 \text{ and } 10.9 \end{cases};$$

f_{ub} is the ultimate tensile stresses of the bolt;

A_b is the gross area of the bolt given by $\frac{\pi d^2}{4}$;

γ_{Mb} is the partial safety factor for bolts = 1,25.

ii. Traction resistance for one bolt

The design resistance for a bolt in tension is given by equation (2.26).

$$F_{t,Rd} = \frac{0{,}9\, f_{ub}\, A_b}{\gamma_{Mb}} \tag{2.26}$$

With A_s being the resisting area of the bolt

iii. Bearing resistance for the plate

For the plate involved in the connection, the bearing resistance is given by Equation (2.27).

$$F_{b,Rd} = \frac{K_1\, \alpha_b\, f_u\, d_0\, t}{\gamma_{M2}} \tag{2.27}$$

$$\alpha_b = \min\left\{\left(\frac{p_1}{3d_0} - 0{,}25\right); \frac{f_{ub}}{f_u}; 1\right\} \tag{2.28}$$

$$K_1 = \min\left\{\left(\frac{2{,}8e_2}{d_0} - 1{,}7\right); 2{,}5\right\} \tag{2.29}$$

f_u is the ultimate tensile strength for the plate;

d_0 is the hole diameter;

T is the thickness of the plate.

e1, e2, p1, p2 are the constructive dispositions on the plate and are represented in Figure 2.6.

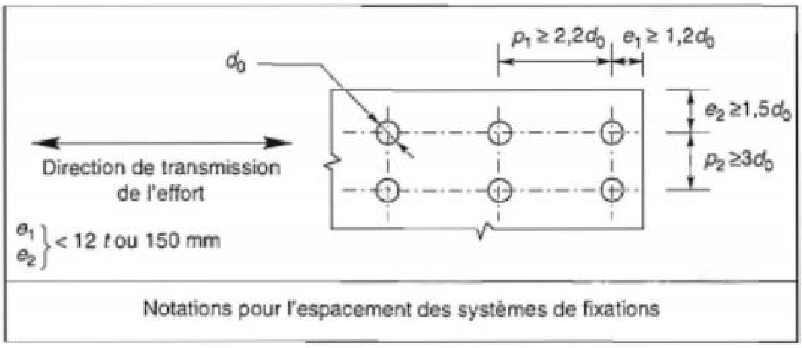


Figure 2.6. Spacing of the holes on the plate (Morel, 2005).

2.7.2.3. Displacement verification under SLS condition

The displacement of the tower should be less than the value in equation 2.30.

$$f \leq \frac{H}{200} \tag{2.30}$$

H is the height of the tower.

2.7.3. Foundation design of the steel structure

In this section, we will design two different types of foundation systems. The first type of foundation we will study is a flexible plinth and the second type of foundation will be a raft footing. With each kind of footing, we will verify the structural stability of the structure in order to know if the foundation type used is appropriate. A column-base plate will be designed to establish the connection between the steel structure and the concrete columns that will be linked to both the flexible plinth and the raft.

2.7.3.1. Column- base plate

The assembly of a column base-plate on a concrete block is carried out by means of a plate supported on the block by anchor bolts. The plate is welded to the base of the column by means of a weld bead applied around the perimeter of the section of the profile forming the member. These connections involve the transmission of a vertical compression or uplift force and a horizontal force. The area of the plate is given by equation 2.31.

$$A = B.C \geq \frac{N_{ed}}{f_{cd}} \tag{2.31}$$

Where, B and C are the dimensions of the plate.

The thickness of the plate should not be less than the value in equation 2.32,

$$t_p \geq \sqrt{\frac{2N_{ed}m^2}{BCf_{yk}}} \tag{2.32}$$

With m, the projection of the plate beyond the steel column.

Determine the anchorage length of the anchors of the plate by equation the adhesion stress to the uplift force of the anchor bolt that is;

$$\pi \emptyset L_s \tau_{su} = N_a \tag{2.33}$$

Where:

N_a is the lifting force on the bolt;

L_s is the anchor length;

$\tau_{su} = 0.6\Psi_s{}^2 f_{tj}$.

With:

Ψ_s= 1 for HA steel;

$f_{tj} = 0.6 + 0.06f_{ck}$.

Check for the shear resistance of the anchors, the traction resistance and bearing resistance of the plate using equation (2.25), (2.26) and (2.27) respectively.

2.7.3.2. Column design

The column is designed using the MN interaction method which is a curve of plot points in which each point has two ordinates. The envelope of the bending moment and the axial force solicitations obtained, the design is done through the M-N interaction diagram. For each level, we have to ensure that the maximum M-N solicitation belong to the M-N interaction diagram of the section considered. The interaction diagram is a diagram that shows all the limit situation that can determine the failure of the section. The points which are lying into the diagram represent the limit configuration: beyond them, failure occurs. This diagram is computed by determining some significant points.

a. First point

The section is completely subjected to tension; hence, the concrete is not reacting. We impose $\varepsilon_s = \varepsilon_{su}$, $\varepsilon_s{}' = \varepsilon_{syd}$ then the stress inside the element correspond to the design yielding strength of the steel reinforcement and the limit axial force and bending moment are obtained from the equations 2.34 and 2.35 respectively.

$$N_{Rd} = f_{yd}As + f_{yd}A's \tag{2.34}$$

$$M_{RD} = f_{yd}A_s\left(\frac{h}{2} - d'\right) - f_{yd}A'_s\left(\frac{h}{2} - d'\right) \tag{2.35}$$

b. Second point

The section is completely subjected to tension. We impose $\varepsilon_s = \varepsilon_{su}$, $\varepsilon_c = 0$. We should verify if the upper steel is yielded or not by determining the strain $\varepsilon_s{}'$. The limit axial force and bending moment is obtained from the equations 2.34 and 2.35 respectively.

c. Third point

We impose that the failure is due to concrete and the lower reinforcement is yielded. We assume $\varepsilon_s \geq \varepsilon_{syd}$, $\varepsilon_c = \varepsilon_{cu2}$ and we determine the neutral axis position. Then we should verify if the upper steel is yielded or not by determining the strain $\varepsilon_s{}'$. In order to determine the corresponding stress. The limit axial force and bending moment are obtained from the equations 2.36 and 2.37 respectively.

$$N_{Rd} = -\beta_1.b.x.f_{cd} + A_s f_{yd} - A_s{}' f_{yd} \tag{2.36}$$

$$M_{RD} = f_{yd} A_s \left(\frac{h}{2} - d'\right) + f_{yd} A'_s \left(\frac{h}{2} - d'\right) + \beta_1.b.x.f_{cd} \left(\frac{h}{2} - \beta_2.x\right) \tag{2.37}$$

d. Fourth point

We impose that the failure is due to concrete and the lower reinforcement reaches exactly the yielding point, $\varepsilon_s = \varepsilon_{yd}$. As for the previous point, we determine the neutral axis position and the strain 's. The limit value of the axial force and the bending moment is determined using the equations 2.36 and 2.37 respectively.

e. Fifth point

We impose that the failure is due to concrete and the lower reinforcement reaches exactly $\varepsilon_s = 0$ then the neutral axis position is equal to the effective depth of the section. The limit axial force and bending moment are obtained from the equations 2.38 and 2.39 respectively.

$$N_{Rd} = -\beta_1.b.x.f_{cd} + A_s{}' f_{yd} \tag{2.38}$$

$$M_{RD} = f_{yd} A_s{}' \left(\frac{h}{2} - d'\right) + \beta_1.b.x.f_{cd} \left(\frac{h}{2} - \beta_2.x\right) \tag{2.39}$$

f. Sixth point

We impose that the section is uniformly compressed. We assume $\varepsilon_s = \varepsilon_c \geq \varepsilon_{cu2}$. The limit axial force and bending moment is obtained from the equations 2.40 and 2.41 respectively.

$$N_{Rd} = -\,b.h.f_{cd} - A_s^{'f_{yd}} - A_s f_{yd} \tag{2.40}$$

$$M_{RD} = f_{yd} A_s^{'\left(\frac{h}{2} - d'\right)} - f_{yd} A_s \left(\frac{h}{2} - d'\right) \tag{2.41}$$

The steel reinforcement of the column is considered taking into account the limitations of the Eurocode 2 defined by equation 2.42 and 2.43

.

$$A_{s,min} = max\left(\frac{0.10N_{ED}}{f_{yd}}; 0.002A_c\right) \tag{2.42}$$

$$A_{s,max} = 0.04A_c \tag{2.43}$$

Where:

N_{Ed} is the design axial compression force;

f_{yd} is the design yield strength of the longitudinal reinforcement.

2.7.3.3. Flexible plinth

The flexible footing shall be designed using bending theory, whereby the footing is considered as a cantilever beam. The footing shall be designed for bending moment, shear force and punching shear solicitations. The area of the footing is given by,

$$S = \frac{N_{SLS}}{\sigma_{adm}} \tag{2.44}$$

Where:

N_{SLS} is the axial load on the footing at SLS;

S being the surface area of the footing.

By homotheties,

$$\frac{A}{B} = \frac{a}{b} = K \tag{2.45}$$

Where:

a and b are the designed sections of the column;

A and B are the sections of the footings.

The depth of the footing H is given by,

$$H \leq max\left[\frac{(A-a)}{4}, \frac{(B-b}{4}\right] + 50mm \tag{2.46}$$

For the verification of the bearing capacity of the footing,

$$\sigma_{sol} < \sigma_{adm} \tag{2.47}$$

Where:

σ_{sol} is the allowable bearing capacity of the footing;

σ_{adm} is the allowable bearing capacity of the soil.

$$\sigma_{sol} = \left(\frac{(N_{SLS} + \Upsilon_C * A * B * H)}{S}\right) \qquad (2.48)$$

Where, Υ_C is the specific weight of concrete.

The bending moment used to obtain the steel reinforcement is calculated using soil pressure acting beneath the footing. Pressure distribution at the base of the footing (Figure 2.7) can be calculated from Navier Stoke's equation. Figure 2.8 shows the critical bending momenth position on flexible plinth.

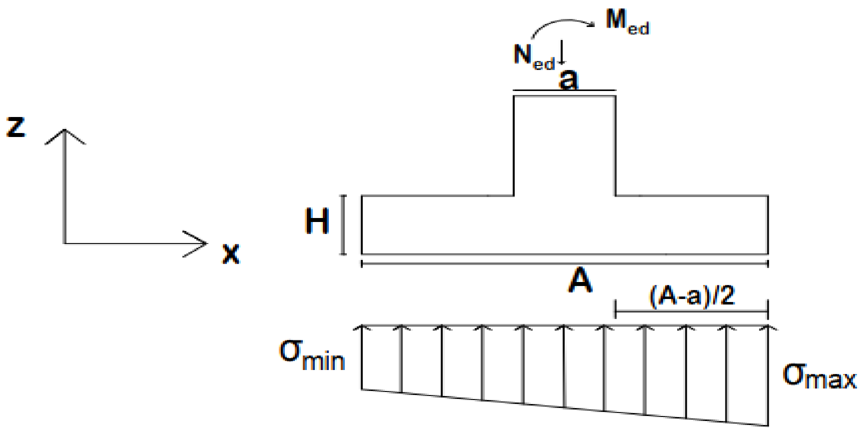


Figure 2.7. Pressure distribution on flexible plinth.

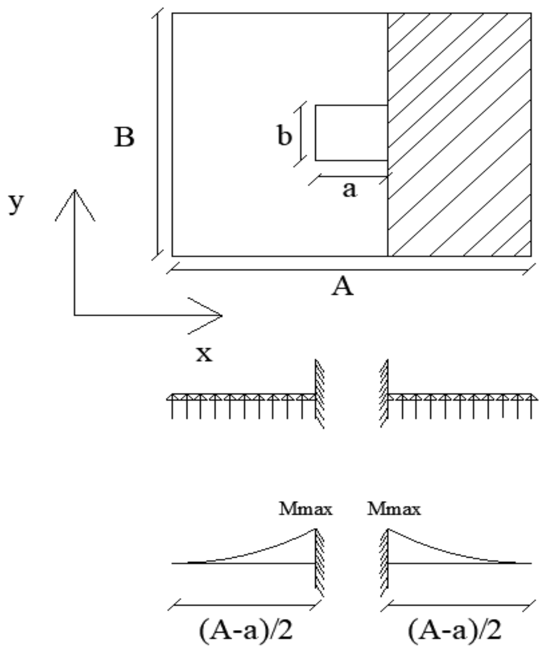


Figure 2.8. Critical bending momenth position on flexible plinth.

For 1 m width of the foundation in the x-z plane, the maximum and minimum values of the soil pressure are given by equations (2.49) and (2.50) respectively.

$$\sigma_{max} = \frac{N_{Ed}}{A} + \frac{M_{Ed}.\left(\frac{A}{2}\right)}{I_y} \tag{2.49}$$

$$\sigma_{min} = \frac{N_{Ed}}{A} - \frac{M_{Ed}.\left(\frac{A}{2}\right)}{I_y} \tag{2.50}$$

Where:

M_{Ed} is the bending moment solicitation from the column around the y-axis;

$I_y = \frac{1\,m \times A^3}{12}$, is the moment of inertia around the y-axis of the foundation plane section.

$$M_{max} = F_1.\frac{(A-a)}{2} + F_2.\frac{2(A-a)}{3} \tag{2.51}$$

Where F_1 and F_2 represented in Figure 2.9 are the resultant forces for soil pressure on one side of the footing section and can be obtained from (2.52) and (2.53) respectively.

$$F_1 = \sigma_f.\frac{A-a}{2}.1m \tag{2.52}$$

$$F_2 = \frac{(\sigma_{max} - \sigma_f)}{2}.\frac{(A-a)}{2}.1m \tag{2.53}$$

With σ_f the soil pressure under the footing, at the face of the column.

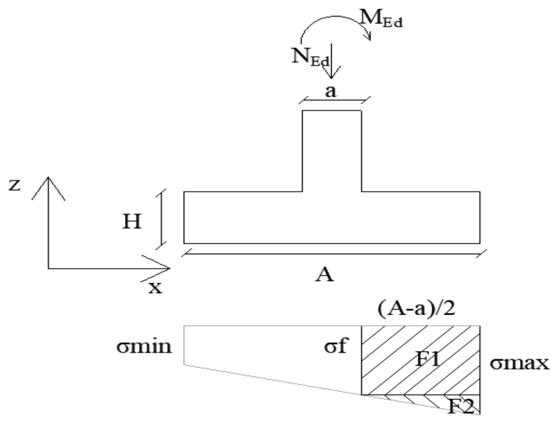


Figure 2.9.F1 and F2 forces on footing.

The longitudinal reinforcement of the footing is given by equations 2.54, 2.55 and 2.56.

$$\frac{M_{Ed}}{0.87f_{yk}z} \leq A_s \tag{2.54}$$

$$A_{s,min} \leq A_s \tag{2.55}$$

$$A_{s,min} = \frac{0.15bd}{100} \tag{2.56}$$

A similar procedure is carried in order to get steel reinforcement in the other direction (y-z plane) for a 1 m length footing.

2.7.3.4. Raft footing

The first step in the design of the raft footing will be the determination of the raft's dimensions. A structural stability verification is done in order to determine the sections and the depth of the raft that will provide us sufficient loads in order to stabilise the structure. Each concrete column exerts a vertical axial load coming from the steel structure on the raft. The calculation of the soil pressure will be done by the ratio of the sum of the axial loads and the self-weight of the raft to the area of the raft. The total soil pressure is calculated using equations 2.47 and 2.48.

The second step is to calculate the longitudinal reinforcement necessary in compression and tension in both x and y directions. The longitudinal reinforcement of the footing is given by equations (2.54) and (2.55)

2.7.3.5. Stability verification of the structure

To ensure structural stability, the effect of stabilising and destabilising actions on the structure should be studied. The stabilising effects concern the actions due to the permanent loads on our tower while the destabilising effects are those actions that have an overturning effect on our structure which in our case are the actions of wind. The respective effects of the destabilising and stabilising actions give:

- An overturning moment M_R;
- A stabilising moment M_S.

We need to verify that the overturning moment is less than the stabilising moment in order to verify the structural stability of our structure through equation 2.57.

$$M_s \geq M_R \tag{2.57}$$

Conclusion

This chapter had as objective to present the different steps necessary to analyze the effects of wind on a telecommunication tower. We started establishing the procedure of collecting data from the field. The norms to be used for the design and verifications of the elements were stated followed by a preliminary design of our structure which will be used to determine the loads on the elements. A clear methodology to obtain the different load patterns and the load combinations on our structure was sighted. We also saw the software we will use to determine the different solicitations on our structure SAP 2000 v22 and a clear description of the steps necessary to design the steel elements present in our structure. This. design will be done manually through the software Excel applying the European standards. Finally, we presented the way the foundation of our structure will be designed and the verifications needed to approve its reliability.

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CHAPTER 3: RESULTS AND INTERPRETATION

Introduction

The methodology presented previously is applied on a case study and the results are highlighted here. This chapter will consist in a preliminary part in the presentation of the case study and the different loads and material properties considered for its analysis. the loads acting on the structure are computed to obtain the solicitations and deflections on our tower. These solicitations are used to determine the different sections of the structural elements of the superstructure and the substructure. After, all the sections being discovered, they will be used to check whether the tower is structurally stable or not.

3.1. General presentation of the city of Douala

The presentation of the city of Douala will be the subject of this section. The city will be presented geographically, the climate and economic conditions in order to know the different conditions which generally have a major influence on the design of structures.

3.1.1. Geographical location

Douala is the largest port in the country and one of the most important in Central Africa, located on the Atlantic Ocean, at the bottom of the Gulf of Guinea, at the mouth of the Wouri. The city stretches along both banks of the shore. Since October 2017, a second bridge stretches across the river to connect the two banks.

3.1.2. Climate

Douala's climate is equatorial, it is characterized by an almost constant temperature of around 26°C and very heavy rainfall, particularly during the rainy season from June to October. The air is almost constantly saturated with humidity, 99% relative humidity in the rainy season, but 80% in the dry season. This rainfall causes frequent flooding, which also contributes to the development of diseases such as cholera and malaria.

3.1.3. Economic parameters

The city of Douala has established itself as the country's economic capital through its port, which has enabled the development of nearly 80% of Cameroon's industrial activity. The port alone accounts for more than 95% of the country's port traffic. The port of Douala-Bonaberi is still the main maritime gateway for Cameroon and the Central African Economic Community, CEMAC. The main products exported are wood (from Cameroon and the Central African

Republic), fruit (especially bananas) and petrol. The country's largest companies have set up their headquarters in Douala rather than in Yaoundé. The city is also home to the African Banana and Plantain Research Centre.

3.2. General presentation of the project

In this part, we will present and describe the case study and present the general characteristics of the materials to be used.

3.2.1. Description of the case study

The tower to be designed is a self-supporting tower with 4 legs, each leg 5m away from the other. The tower has a height of 70m above ground level and will be made up of steel profiles connected to each other, with a straight part at the top 8 m long. All the elements constituting the tower are schifflerized angle sections with equal wings. The material used is steel (235 MPa) due to its availability in the market, its strength and its ease of assembly. The tower will be subdivided into 13 panels for reasons of transport constraints and in order to simplify the analysis and to take into account the variation of the sections with height consequently to have correct results (Table 3.1). The tower is symmetric in all planes, Figure 3.1 shows the 2D representation of the structure.

Table 3.1.Different panels and their corresponding height above the ground.

Panel	Height(m)
1	6
2	12
3	18
4	24
5	30
6	36
7	42
8	48
9	54
10	60
11	62
12	66
13	70

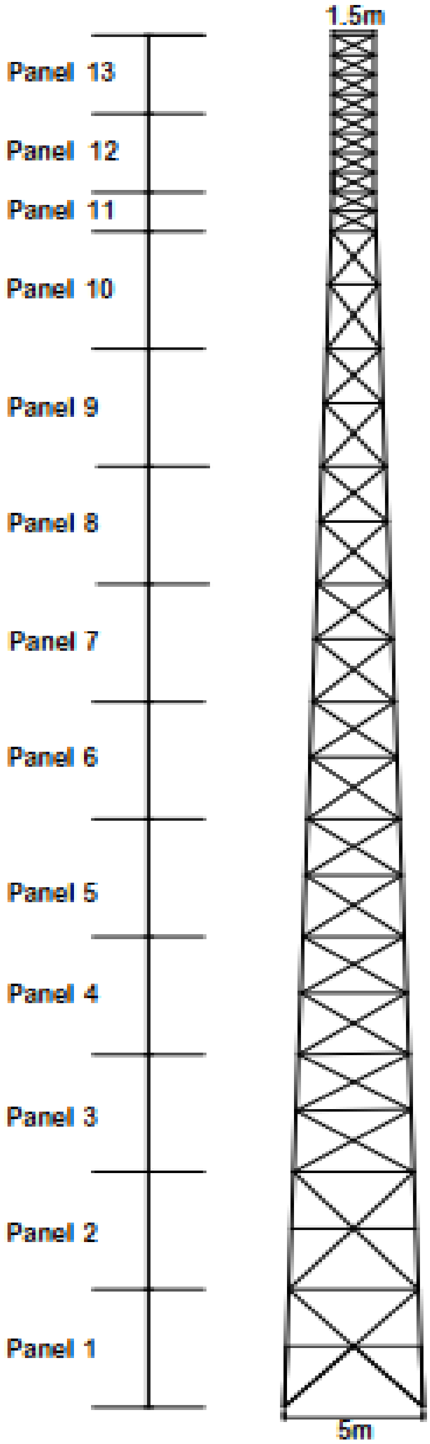


Figure 3.1.70m tall self-supporting telecommunication tower in X-Z and Y-Z plane.

The tower was modelled on SAP 2000 v22 through the following steps.4 nodes were created at the base of our structure that is at z = 0m, the 4 nodes with fixed supports had as coordinates (-2.5, 2.5, 0), (-2.5, -2.5 ,0), (2.5, 2.5, 0) and (2.5, -2.5,0) in the respective directions (X, Y, Z). Thereafter, we created 4 other nodes at a height of z = 62m, having as coordinates (-0.75,0.75,62), (-0.75,-0.75 62), (0.75,0.7 ,62) and (0.75,-0.75,62). Frame elements were created and used to connect the nodes from the base of the tower to those at a height of 62m.The elements were connected following the same order as stated above. We then created the top nodes of our tower at a height of 70m having coordinates (-0.75,0.75, 70), (-0.75,-0.75,70), (0.75,0.75,70) and (0.75,-0.75 ,70). Frame elements were also used to connect the nodes at 62m height to those at the top of the tower. The elements were divided in order to form panels and additional loads and frame elements were created by this division. Moment was released (start and end) in all the elements. Through this procedure the model presented in figure 3.2 was obtained.

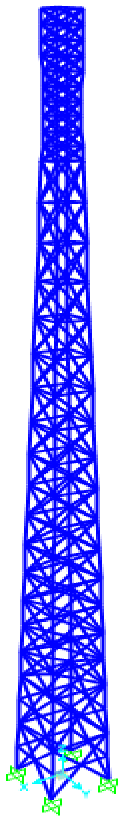


Figure 3.2.3-D view of telecom tower.

Two types of brace configuration will be used in our structure Figure 3.3. Type one is used at the base of our structure up till 12m high and the second type used 12m high to the top of the tower. The base of our structure is 5m large and the top made up of a straight part is 1.5m large.

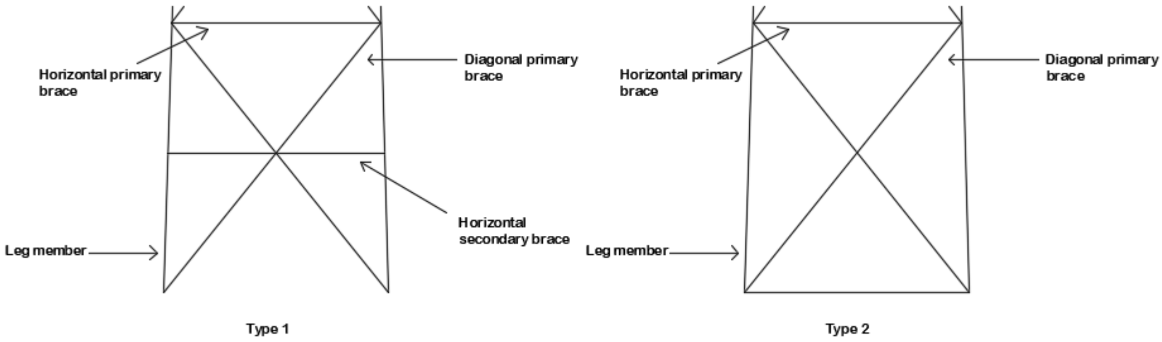


Figure 3.3. Brace configuration and panel elements presentation.

3.2.2. Characteristics and properties of the materials

The structure is made of two main material, steel grade S235 as presented in Table 3.2 for the structural elements and Table 3.3 for concrete C25/30 for the footing.

Table 3.2. Steel structural sections material properties.

Property	**Value**	**Unit**	**Definition**
Steel grade	S235	-	Gives the steel resistance
fy	235	N/mm^2	Characteristic yield strength
fu	360	N/mm^2	Characteristic ultimate strength
E	210 000	N/mm^2	Young modulus
G	80769.231	N/mm^2	Shear modulus
υ	0.3	-	Poisson's ratio
α	0.000012	perK	Coef. of thermal expansion

γ_{M0}	1	-	Safety factor for resistance whatever the section
γ_{M1}	1	-	Safety factor for resistance of members to instability
γ_{M2}	1.25	-	Safety factor for cross section in tension
$f_{ya} = \sigma_{0.2}$	940	N/mm^2	proof strength of bolt grade
f_{ua}	1000	N/mm^2	Ultimate strength of the anchor

Table 3.3. Concrete footing, reinforcing steel and bearing soil material properties.

Property	**Value**	**Unit**	**Definition**
Concrete class	C25/30	-	Concrete class
f_{ck}	25	N/mm^2	Cylindrical crushing strength
E_{cm}	31476	N/mm^2	Static (secant) modulus of concrete
ε_{cu2}	0.35%	-	Ultimate compressive strain of concrete
ε_{cu3}	0.35%	-	Ultimate compressive strain of concrete
γ_c	1.5	-	Safety factor for concrete
Reinf. Steel type	B500C	-	Reinforcement steel type
f_{yk}	500	N/mm^2	Characteristic yield strength of reinforcing steel
E_s	210000	N/mm^2	Modulus of elasticity of reinforcing steel
γ_s	1.15	-	Safety factor for steel
Q_{adm}	0.3	MPa	Admissible soil bearing capacity

3.3. Preliminary design of our structure

As the faces of the tower are identical, Table 3.4 and Table 3.5 gives the preliminary design of the elements of one of the faces of the structure.

Table 3.4. Preliminary design of tower from panel 13 to panel 6.

Panel	Element	Section	Height
13	Member leg Diagonal primary brace Horizontal primary brace	70x7 35x3 35x3	70
12	Member leg Diagonal primary brace Horizontal primary brace	70x7 35x3 35x3	66
11	Member leg Diagonal primary brace Horizontal primary brace	90x5.5 40x4 40x4	62
10	Member leg Diagonal primary brace Horizontal primary brace	110x12 50x6 50x6	60
9	Member leg Diagonal primary brace Horizontal primary brace	110x12 60x5 60x5	54
8	Member leg Diagonal primary brace Horizontal primary brace	110x12 60x5 60x5	48
7	Member leg Diagonal primary brace Horizontal primary brace	120x12 60x5 60x5	42
6	Member leg Diagonal primary brace Horizontal primary brace	120x15 70x5 70x5	36

Table 3.5. Preliminary design of tower from panel 5 to panel 1.

Panel	Element	Section	Height
5	Member leg Diagonal primary brace Horizontal primary brace	120x18 70x5 70x5	30
4	Member leg Diagonal primary brace Horizontal primary brace	140x18 70x5 70x5	24
3	Member leg Diagonal primary brace Horizontal primary brace	140x18 80x5 80x5	18
2	Member leg Diagonal primary brace Horizontal primary brace Horizontal secondary brace	140x18 80x8 80x 40x4	12
1	Member leg Diagonal primary brace Horizontal primary brace Horizontal secondary brace	140x18 80x8 80x 40x4	6

3.4. Determination of loads on our structure

The charges present in our structure are permanent loads, variable loads and wind loads.

3.4.1. Permanent loads

There are 2 kinds of permanent loads acting on our structure that is the self-weight of the structural elements constituting the tower (section profiles and footings) and that of the non-structural elements made up of antennas and rest levels. Table 3.6 shows the loads due to the non-structural elements on the tower.

Table 3.6. Loads due to antennas.

ANTENNAS	GSM	FH1	WCDMA
MASS (kg)	126	21	154.5
WEIGHT(KN)	1.2348	0.2058	1.5141
NUMBER OF NODES	8	4	8
NODAL FORCE (KN)	0.15	0.05	0.19

GSM loads will be placed on 8 nodes at a height of 70m, FH1 loads will be placed at a height of 60m and WCDMA loads at a height of 64m on the tower.

The permanent loads due to the rest levels will be determined and places at their corresponding heights (Table 3.7).

Table 3.7. Permanent loads due to rest level.

Rest level	Height(m)	SURFACE(m2)	G(KN/m2)	G(KN)	Number of nodes	G/Node(KN)
1	68	2.25	0.5	1.125	4	0.28
2	59	2.56	0.5	1.28	4	0.32

Table 3.8. Nature and description of permanent loads

Nature	**Description**
G0	Self weight of structural element
G1	Self weight of non-structural element

3.4.2. Variable loads

The variable loads on our structure are imposed loads and wind loads.

3.4.2.1. Impose load

The main variable loads on the structure will act on the rest levels. They will be calculated with an operating load of 2.5 KN/m2 as given in EUROCODE 3-3-1. The imposed loads on the structure will be of known as Q (Table 3.9).

Table 3.9.Imposed loads due to rest level.

Rest level	Height(m)	SURFACE(m2)	Q(KN/m2)	Q(KN)	Number of nodes	Q/NOEUD(KN)
1	68	2.25	2.5	5.625	4	1.41
2	59	2.56	2.5	6.4	4	1.6

3.4.2.2. Wind loads

The wind acting on our structure acts on the tower and the antennas present in our structure.

a. Overall action on the tower

This wind is made up of two components, one acting normal to the face of the tower and a bisector wind acting on a direction diagonal to the face of the tower. The norm used to obtain the wind loads is NV65.

Here is an example of wind calculation for the first section at the top of the structure (panel 13) following the 2 incidences. Let's start with the normal incidence followed by the incidence along the diagonal.

According to the data presented in 2.5.2.2, we are located in zone 2 hence:
q_{10} = 0.6(KN/m^2);
K_s =1.3 because we are in the littoral;
K_m= 1.

i. Charge normal to the face

Total surface S = 1.5 x 4 = 6m^2
Exposed surface Se = S (main leg) + S (diagonal brace) + S (horizontal brace) = 1.66m^2
ϕ =Se/S = 0.28
H= 50m hence δ = 0.90 from figure 2.3.
Ct = 3,2 - 2 φ donc : Ct = 2.65

$K_h = \frac{q_H}{q_{10}} = 2.5\ \frac{H+18}{H+60}$

$\beta_{normal} = \theta\ (1+ \xi\ \tau)$

$\theta = 1$ for $H > 60m$; hence $\theta = 1$

ξ depends on the period of the fundamental mode of oscillation T

$T\ = CH^{\frac{3}{4}} = 0.05 \times 70$ ^ (3/4) =1.21s

Hence $\xi = 1.4$ from figure 2.2

$\tau = 0.27$

$\beta = 1.38$

Hence the force T on the panel is given by;

T= q_{10} x C_t x K_s x K_mx β x δ x Se = 7.20KN

ii. Charge incident to the diagonal T_{inc}

$\chi = 1+0.6\varphi$

$T_{inc} = T * \chi = 8.40KN$

This load then separates into 2 equal components, one acting normal to the face of the tower and the other one parallel to it.

T incident by face = T_{inc} / ($\sqrt{2}$) = 5.94KN

Table 3.10 and Table 3.11 shows the values of wind loads acting on each panel.

Table 3.10.Wind loads acting on panel 13 down to panel 11.

				Normal	Incident	Incident by face
Panel	Element	Section	Height	T(KN)	T(KN)	T(KN)
13	Member leg Diagonal primary brace Horizontal primary brace	80x8 40x5 40x5	70	7.2	8.39	5.94
12	Member leg Diagonal primary brace Horizontal primary brace	80x8 40x5 40x5	66	7.39	8.68	6.14
11	Member leg Diagonal primary brace Horizontal primary brace	80x8 40x5 40x5	62	3.91	4.62	3.27

Table 3.11. Wind loads acting from panel 10 down to panel 1.

Panel	Element	Section	Height	Normal T(KN)	Incident T(KN)	Incident by face T(KN)
10	Member leg Diagonal primary brace Horizontal primary brace	120x15 80x8 60x8	60	12.29	14.3	10.11
9	Member leg Diagonal primary brace Horizontal primary brace	120x15 80x8 60x8	54	12.33	14.13	9.99
8	Member leg Diagonal primary brace Horizontal primary brace	150x18 90x12 80x12	48	13.22	15.22	10.76
7	Member leg Diagonal primary brace Horizontal primary brace	150x18 90x12 80x12	42	11.9	13.57	9.6
6	Member leg Diagonal primary brace Horizontal primary brace	150x18 90x12 80x12	36	11.11	12.68	8.97
5	Member leg Diagonal primary brace Horizontal primary brace	200x24 120x15 120x15	30	10.9	12.56	8.88
4	Member leg Diagonal primary brace Horizontal primary brace	200x24 120x15 120x15	24	11.04	12.67	8.96
3	Member leg Diagonal primary brace Horizontal primary brace	200x24 120x15 120x15	18	10.75	12.26	8.67
2	Member leg Diagonal primary brace Horizontal primary brace Horizontal secondary brace	200x24 120x15 120x15 40x5	12	10.41	11.84	8.37
1	Member leg Diagonal primary brace Horizontal primary brace Horizontal secondary brace	200x24 120x15 120x15 40x5	6	10.05	11.48	8.12

The wind normal to the face will be knowns as Q_w, the incident to the diagonal will be Q_{wD}.

b. Overall action on the antennas

To obtain the overcall action on the antennas, you first need to obtain the equivalent antenna surface. In our structure the equivalent surface S_{eq} was given to be 12m^2. The actions due to wind on the antenna is supposed to act at the top of the structure thus at a height of 70m above the ground and its action is in the direction normal to the face of the tower.

$T = q_{10} * K_h * K_s * K_m * \delta * \beta * S_{eq} = 19.6\text{KN}$

This action will be loaded on the 4 nodes at the top of the tower hence the charge on each node will be 4.9KN.

3.5. Load combinations

For the verification of the structure, the load combinations are divided in two groups, the Ultimate limit states (ULS) and serviceability state combinations (SLS). A load envelope is obtained for the ULS combinations to have the most unfavorable condition for an element.

$$ULS1: 1.3G_1 + 1.3G_2 \quad (3.1)$$

$$ULS2: 1.3G_1 + 1.3G_2 + 1.5Q \quad (3.2)$$

$$ULS3: 1.3G_1 + 1.3G_2 + 1.5Q + 0.9Q_w \quad (3.3)$$

$$ULS4: 1.3G_1 + 1.3G_2 + 1.05Q + 1.5Q_w \quad (3.4)$$

$$ULS5: 1.3G_1 + 1.3G_2 + 1.5Q + 0.9Q_{wD} \quad (3.5)$$

$$ULS6: 1.3G_1 + 1.3G_2 + 1.05Q + 1.5Q_{wD} \quad (3.6)$$

$$SLS1: G_1 + G_2 + Q + 0.6Q_w \quad (3.7)$$

$$SLS2: G_1 + G_2 + 0.7Q + Q_w \quad (3.8)$$

$$SLS3: G_1 + G_2 + Q + 0.6Q_{wD} \quad (3.9)$$

$$SLS4: G_1 + G_2 + 0.7Q + Q_{wD} \quad (3.10)$$

3.6. Design results and verification

In this chapter, we will use the solicitations obtained from our SAP 2000 model and use them to design the steel elements in our tower, their connections and the foundations. The structural stability check will also be made.

3.6.1. Design of steel profiles

Our structure is divided into panels, the design of our panels has been done and the different solicitation on our elements have been obtained. Our elements work under either compression or tension due to axial loads acting upon them. Shear force and moment on our structure is considered negligeable since the results obtained are very small. All the verifications are done under ULS combination.

Let's start the verification on elements of panel 1 up to 6m height (Figure 3.3). The maximum compression (Table 3.12) and tension (Table 3.13) on each element type will be obtained and used to design and verify the element.

Table 3.12.Maximum compression loads on panel 1 elements.

Type	Element	Compression (KN)
Main leg	8	1010.6
Diagonal primary brace	1006	153.4
Horizontal primary brace	1075	141.4
Horizontal secondary brace	1035	1.84

Table 3.13.Maximum tension loads on panel 1 elements.

Type	Element	Tension (KN)
Main leg	117	851.2
Diagonal primary brace	1202	124.1
Horizontal primary brace	881	173.6
Horizontal secondary brace	1193	1.68

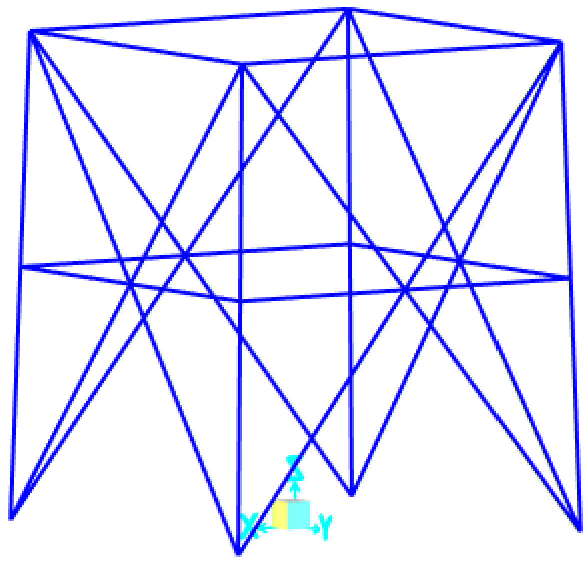


Figure 3.4.3D view of panel 1

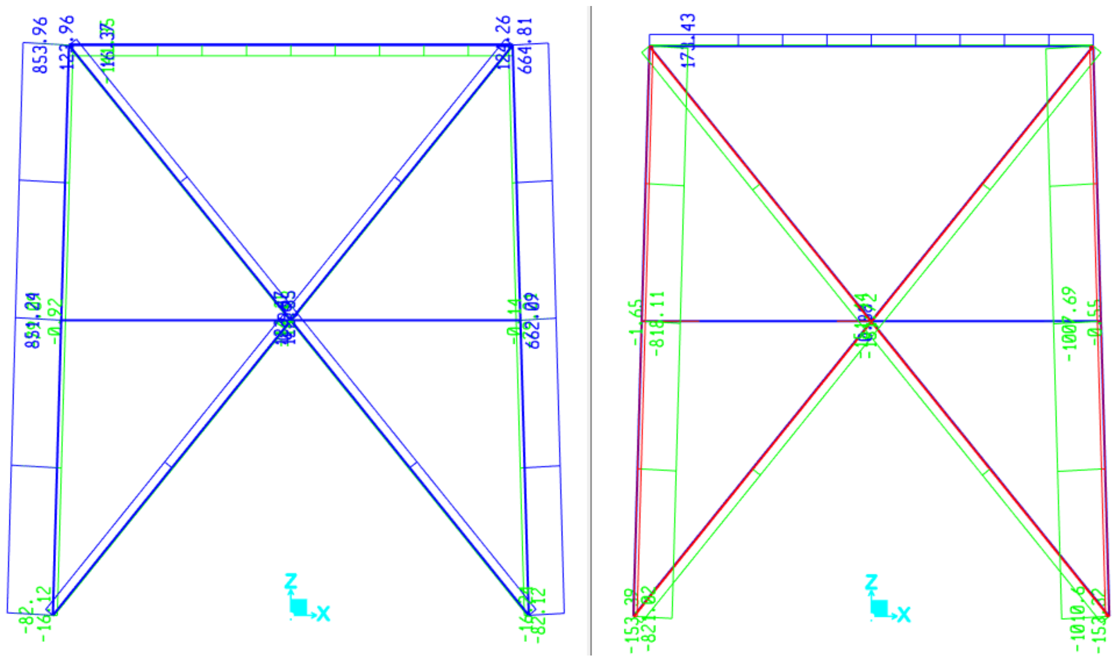


Figure 3.5.X-Z plane view of panel 1 with axial loads.

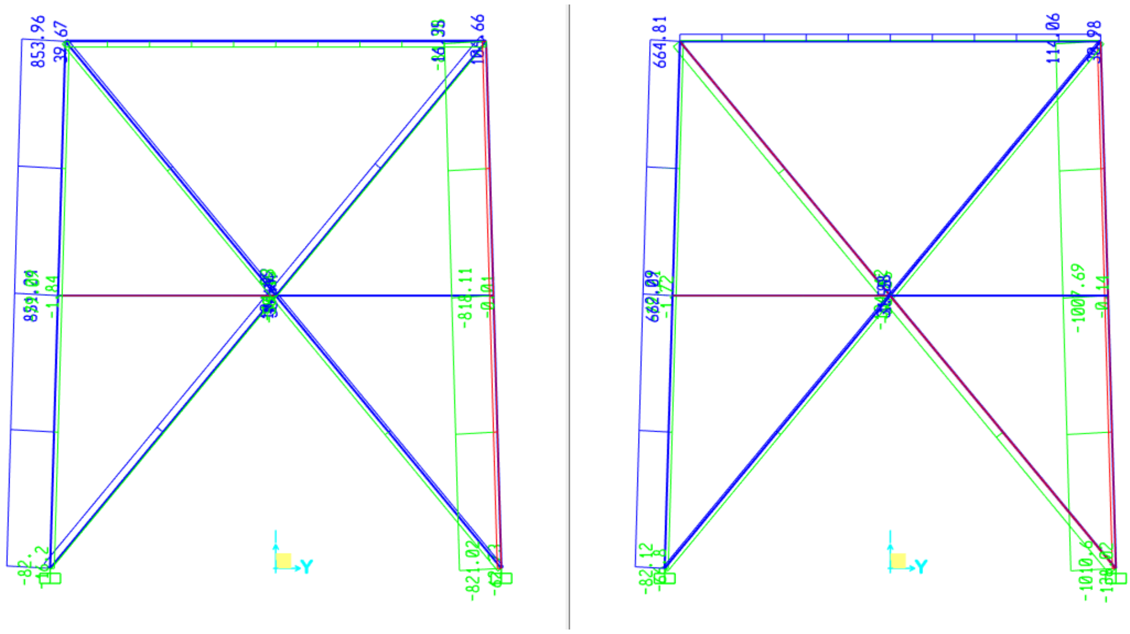


Figure 3.6.Y-Z plane view of panel 1 with axial loads.

- Design of main leg

$$A_{nec}=\frac{N_{Ed}\,\gamma_{M1}}{\chi f_{yk}}$$

For the element in compression, χ= 0.5 hence,

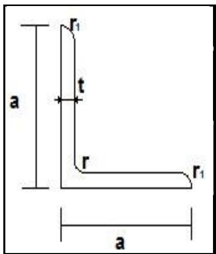
$$A_{nec}=\frac{(1010.6)(1)(10)}{(0.5)(235)}=86\text{cm}^2$$

For the element in compression, χ= 1 hence,

$$A_{nec}=\frac{(851.2)(1)(10)}{(1)(235)}=36.2\text{cm}^2$$

Hence A_{nec} =86cm^2 and the profile chosen is L200X200X24

Table 3.14.L200x200x24 characteristics

	Dimensions					Mass	Area	Characteristics		
	A	B	t	R	r_1	P	A	I_y=I_z	$W_{el,y}$=$W_{el,z}$	i_y=i_z
	Mm	Mm	mm	mm	mm	kg/m	cm^2	cm^4	cm^3	cm
	200	200	24	18	9	71.1	90.6	3330	235	6,06

- Classification of the section

$$\varepsilon = \sqrt{\frac{f_{yk}}{235}} = 1$$

For class 3 sections;

$$\frac{h}{t} < 15\varepsilon \text{ and } \frac{b+h}{2t} < 11.5\varepsilon$$

8.33 < 15 and 8.33 < 11.5; hence class 3 section

- Tension verification

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1\ ;\ N_{Rd} = \frac{Af_y}{\gamma_{Mo}} = (90.6)\ (235)/10 = 2129.1KN$$

Hence N_{Ed}=852.1KN < $N_{t,Rd}$

- Compression verification

$$\frac{N_{Ed}}{N_{c,Rd}} \leq 1\ ;\ N_{c,Rd} = \frac{Af_y}{\gamma_{Mo}} = (90.6)\ (235)/10 = 2129.1KN$$

N_{Ed}=1010.6KN < $N_{c,Rd}$

The steel element shall also be checked for buckling. To do so, the design compressive force should be lower than the buckling resistance force.

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1;\ N_{b,Rd} = \frac{\chi Af_y}{\gamma_{Mo}}$$

$$N_{cr} = \frac{\pi^2 E_x I_x}{{L_0}^2} = \frac{\pi^2(210000)(3330)(10^4)}{3150^2} = 7158.8KN$$

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \sqrt{\frac{(90.6)(10^2)235}{(7158.8)10^3}} = 0.545$$

$$\emptyset = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right] = 0.5[1 + 0.34(0.545 - 0.2) + 0.545^2] = 0.71$$

$$\chi = \frac{1}{\emptyset + \sqrt{\emptyset^2 - \bar{\lambda}^2}} = 0.86 \leq 1$$

$N_{b,Rd} = \chi \frac{Af_y}{\gamma_{M0}} = 1838.6KN$

Hence N_{Ed}=1010.6KN $< N_{b,Rd}$

The other elements of panel 1 are calculated using the same procedure and the result obtained as presented in Table 3.15.

Table 3.15.Panel 1 elements verifications.

Type	Tension(KN)	Compression(KN)	Element	As(cm2)	Nrd(KN)	Nb,rd(KN)
Main leg	851.2	1010.6	200x24	90.6	2129.1	1838.60
Diagonal primary brace	124.1	153.7	120x15	33.9	796.65	393.55
Horizontal primary brace	173.6	141.2	120x15	33.9	796.65	319.11
Horizontal secondary brace	1.68	1.84	40x5	3.79	89.065	16.45

From the table $\frac{N_{Ed}}{N_{b,Rd}} \leq 1$ and $\frac{N_{Ed}}{N_{t,Rd}} \leq 1$ hence all sections are verified.

The elements of panel 1 will be maintained up till the beginning of panel 6 at a height of 30m above the base level.

Panel 6 elements (Figure 3.4) will be designed from the solicitations obtained and the sections obtained will be verified under compression and tension Tables 3.16 and 3.17.

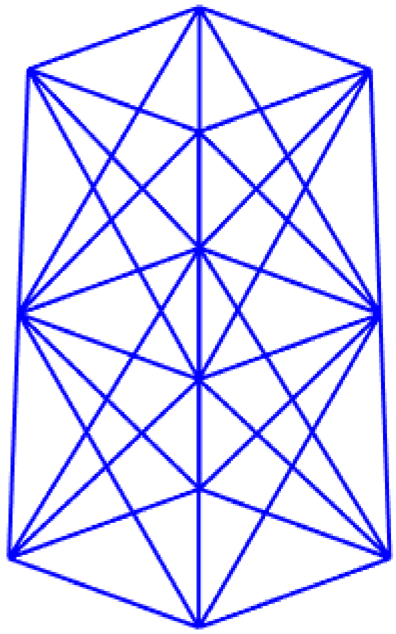


Figure 3.7.3D view of panel 6.

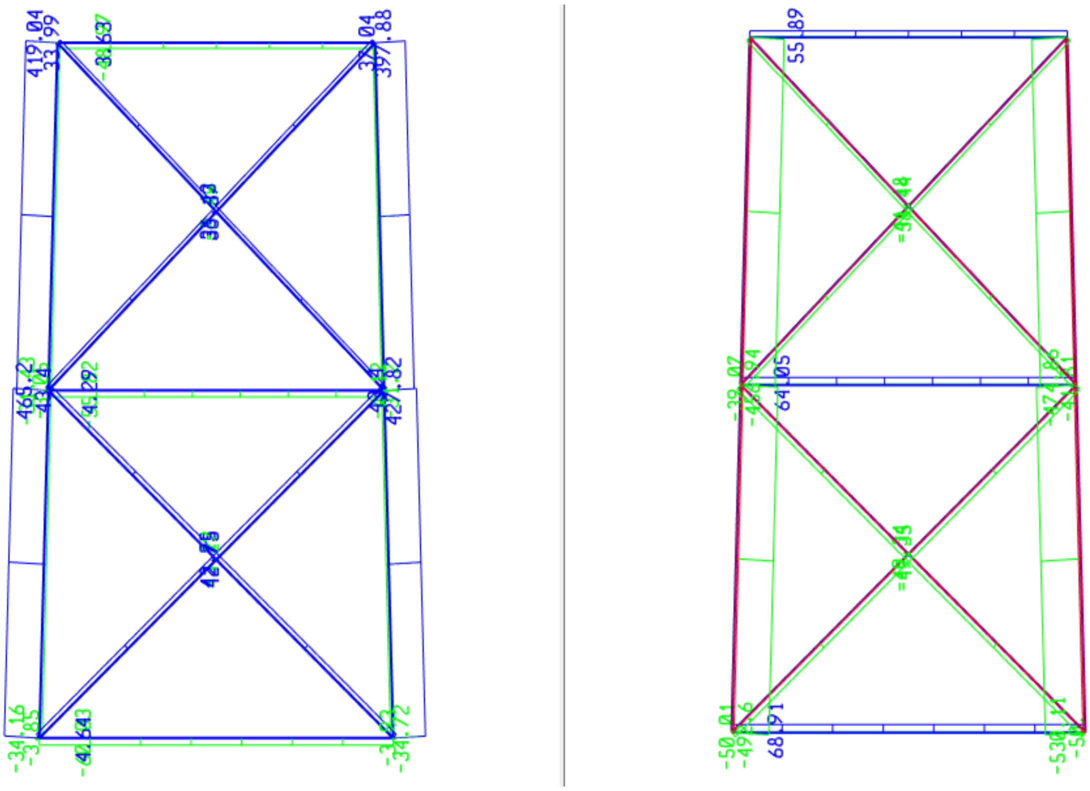


Figure 3.8.X-Z plane view of panel 6 with axial loads.

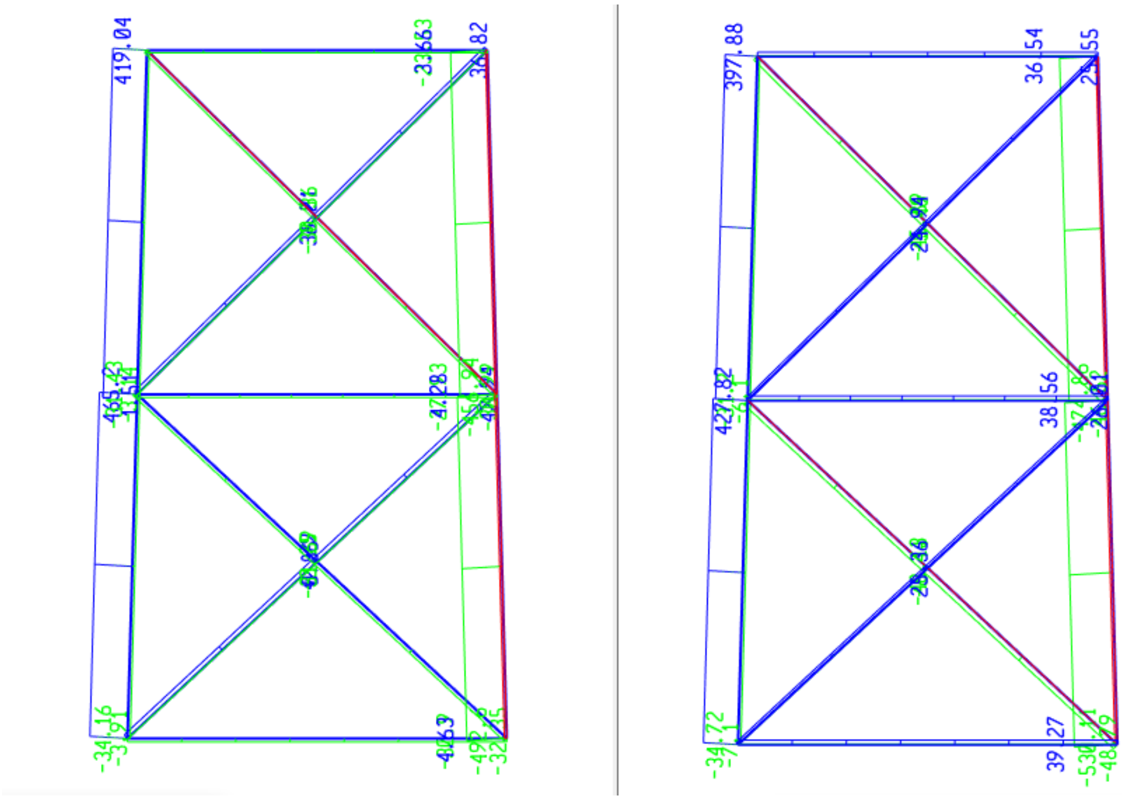


Figure 3.9.Y-Z plane view of panel 6 with axial loads.

Table 3.16.Maximum compression loads on panel 6 elements.

Type	Element	Compression(KN)
Main leg	97	531.2
Diagonal primary brace	937	50.1
Horizontal primary brace	1082	60.03

Table 3.17. Maximum tension loads on panel 6 elements.

Type	Element	Tension(KN)
Main leg	180	464.2
Diagonal primary brace	1114	43.3
Horizontal primary brace	888	68.9

The elements of panel 6 are calculated using the same procedure above and the results are presented in Table 3.18.

Table 3.18. Panel 6 elements verifications.

Type	Tension(KN)	Compression(KN)	Element	As(cm2)	Nrd(KN)	Nb,rd(KN)
Main leg	464.2	531.2	150x18	51	1198.5	1027.68
Diagonal primary brace	43.3	50.1	90x12	20.3	477.05	128.09
Horizontal primary brace	68.9	60.03	80x12	17.9	420.65	156.85

All sections are verified. The sections in panel 6 will be maintained up to the beginning of panel 9.

Panel 9 will be designed from the solicitations obtained and the sections obtained will be verified under compression and tension.

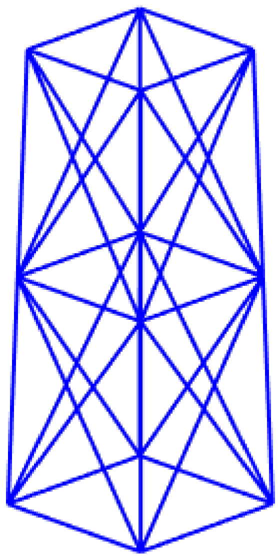


Figure 3.10.3D view of panel 9.

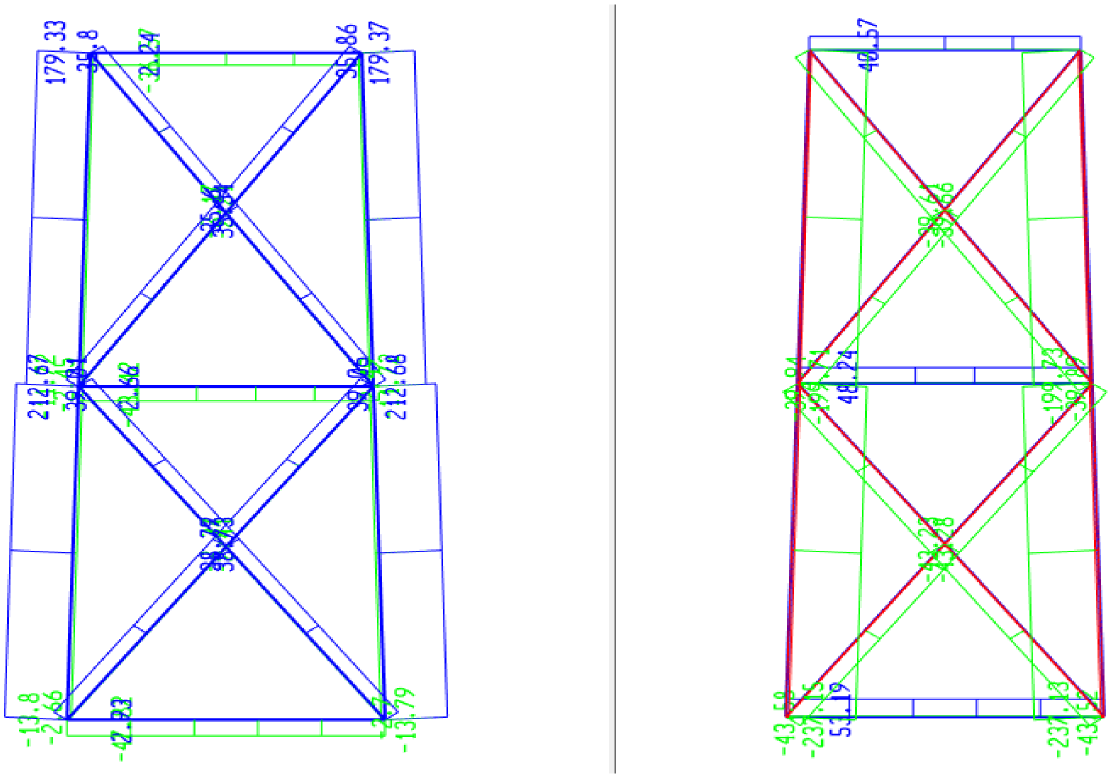


Figure 3.11.X-Z plane view of panel 9 with axial loads.

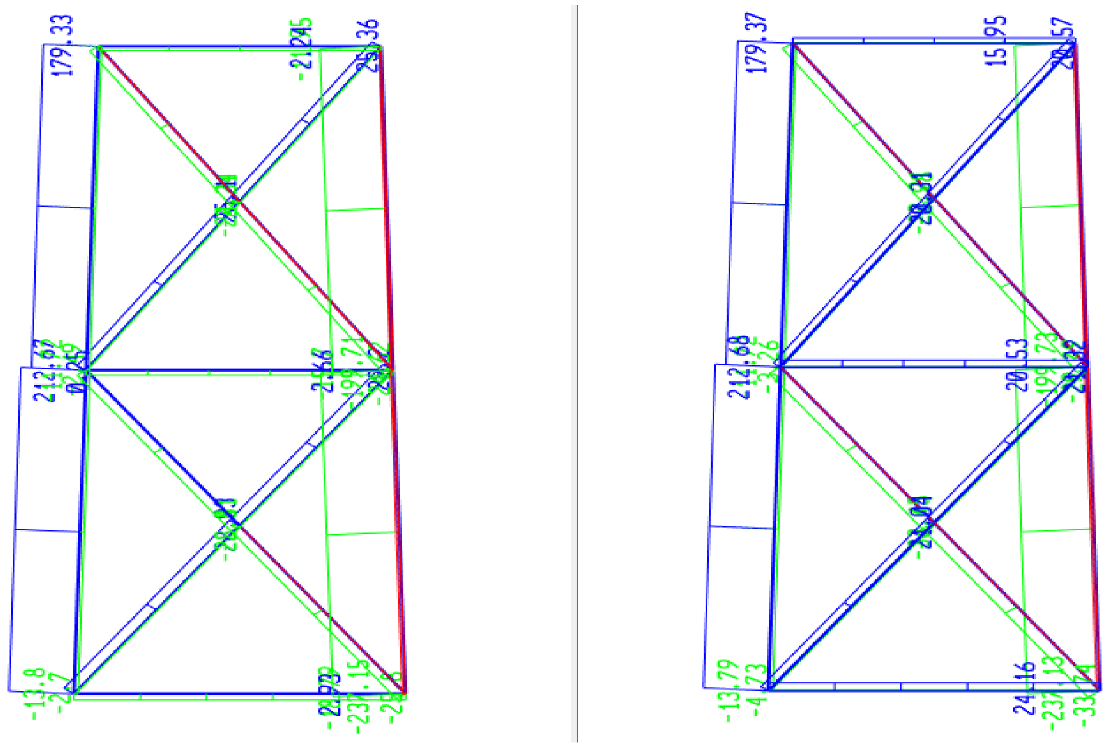


Figure 3.12.Y-Z plane view of panel 9 with axial loads.

Table 3.19. Maximum compression loads on panel 9 elements.

Type	Element	Compression(KN)
Main leg	401	237.1
Diagonal primary brace	949	43.6
Horizontal primary brace	47.8	47.8

Table 3.20. Maximum tension loads on panel 9 elements.

Type	Element	Tension(KN)
Main leg	59	212.7
Diagonal primary brace	1114	39.1
Horizontal primary brace	879	53.2

The sections obtained and their verification is presented in Table 3.21.

Table 3.21.Panel 9 elements verifications.

Type	Tension(KN)	Compression(KN)	Element	As(cm2)	Nrd(KN)	Nb,rd(KN)
Main leg	212.7	237.1	120x15	33.9	796.65	536.09
Diagonal primary brace	39.1	43.6	80x8	12.3	289.05	85.85
Horizontal primary brace	53.2	47.8	60x8	9.03	212.205	84.89

The sections are all verified and will be used up till the beginning of panel 11 at a height of 60m above base level.

The design and verification of panel 11 is then computed.

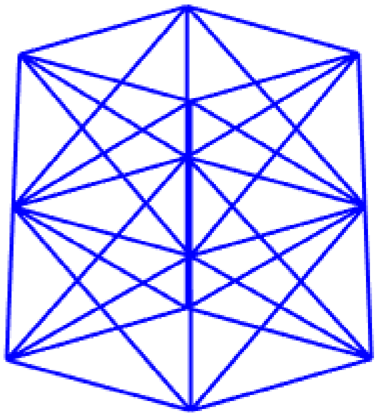


Figure 3.13.3D view of panel 11.

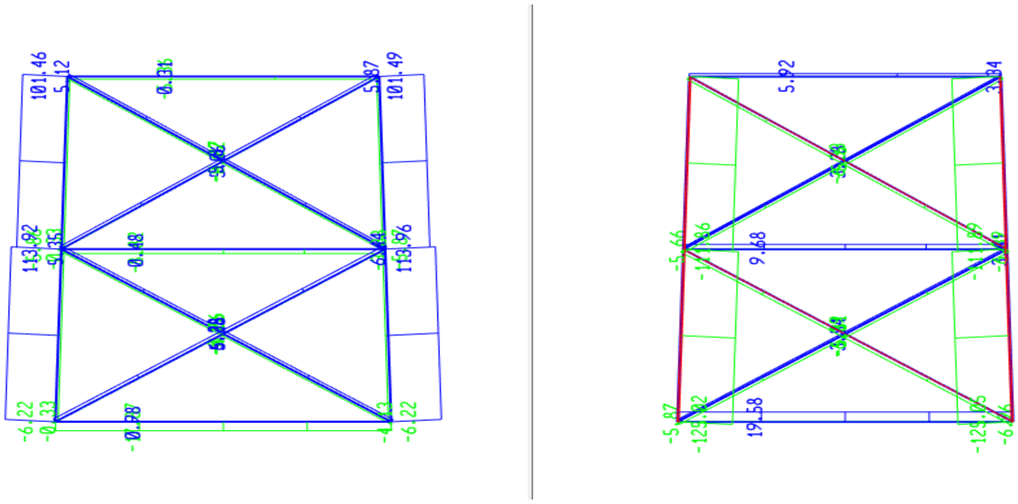


Figure 3.14.X-Z plane view of panel 11 with axial loads.

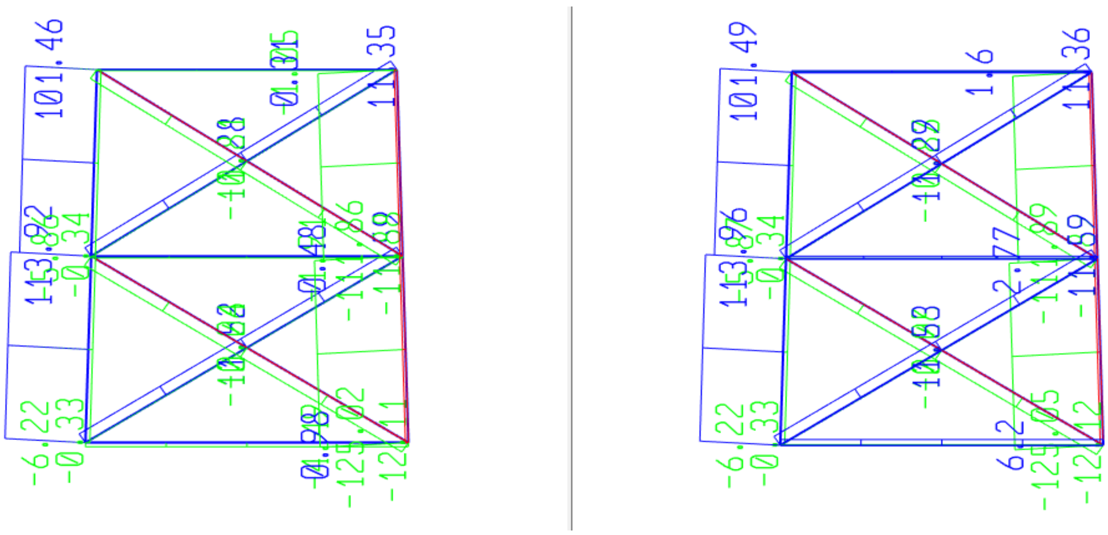


Figure 3. 15.Y-Z plane view of panel 11 with axial loads.

Table 3.22. Maximum compression loads on panel 11 elements.

Type	Element	Compression(KN)
Main leg	48	125.1
Diagonal primary brace	957	12.1
Horizontal primary brace	1069	17.9

Table 3.23.Maximum tension loads on panel 11 elements.

Type	Element	Tension(KN)
Main leg	46	114
Diagonal primary brace	1174	11.6
Horizontal primary brace	875	19.6

The sections obtained and their verification is presented in table 3.24.

Table 3.24.Panel 11 elements verifications.

Type	Tension(KN)	Compression(KN)	Element	As(cm2)	Nrd(KN)	Nb,rd(KN)
Main leg	114	125.1	80x8	12.3	289.05	262
Diagonal primary brace	11.6	12.1	50x4	3.89	91.415	38.01
Horizontal primary brace	19.6	17.9	50x4	3.89	91.415	47.14

These sections will be maintained up till the tow of our tower.

Table 3.25 presents the different sections that have been obtained on the different panels of our telecommunication pylon.

Table 3.25.Panel elements and sections from panel 13 down to panel 3.

Panel	Element	Section	Height
13	Member leg Diagonal primary brace Horizontal primary brace	80x8 40x5 40x5	70
12	Member leg Diagonal primary brace Horizontal primary brace	80x8 40x5 40x5	66
11	Member leg Diagonal primary brace Horizontal primary brace	80x8 40x5 40x5	62
10	Member leg Diagonal primary brace Horizontal primary brace	120x15 80x8 60x8	60
9	Member leg Diagonal primary brace Horizontal primary brace	120xx15 80x8 60x8	54
8	Member leg Diagonal primary brace Horizontal primary brace	150x18 90x12 80x12	48
7	Member leg Diagonal primary brace Horizontal primary brace	150x18 90x12 80x12	42
6	Member leg Diagonal primary brace Horizontal primary brace	150x18 90x12 80x12	36
5	Member leg Diagonal primary brace Horizontal primary brace	200x24 120x15 120x15	30
4	Member leg Diagonal primary brace Horizontal primary brace	200x24 120x15 120x15	24
3	Member leg Diagonal primary brace Horizontal primary brace	200x24 120x15 120x15	18

Table 3. 26. Panel 1 and 2 elements and sections.

Panel	Element	Section	Height
2	Member leg Diagonal primary brace Horizontal primary brace Horizontal secondary brace	200x24 120x15 120x15 40x5	12
1	Member leg Diagonal primary brace Horizontal primary brace Horizontal secondary brace	200x24 120x15 120x15 40x5	6

3.6.2. Displacement verification

The maximum displacement allowed is given by;

U = H/200 = 70/200 = 0.35m = 35cm

Figure 3.16 shows a graph of height against max displacements in both x and y directions on the tower. This is due to wind loads of different magnitude acting in both directions applied in section 3.4.2.2. The coordinates used are presented in a table in annex 1.

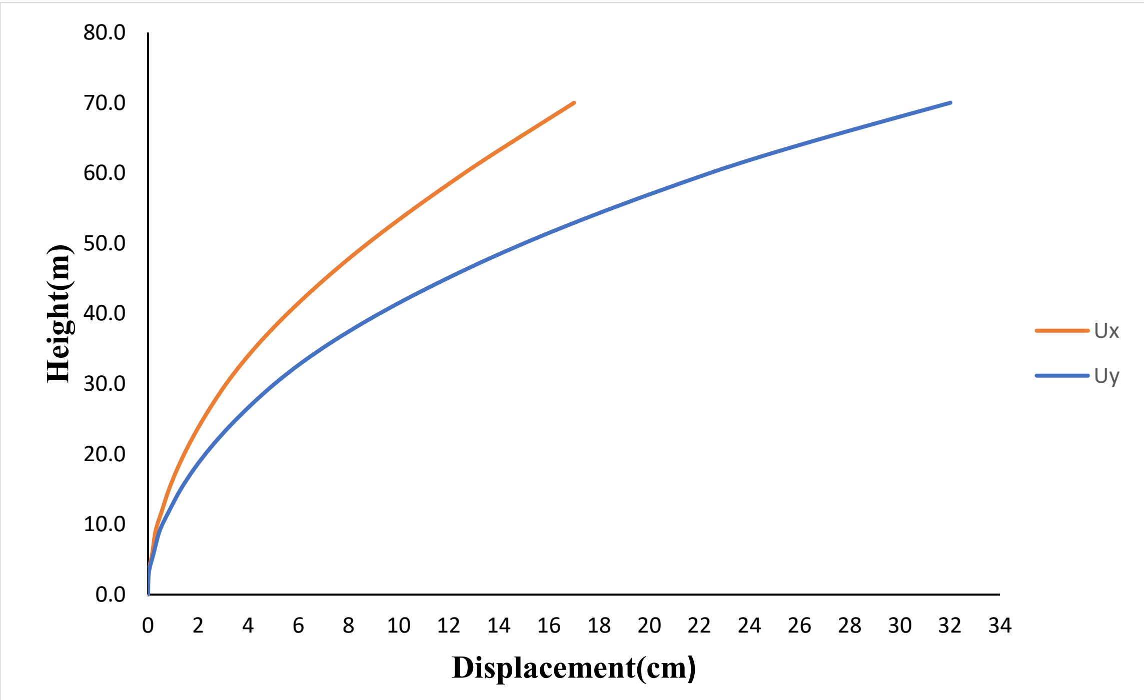


Figure 3.16. Height vs maximum displacement curves of the tower.

The value of the displacement at the tower's head obtained is 32cm hence it is okay.

This displacement is calculated under serviceability limit state under characteristic or rare load combination. The deformation of our tower is presented in Figure 3.10.

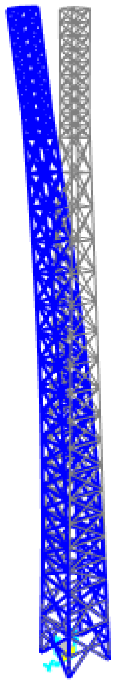


Figure 3.17. Deformed shape of tower.

3.6.3. Design of connections

The overall performance of a steel structure is directly related to its design and particularly to the calculation of the connections of the parts that compose it. The connections are a device for joining and securing the parts together and used for the transmission and distribution of the various stresses in the different structural components. The main methods of connection are bolting and welding. In our project, bolting is the type of connection applied to our structure. The study of the connection of the members consists of giving the constructive layout of the connection while determining the number of bolts required and sizing the plate. The next step is to check the modes of failure of the connection, which are shear resistance of the bolt and the bearing capacity

3.6.3.1. Member leg-member leg connection

The joint of the member legs is made up of its connection by bolting. The loads are transferred directly between the ends of the sections. The two profiles are joined together by means of double plate that maintain their alignment. Only the normal tensile and compressive forces act on the members. The connection we will consider is that of the member leg between panel 1 and panel 2 at 6m height.

Table 3.27. Characteristics of the elements at the connection between panel 1 and 2 member legs.

Lower member leg		Upper member leg		Bolts	
Profile	L200x200x24	Profile	L200x200x24	Class	8.8
a [mm]	200	a [mm]	200	f_{yb} [MPa]	640
b [mm]	200	b [mm]	200	f_{ub} [MPa]	800
t [mm]	24	t [mm]	24	d [mm]	16
y[cm]	5.4	y[cm]	5.4	d_0 [mm]	17
f_y [MPa]	235	f_y [MPa]	235	A [mm2]	201.1
f_u [MPa]	360	f_u [MPa]	360		

Ned = 1005.1KN

$F_{v,Rd} = \frac{\alpha_v \, f_{ub} \, A_b n_r}{\gamma_{Mb}} = \frac{(0.6)(800)(201.1)(2)(10^{-3})}{1.25} = 154.4KN$

Number of bolts n = 8 bolts

$F_{v,ed} = \frac{N_{ed}}{n} = 125.6KN$

$F_{v,ed} < F_{v,Rd}$ hence bolts resist to shear.

We must now check the bearing resistance; $F_{v,ed} \leq F_{b,Rd} = \frac{K_1 \, \alpha_b \, f_u \, d_0 \, t}{\gamma_{M2}}$

$$\alpha_b = \min\left\{\left(\frac{p_1}{3d_0} - 0,25\right); \frac{f_{ub}}{f_u}; 1\right\}$$

$$K_1 = \min\left\{\left(\frac{2,8e_2}{d_0} - 1,7\right); 2,5\right\}$$

t = 15mm

$1.2d_0 \leq e_2 \leq 4t + 40mm$, hence $20.4mm \leq e_2 \leq 100mm$, thus $e_2 = e_1 = 60mm$

$1.2d_0 \leq e_1 \leq 4t + 40mm$

$2.2d_0 \leq P_1 \leq \min(14t, 200mm)$, hence $37.4\ mm \leq P_1 \leq 200mm$, thus $P_1 = 70mm$

$2.4d_0 \leq P_2 \leq \min(14t, 200mm)$, $P_2 = 70mm$

Thus $\alpha_b = 1$ and $K_1 = 2.5$

$$F_{b,Rd} = \frac{K_1\ \alpha_b\ f_u\ d_0\ t}{\gamma_{M2}} = \frac{(2.5)(1)(360)(16)(15)(10^{-3})}{1.25} = 172.8KN$$

$$F_{b,ed} = \sqrt{\left((\frac{N_{ed}}{8})^2 + (\frac{M_{ed}}{3P_1})^2\right)}$$

$$M_{ed} = \frac{N_{ed}}{8}(e + e')$$

$e = e_2 - y = 16mm$, $e' = e_2 + P_2 - y = 71.6mm$

$F_{b,ed} = 136.1.0KN \leq F_{b,Rd}$

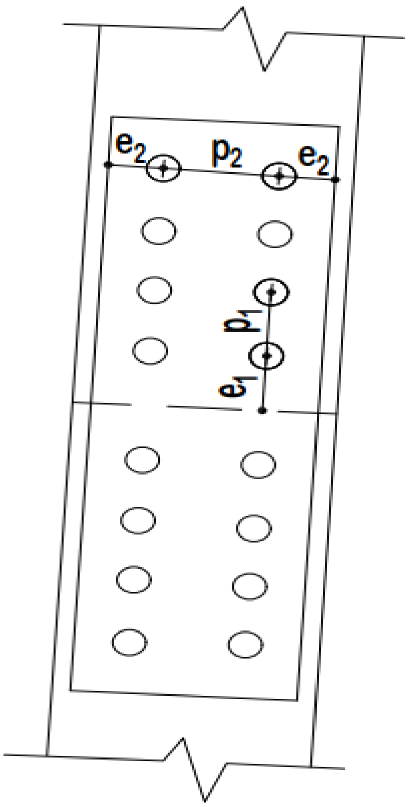


Figure 3.18.Member leg joint connection.

3.6.3.2. Member leg-diagonal primary brace connection

The elements we will consider belong to panel 1.

Table 3.28. Characteristics of the elements at the connection between panel 1 member leg and diagonal primary brace.

Member leg		Diagonal primary brace		Bolts	
Profile	L200x200x24	Profile	L120x120x15	Class	8.8
a [mm]	200	a [mm]	120	f_{yb} [MPa]	640
b [mm]	200	b [mm]	120	f_{ub} [MPa]	800
t [mm]	24	t [mm]	15	d [mm]	16
y[cm]	5.4	y[cm]	3.51	d_0 [mm]	17
f_y [MPa]	235	f_y [MPa]	235	A [mm2]	201.1
f_u [MPa]	360	f_u [MPa]	360		

The axial load N_{ed} on the diagonal N_{ed} = 153.7KN

$$F_{v,Rd} = \frac{\alpha_v\, f_{ub}\, A_b n_r}{\gamma_{Mb}} = \frac{(0.6)(800)(201.1)(2)(10^{-3})}{1.25} = 154.4KN$$

Number of bolts n = 2 bolts

$$F_{v,ed} = \frac{N_{ed}}{n} = 76.9KN$$

$F_{v,ed} < F_{v,Rd}$ hence bolts resist to shear.

We must now check the bearing resistance; $F_{v,ed} \leq F_{b,Rd} = \frac{K_1\, \alpha_b\, f_u\, d_0\, t}{\gamma_{M2}}$

$$\alpha_b = \min\left\{\left(\frac{p_1}{3d_0} - 0{,}25\right); \frac{f_{ub}}{f_u}; 1\right\}$$

$$K_1 = \min\left\{\left(\frac{2{,}8e_2}{d_0} - 1{,}7\right); 2{,}5\right\}$$

t = 15mm

$1.2d_0 \leq e_2 \leq 4t + 40mm$, hence $20.4mm \leq e_2 \leq 100mm$, thus $e_2 = e_1 = 60mm$

$1.2d_0 \leq e_1 \leq 4t + 40mm$

$2.2d_0 \leq P_1 \leq$ min (14t, 200mm), hence 37.4 mm $\leq P_1 \leq$ 200mm, thus P_1 = 50mm

Thus α_b= 0.73 and K_1=2.5

$$F_{b,Rd} = \frac{K_1\, \alpha_b\, f_u\, d_0\, t}{\gamma_{M2}} = \frac{(2.5)(0.73)(360)(16)(15)(10^{-3})}{1.25} = 126.1KN$$

$$F_{b,ed} = \sqrt{\left(\left(\frac{N_{ed}}{2}\right)^2 + \left(\frac{M_{ed}}{P_1}\right)^2\right)}$$

$$M_{ed} = \frac{N_{ed}}{2}(e)$$

$e = e_2 - y$ = 24.9mm

$F_{b,ed}$ = 85.9KN $\leq F_{b,Rd}$

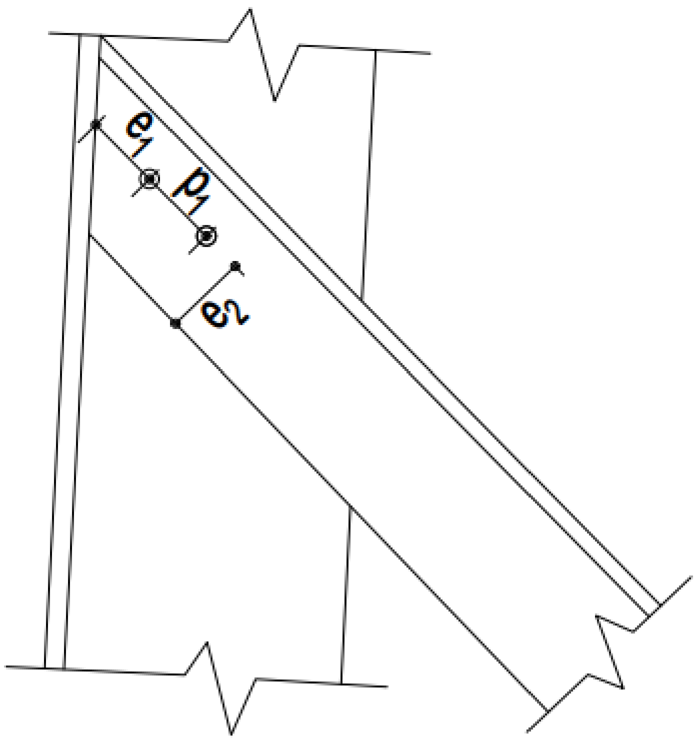


Figure 3.19.Member leg and diagonal primary brace connection.

3.6.3.3. Member leg-horizontal primary brace connection

The elements we will consider belong to panel 1.

Table 3.29.Characteristics of the elements at the connection between panel 1 member leg and horizontal primary brace.

Member leg		Diagonal primary brace		Bolts	
Profile	L200x200x24	Profile	L120x120x15	Class	8.8
a [mm]	200	a [mm]	120	f_{yb} [MPa]	640
b [mm]	200	b [mm]	120	f_{ub} [MPa]	800
t [mm]	24	t [mm]	15	d [mm]	16
y[cm]	5.4	y[cm]	3.51	d_0 [mm]	17
f_y [MPa]	235	f_y [MPa]	235	A [mm2]	201.1
f_u [MPa]	360	f_u [MPa]	360		

The axial load N_{ed} on the horizontal brace is N_{ed} = 173.6KN

$$F_{v,Rd} = \frac{\alpha_v\, f_{ub}\, A_b n_r}{\gamma_{Mb}} = \frac{(0.6)(800)(201.1)(2)(10^{-3})}{1.25} = 154.4KN$$

Number of bolts n = 2 bolts

$$F_{v,ed} = \frac{N_{ed}}{n} = 86.8KN$$

$F_{v,ed} < F_{v,Rd}$ hence bolts resist to shear.

We must now check the bearing resistance; $F_{v,ed} \leq F_{b,Rd} = \frac{K_1\, \alpha_b\, f_u\, d_0\, t}{\gamma_{M2}}$

$$\alpha_b = \min\left\{\left(\frac{p_1}{3d_0} - 0{,}25\right); \frac{f_{ub}}{f_u}; 1\right\}$$

$$K_1 = \min\left\{\left(\frac{2{,}8e_2}{d_0} - 1{,}7\right); 2{,}5\right\}$$

t = 15mm

$1.2d_0 \leq e_2 \leq 4t + 40mm$, hence $20.4mm \leq e_2 \leq 100mm$, thus $e_2 = e_1 = 60mm$

$1.2d_0 \leq e_1 \leq 4t + 40mm$

$2.2d_0 \leq P_1 \leq$ min (14t, 200mm), hence 37.4 mm $\leq P_1 \leq$ 200mm, thus P_1 = 50mm

Thus α_b= 0.73 and K_1=2.5

$$F_{b,Rd} = \frac{K_1\, \alpha_b\, f_u\, d_0\, t}{\gamma_{M2}} = \frac{(2.5)(0.73)(360)(16)(15)(10^{-3})}{1.25} = 126.1KN$$

$$F_{b,ed} = \sqrt{\left((\frac{N_{ed}}{2})^2 + (\frac{M_{ed}}{P_1})^2\right)}$$

$$M_{ed} = \frac{N_{ed}}{2}\ (e)$$

e = e_2 - y = 24.9mm

$F_{b,ed}$ =97.0 KN $\leq F_{b,Rd}$

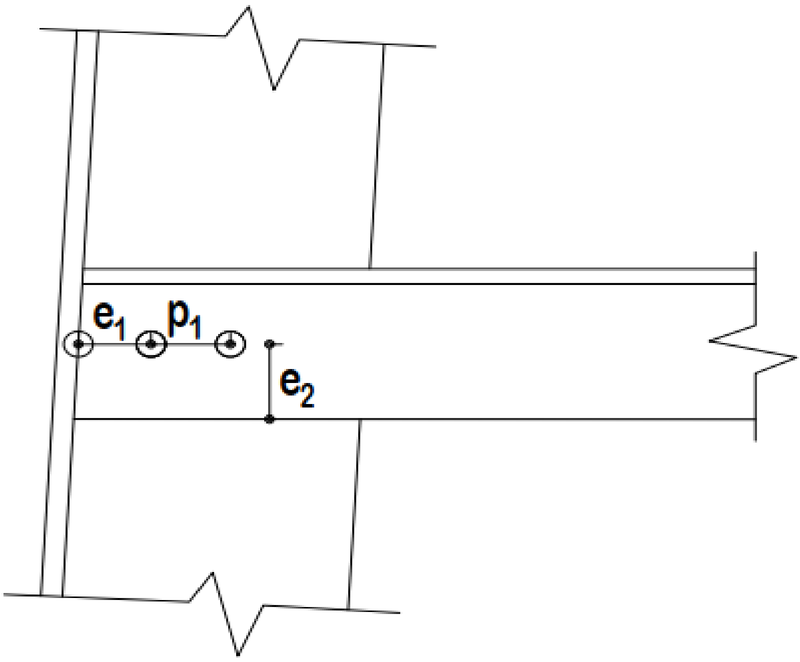


Figure 3.20. Member leg and horizontal primary brace connection.

The axial load on the horizontal secondary brace is too small hence no calculation needs to be done to determine the number of bolts for the its connection with the member leg. One bolt of 8mm diameter of same class as the other bolts above will be sufficient.

3.6.3.4. Column base plate connection

The forces acting on the base plate are;

-Uplift force N_a=1012.6KN

-Compression Nc,ed = 1010.6KN

-Tension Nt,ed = 851.2KN

-Shear force Ved,x = 120.7KN

- Shear force Ved,y =94.2KN

The area of the base plate A is obtained from equation 2.29

$$A = B.C \geq \frac{N_{ed}}{f_{cd}} = \frac{1012.6}{\frac{0.85*25}{1.5}} = 715\text{cm}^2$$

A = 30x30 = 900cm^2

$$t_p \geq \sqrt{\frac{2N_{ed}m_2}{BCf_{yk}}} = 15.4\text{mm} \text{ , hence } t_p = 20mm$$

The anchor length is calculated using anchor bolt of 30mm diameter and of class 8.8 from equation 2.30;

$$L_s = \frac{N_a}{\pi\emptyset\tau_{su}} = \frac{253.1*10^3}{\pi*30*2.84} = 947.9 \text{ cm, hence } L_s = 1m$$

The shear resistance is calculated,

$$\mathrm{F_{v,Rd}} = \frac{\alpha_v \, f_{ub} \, A_b n_r}{\gamma_{Mb}} = \frac{(0.6)(800)(706.9)(1)(10^{-3})}{1.25} = 271.4\text{KN}$$

$$\mathrm{V_{Ed}} = \frac{120.7}{4} = 30.2KN \leq \mathrm{F_{v,Rd}}$$

The bearing resistance of the plate;

$$\mathrm{F_{b,Rd}} = \frac{K_1 \, \alpha_b \, f_u \, d_0 \, t}{\gamma_{M2}} = \frac{(2.5)(1)(360)(30)(20)(10^{-3})}{1.25} = 432\text{KN}$$

$$\mathrm{F_{b,ed}} = 30.2\text{KN} \leq \mathrm{F_{b,Rd}}$$

The traction resistance;

$$\mathrm{F_{t,Rd}} = \frac{0.9 \, f_{ub} \, A_b n_r}{\gamma_{Mb}} = \frac{(0.9)(800)(706.9)(1)(10^{-3})}{1.25} = 407.2\text{KN}$$

$$\mathrm{F_{t,Ed}} = \frac{851.4}{4} = 212.9N \leq \mathrm{F_{t,Rd}}$$

3.6.4. Column design

The concrete column has a horizontal force acting at its top which will create a maximum moment at the bottom of the column.

Compressive force Ned = 1010.6KN

Moment Med,x = 120.6 x 1m = 120.6KNm

Moment Med,y = 94.2 x 1m = 94.2KNm

The column is of square dimension 40x40. The longitudinal reinforcement is obtained from equations 2.38 and 2.39;

$A_{s,min} = max\left(\frac{0.10N_{ED}}{f_{yd}}; 0.002A_c\right)$ and $A_{s,max} = 0.04A_c$

$A_{s,min}$= max (258.2mm^2 ; 320mm^2)

$A_{s,max}$= 6400mm^2

The area of longitudinal reinforcement is 853.7mm^2 which corresponds to 6Ø12 and 2Ø.

The M-N diagrams for the column are presented in figure 3.28 and figure 3.29 for x-direction and y-direction respectively.

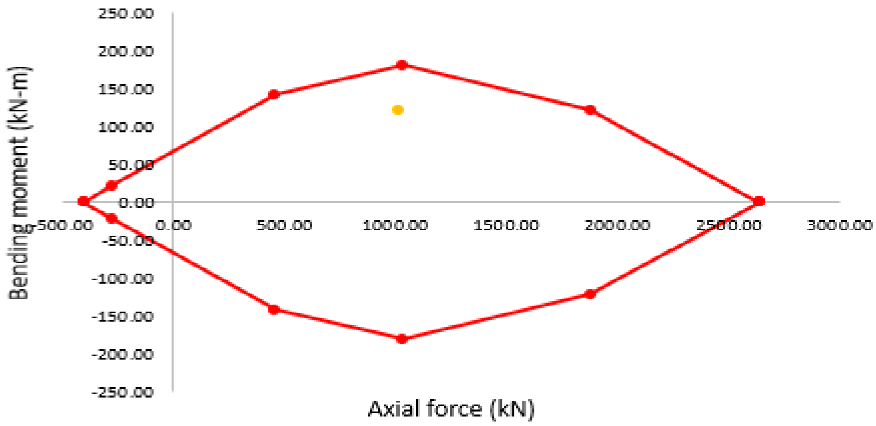


Figure 3.21.MN interaction curve of column in X direction.

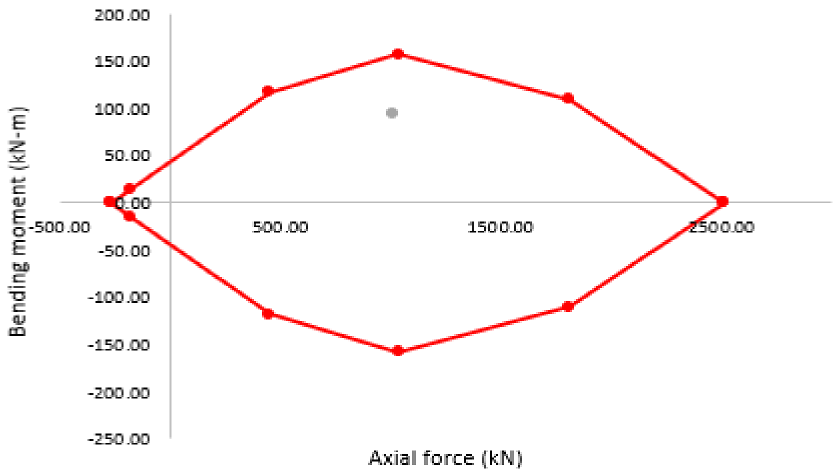


Figure 3.22.MN interaction curve of column in Y direction.

Figure 3.23 shows the details of the designed column.

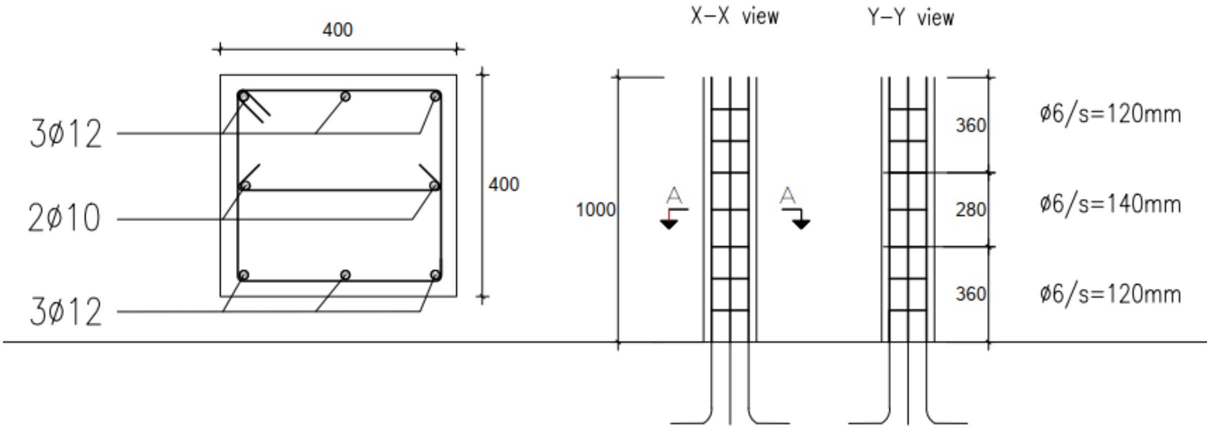


Figure 3.23.Column detailing.

3.6.5. Flexible plinth design

We shall design the footing as a flexible plinth. To do so, we follow the design procedure described in section 2.7.2.2.

Self- weight of column = (25) (0.4^2) (1) = 4KN

Table 3.30. Moment and Axial loads on footing at ULS and SLS

	Ned(KN)	MedY(KNm)	MedX(KNm)
ULS	1015.8	120.6	94.2
SLS	362.9	81.67	62.86

$S = \frac{N_{SLS}}{\sigma_{adm}} = \frac{362.1KN}{0.2MPa} = 1.8\text{m}^2$

With an area of 1.8 m^2, we shall use a 1.5x1.5m footing hence S = 2.25m^2.

The depth of the footing H is given by;

$H \leq max\left[\frac{(A-a)}{4}, \frac{(B-b)}{4}\right] + 50mm$, hence $H \leq$ 32.5mm so H = 30mm

$\sigma_{sol} = \left(\frac{(N_{SLS}+ \Upsilon_C * A * B * H)}{S}\right) = \frac{(362.9KN+\frac{25KN}{m3}* 1.5*1.5*0.3m3)}{1.5^2 m2} = 0.17\text{Mpa}$

$\sigma_{sol} < \sigma_{adm}$= 0.3Mpa

- Maximum moment acting on the X-Z plane with Med,y = 120.6KNm

$$\sigma_{max} = \frac{N_{Ed}}{A} + \frac{M_{Ed}\cdot\left(\frac{A}{2}\right)}{I_y} = 0.77 \text{ MPa}$$

$$\sigma_{min} = \frac{N_{Ed}}{A} - \frac{M_{Ed}\cdot\left(\frac{A}{2}\right)}{I_y} = 0.13\text{MPa}$$

The critical section for bending moment is found at the face of the column;

$\sigma_F = 0.54\text{MPa}$

$$F_1 = \sigma_f.\frac{A-a}{2}.1m = \frac{0.54Mpa*1100mm*1000mm}{2} = 297\text{KN}$$

$$F_2 = \frac{(\sigma_{max}-\sigma_f)}{2}.\frac{(A-a)}{2}.1m == \frac{(0.77-0.54)Mpa*(1100mm)*(1000mm)}{2*2} = 66\text{KN}$$

$$M_{max} = F_1.\frac{(A-a)}{2} + F_2.\frac{2(A-a)}{3} = 211.8\text{KNm}$$

The longitudinal reinforcement of the footing A_s;

$$A_s \geq \frac{M_{max}}{0.87f_{yk}Z} = \frac{211.8KNm}{0.87(\frac{450N}{mm2})(0.9*250mm)} = 24.04\text{cm}^2$$

Hence A_s for total length A= 1.5m is given by;

$A_s = 24.04*\ 1.5 = 36.1\ \text{cm}^2$, thus we shall consider 16Ø18 = 40.6 cm^2

- Now let us consider the plane Y-Z with Medx = 94.2KNm

$$\sigma_{max} = \frac{N_{Ed}}{A} + \frac{M_{Ed}\cdot\left(\frac{A}{2}\right)}{I_y} = 0.71\text{MPa}$$

$$\sigma_{min} = \frac{N_{Ed}}{A} - \frac{M_{Ed}\cdot\left(\frac{A}{2}\right)}{I_y} = 0.2\text{MPa}$$

The critical section for bending moment is found at the face of the column;

$\sigma_F = 0.52\text{MPa}$

$$F_1 = \sigma_f.\frac{A-a}{2}.1m = \frac{0.52Mpa*1100mm*1000mm}{2} = 286\text{KN}$$

$$F_2 = \frac{(\sigma_{max}-\sigma_f)}{2}.\frac{(A-a)}{2}.1m == \frac{(0.71-0.52)Mpa*(1100mm)*(1000mm)}{2*2} = 52.3\text{KN}$$

$$M_{max} = F_1.\frac{(A-a)}{2} + F_2.\frac{2(A-a)}{3} = 195.7\text{KNm}$$

The longitudinal reinforcement of the footing A_s;

$$A_s \geq \frac{M_{max}}{0.87 f_{yk} Z} = \frac{195.7KNm}{0.87(\frac{450N}{mm2})(0.9*250mm)} = 23.0\text{cm}^2$$

Hence A_s for total length A= 1.5m is given by;

$A_s = 23.0 * 1.5 = 34.5\text{cm}^2$, thus we shall consider 16Ø18 = 40.6 cm^2

To verify the structural stability of the tower, we will determine the stabilizing moment and the overturning moment on our structure.

- Stabilising moment M_s

The stabilising effect on the tower is obtained from the following:

-The vertical reactions at the base joints due to the actions of the self-weight of the tower and the load of antennas and rest levels;

-The self-weight of the 4 columns below the column-base plate of the tower;

-The self-weight of the 4 flexible plinths at foundations.

- Overturning moment M_R

The overturning effect is due to:

-The overall wind action acting in the y direction T_1 acting at a height of 31.5m;

-The overall wind action on the antennas T_2 acting at the top of the tower hence at a height of 70m.

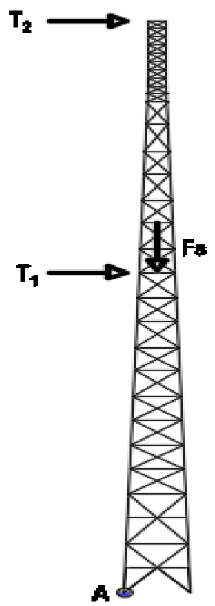


Figure 3.24.Overall actions on the tower with flexible plinth.

The stabilising force F_s is given by;

$F_s = 317.5KN + 4(25*0.4*0.4*1) + 4(25*1.5*1.5*0.3) = 401.0KN$

The stabilising moment M_s is calculated about A and is given by;

$M_s = F_s\ d_1$

d_1 is the lever arm of the force due the extreme base joint hence $d_1 = 2.5m$

$M_s = 1002.6KNm$

The overturning moment about A is given by;

$M_R = T_1 d_2 + T_2 d_3$

Where;

$d_2 = 35.1m$

$d_1 = 70m$

$M_R = 132.5*35.1 + 19.6*70 = 6022.8KNm$

Hence $M_R \geq M_s$ hence structural stability is not satisfied and we will change the foundation type.

3.6.6. Raft footing

The overturning moments is by far greater than the stabilising moment hence an appropriate raft foundation has to be designed in order to ensure structural stability. We decided to do a raft of section 7x7m of thickness 1.1m at a depth of 2.2m below ground level. On top of the raft is poured gravel with sand having a specific weight of 19.6KN/m^2 and 1m thickness. 4 concrete columns are also found on top of the raft of height 1m each. Above this a 10cm thick concrete slab is placed. In order to avoid the gravel with sand layer to escape on the lateral sides of the foundation, 20cm walls thick masonry walls are placed all around the 4 lateral sides of the foundation on top of the raft, these walls will play the role of retaining walls but no calculation needs to be done for its design because of the small lateral push on them by the gravel with sand layer. Anchor bolts of length 1.65m and 30mm diameter are placed on the column-base plate. Figure 3.25 presents the raft foundation and its components.

The raft is made of panels. The panels are designed as a plate thick element. The panels will be modelled in SAP2000 v22 as a plate resting on Winkler's springs. The stiffness of the springs is obtained from the modulus of subgrade reaction of the soil times the area of the meshing square using relation. The soil considered is moist clay soil and the modulus of subgrade reaction is obtained using the figure presented in Annex 2.

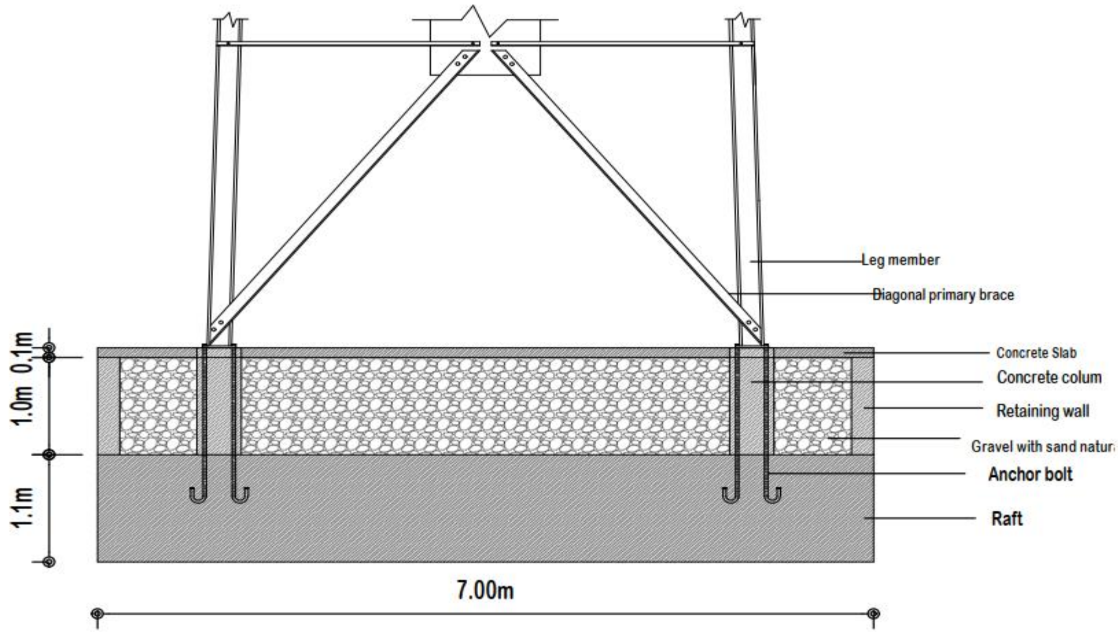


Figure 3.25.2D view of raft foundation on both X-Z and X-Y planes.

For the analysis, the tower is considered resting on raft foundation made of panels and the soil is idealized as springs as highlighted in the figure 3.26.

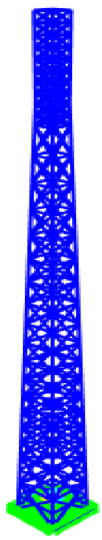


Figure 3.26. Tower resting on raft foundation.

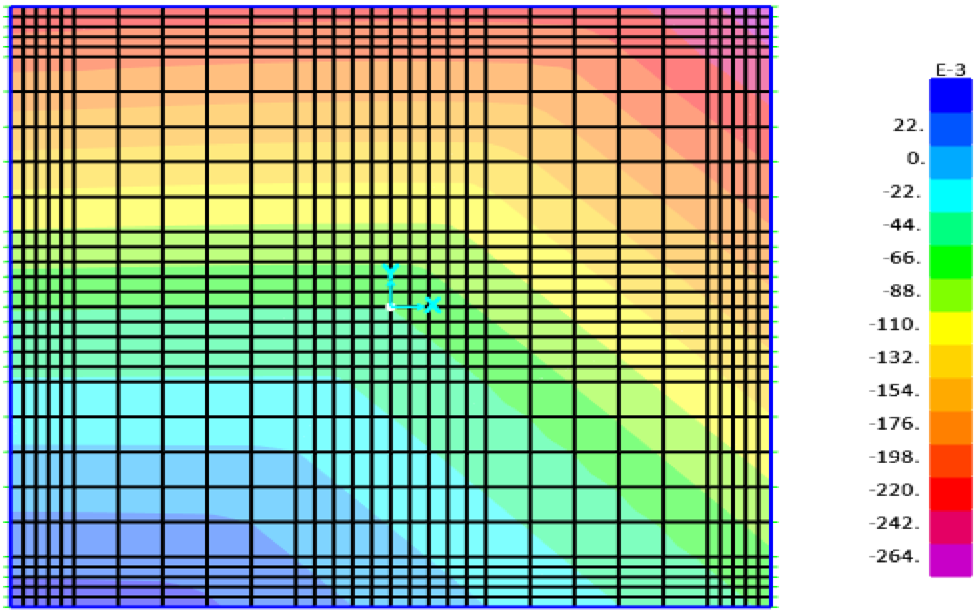


Figure 3.27. Soil pressure under raft footing in MPa

The maximum soil pressure is 0.28 Mpa< $\sigma_{adm} = 0.3\ MPa$. The admissible soil stress condition is satisfied.

The moment distribution on the raft along x and y are presented in the figure 3.28 and 3.29..

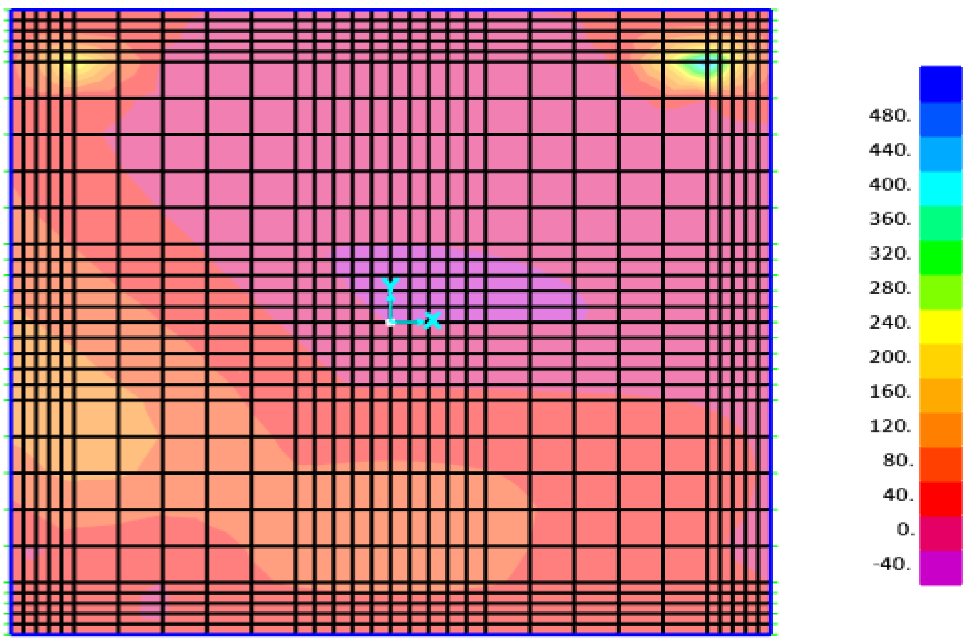


Figure 3.28. Moment distribution along x direction on the raft.

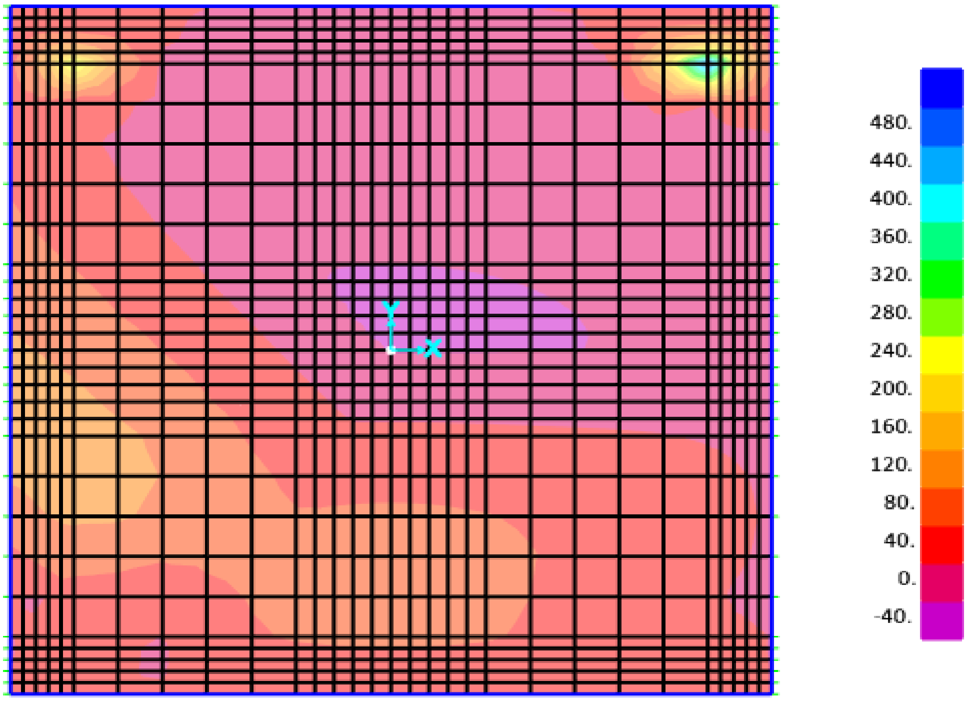


Figure 3.29. Moment distribution along y direction on the raft.

The designed moments are presented in table 3.31.

Table 3.31. Maximum moments in the raft

	X direction	Y direction	Units
Maximum positive moment	495.37	494.70	kN.m
Maximum negative moment	397.40	391.24	kN.m

The recapitulative of the flexural design is presented in Table 3.32 and 3.33.

The minimum longitudinal reinforcement $A_{s,min}$= 1575mm²/m

Table 3.32.Reinforcement at the bottom of the raft.

	Theoretical reinforcement (mm² /m)	Provided reinforcement per meter	Spacing (mm)
X direction	1575	8Φ16	142
Y direction	1575	8Φ16	142

.

Table 3.33.Reinforcement at the top of the raft.

	Theoretical reinforcement (mm² /m)	Provided reinforcement per meter	Spacing (mm)
X direction	1575	8Φ16	142
Y direction	1575	8Φ16	142

Figure 3.30 shows the structural detailing of the rat footing along both x and y direction due to symmetric size and equal longitudinal reinforcements.

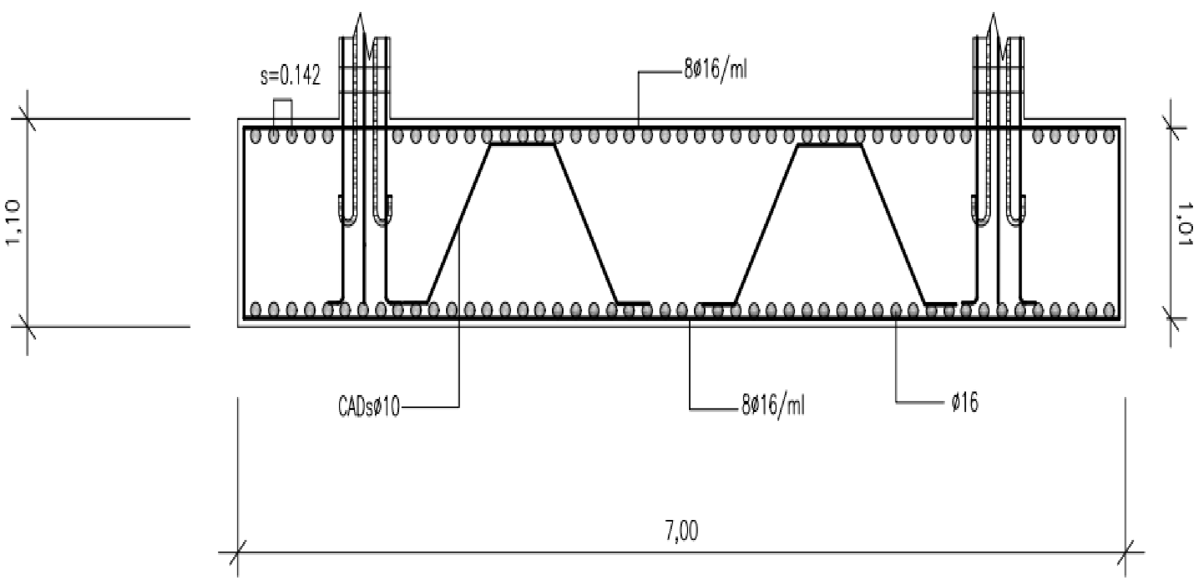


Figure 3.30. Raft detailing long both x and y directions.

- Structural stability verification

The stabilizing force is given by the sum of the self-weight of the tower and the self-weight of the foundation thus;

F_s = 317.5KN + 4(25*0.4*0.4*1) + (25*7*7*1.1) + (19.6*6.8*6.8*1) + 4(25*0.2*1*7) + (25*7*7*0.1) - 4(19.6*0.4*0.4*1) = 2837.3KN

The stabilising moment M_s is calculated about A and is given by;

$M_s = F_s d_1$

d_1 is the lever arm of the force due the extreme base joint hence d_1 = 3.5m

M_s = 9930.4KNm

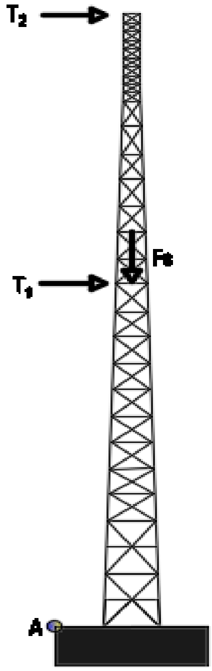


Figure 3.31. Overall action on tower with raft foundation.

The overturning moment about A is given by;

$M_R = T_1d_2 + T_2d_3$

Where;

$d_2 = 35.1m$

$d_1 = 70m$

$M_R = 132.5*35.1 + 19.6*70 = 6022.8KNm$

Hence $M_R \leq M_s$ hence structural stability is satisfied.

Conclusion

In conclusion, the results of the methodology of the previous chapter have been developed and applied to the study case which was a 70m tall telecommunication steel tower in Douala in Makepe headquarter. After a general presentation of the site, a presentation of the project was followed through its geometry and section and characteristics of the materials used. We modelled our structure on SAP2000 v22 and we have obtained the different solicitations acting on the steel elements. These solicitations helped us to design the elements manually under ULS conditions by determining the appropriate sections needed to overcome the loads acting upon them and verify if they satisfy the necessary conditions of instability. The type of connection used between the elements was bolting, the bolts were designed and verified using these solicitations and their characteristics determined. The maximum displacement on our structure was determined under SLS conditions and was found to be 32cm. In order to design the substructure, we started by designing the column base-plate followed by the anchorage lengths and thickness necessary to overcome the uplifting effect caused by wind actions at the base of our tower. We decided to use a flexible plinth as type of footing but we observed that it did not verify the conditions for the structural stability of tower due to its stabilizing moment far less than the overturning moment due to wind at the base of the tower. For this reason, we decided to use a raft footing which has a greater weight and with this footing, the structural stability of the tower was verified.

GENERAL CONCLUSION

The purpose of this study entitled the analysis of the wind effects on the tall steel reticular structures was to study the wind actions on a 70m high telecommunication tower in Douala, Cameroon. In order to achieve this objective, we reviewed the concept of steel, reticular structures, telecommunication tower and wind loads on structures in chapter 1, presented the methodology to be used in chapter 2 and the results presentation and interpretation was done in chapter 3. The structure was modelled using the software SAP 2000 v22 where all the elements were assigned as frame elements and moment was released on both ends of each element in the structure. The modelling was done in order to have only axial compressive and tensile loads. The wind loads on each panel on our structure were calculated manually and applied only on the nodes present in our model. The solicitations were obtained and were used to design steel elements and the connections between them. The foundations were also designed and the structural stability of our structure checked.

The results obtained from the analysis revealed that the solicitations due to the self-weight of the tower, the weight of antennas and rest levels are insignificant compared to the solicitations induced on the structure due to wind action which caused a maximum displacement of 32cm on the tower. We have also seen thar the wind action on the tower creates an uplifting effect at the base of the tower which tends to raise the tower if this action is not resisted using anchor bolts in the foundation. Besides, we saw that the resultant of the wind loads on the tower creates a moment at the base of the structure, this moment. This moment is obtained by multiplying the overall wind action on the tower with its lever arm which is the perpendicular distance from the base of the tower to the point of application of the resultant force. This moment is called overturning moment and is the most important factor to be considered when choosing and designing the foundation of the tower.

The work also brought forth certain perspectives to be considered for further studies: the investigation of the effect of different type of bracing system for the transfer of wind loads on our structure; study different methods to resist the overturning moment due to wind loads on our structure either by using a different type of foundation or by adding additional weight on the superstructure of our tower.

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ANNEXES

Annex 1.Joint number, height and displacements.

		Joint Displacements	
Joint	**Height**	**U1**	**U2**
Number	m	cm	cm
207	70.0	17.0	32.0
234	69.0	16.6	31.0
233	68.0	16.1	30.0
232	67.0	15.7	29.0
231	66.0	15.2	28.0
230	65.0	14.8	27.0
229	64.0	14.3	26.0
228	63.0	13.9	25.1
205	62.0	13.5	24.2
242	61.0	13.0	23.3
208	60.0	12.6	22.4
251	57.0	11.4	20.0
218	54.0	10.2	17.8
250	51.0	9.1	15.7
217	48.0	8.1	13.7
249	45.0	7.1	11.9
216	42.0	6.1	10.3
248	39.0	5.3	8.7
215	36.0	4.5	7.4
247	33.0	*3.8*	*6.1*
214	30.0	3.1	5.0
246	27.0	2.6	4.1
213	24.0	2.0	3.3
245	21.0	1.6	2.5
212	18.0	1.2	1.9
244	15.0	0.8	1.3
211	12.0	0.6	0.9
297	9.1	0.3	0.5
210	6.0	0.2	0.2
294	3.1	0.0	0.0
202	0	0	0

Annex 2. Characteristics of schifflerized angle sections.

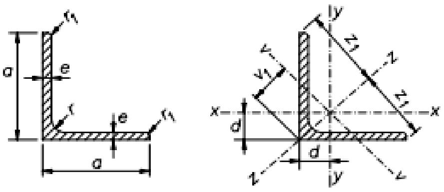

Tabella 6. Profilati a L ad ali uguali ed a spigoli arrotondati.

Designazione (• profilati non normali)	a	e	r	r_1	Area S	Massa lineica p	Posizione del centro di gravità			Asse xx = asse yy				Asse zz			Asse vv		
	mm	mm	mm	mm	cm²	kg/m	d cm	z_1 cm	v_1 cm	I_{xy} cm⁴	$I_x = I_y$ cm⁴	$W_x = W_y$ cm³	$i_x = i_y$ cm	I_z cm⁴	W_z cm³	i_z cm	I_v cm⁴	W_v cm³	i_v cm
L 20 × 3 UNI EU 56	20	3	4	2	1,13	0,88	0,60	1,41	0,84	0,23	0,39	0,28	0,59	0,61	0,43	0,74	0,16	0,19	0,38
» 20 × 4 •	20	4	4	2	1,46	1,14	0,63	1,41	0,90	0,28	0,49	0,36	0,58	0,77	0,54	0,72	0,21	0,23	0,38
» 25 × 3 UNI EU 56	25	3	4	2	1,43	1,12	0,72	1,77	1,02	0,47	0,80	0,45	0,75	1,26	0,71	0,94	0,33	0,33	0,48
» 25 × 4 »	25	4	4	2	1,86	1,46	0,76	1,77	1,07	0,59	1,01	0,58	0,74	1,60	0,90	0,93	0,43	0,40	0,48
» 25 × 5 •	25	5	4	2	2,27	1,78	0,80	1,77	1,13	0,68	1,20	0,71	0,73	1,89	1,07	0,91	0,52	0,46	0,48
» 30 × 3 UNI EU 56	30	3	5	2,5	1,74	1,36	0,84	2,12	1,18	0,82	1,40	0,65	0,90	2,22	1,05	1,13	0,58	0,49	0,58
» 30 × 4 »	30	4	5	2,5	2,27	1,78	0,88	2,12	1,24	1,05	1,80	0,85	0,89	2,85	1,34	1,12	0,75	0,61	0,58
» 30 × 5 •	30	5	5	2,5	2,78	2,18	0,92	2,12	1,30	1,25	2,16	1,04	0,88	3,41	1,61	1,11	0,92	0,71	0,57
» 35 × 3 •	35	3	5	2,5	2,04	1,60	0,96	2,47	1,36	1,34	2,29	0,90	1,06	3,63	1,47	1,34	0,95	0,70	0,68
» 35 × 3,5 UNI EU 56	35	3,5	3,5	1,75	2,35	1,85	0,98	2,47	1,39	–	2,63	1,04	1,06	4,03	–	1,33	1,18	0,85	0,67
» 35 × 4 •	35	4	5	2,5	2,67	2,09	1,00	2,47	1,42	1,73	2,95	1,18	1,05	4,68	1,89	1,33	1,23	0,86	0,68
» 35 × 5 UNI EU 56	35	5	5	2,5	3,28	2,57	1,04	2,47	1,48	2,07	3,56	1,45	1,04	5,64	2,28	1,31	1,49	1,01	0,67
» 40 × 3 »	40	3	5	3	2,35	1,84	1,07	2,83	1,52	2,01	3,45	1,18	1,21	5,45	1,93	1,52	1,44	0,95	0,78
» 40 × 4 »	40	4	5	3	3,08	2,42	1,12	2,83	1,58	2,61	4,47	1,55	1,21	7,09	2,51	1,52	1,86	1,17	0,78
» 40 × 5 »	40	5	5	3	3,79	2,97	1,16	2,83	1,64	3,17	5,43	1,91	1,20	8,60	3,04	1,51	2,26	1,37	0,77
» 40 × 6 •	40	6	5	3	4,48	3,52	1,20	2,83	1,70	3,66	6,31	2,26	1,19	9,98	3,53	1,49	2,65	1,56	0,77
» 45 × 3 •	45	3	7	3,5	2,66	2,09	1,18	3,18	1,67	2,85	4,93	1,49	1,36	7,78	2,44	1,71	2,07	1,24	0,88

(segue)

(seguito della tabella 6)

Designazione (• profilati non normali)	a	e	r	r_1	Area S	Massa lineica p	Posizione del centro di gravità			Asse xx = asse yy				Asse zz			Asse vv		
	mm	mm	mm	mm	cm²	kg/m	d cm	z_1 cm	v_1 cm	I_{xy} cm⁴	$I_x = I_y$ cm⁴	$W_x = W_y$ cm³	$i_x = i_y$ cm	I_z cm⁴	W_z cm³	i_z cm	I_v cm⁴	W_v cm³	i_v cm
L 45 × 4 UNI EU 56	45	4	7	3,5	3,49	2,74	1,23	3,18	1,75	3,75	6,43	1,97	1,36	10,2	3,20	1,71	2,67	1,53	0,88
» 45 × 5 »	45	5	7	3,5	4,30	3,38	1,28	3,18	1,81	4,58	7,84	2,43	1,35	12,4	3,90	1,70	3,26	1,80	0,87
» 45 × 6 •	45	6	7	3,5	5,09	4,00	1,32	3,18	1,87	5,33	9,16	2,88	1,34	14,5	4,56	1,69	3,82	2,05	0,87
» 50 × 4 UNI EU 56	50	4	7	3,5	3,89	3,06	1,36	3,54	1,92	5,24	8,97	2,46	1,52	14,2	4,02	1,91	3,72	1,94	0,98
» 50 × 5 »	50	5	7	3,5	4,80	3,77	1,40	3,54	1,99	6,42	11,0	3,05	1,51	17,4	4,92	1,90	4,54	2,29	0,97
» 50 × 6 »	50	6	7	3,5	5,69	4,47	1,45	3,54	2,04	7,50	12,8	3,61	1,50	20,3	5,75	1,89	5,33	2,61	0,97
» 50 × 7 •	50	7	7	3,5	6,56	5,15	1,49	3,54	2,10	8,50	14,6	4,16	1,49	23,1	6,54	1,88	6,11	2,91	0 96
» 50 × 8 •	50	8	7	3,5	7,41	5,82	1,52	3,54	2,16	9,41	16,3	4,68	1,48	25,7	7,27	1,86	6,87	3,19	0,96
» 55 × 4 •	55	4	8	4	4,31	3,38	1,47	3,89	2,08	6,99	12,0	2,98	1,67	19,0	4,88	2,18	5,01	2,41	1,08
» 55 × 5 •	55	5	8	4	5,32	4,18	1,52	3,89	2,15	8,60	14,7	3,70	1,66	23,3	6,00	2,09	6,11	2,84	1,07
» 60 × 4 •	60	4	8	4	4,71	3,70	1,60	4,24	2,26	9,20	15,8	3,58	1,83	25,0	5,89	2,30	6,58	2,91	1,18
» 60 × 5 UNI EU 56	60	5	8	4	5,82	4,57	1,64	4,24	2,32	11,3	19,4	4,45	1,82	30,7	7,24	2,30	8,02	3,45	1,17
» 60 × 6 »	60	6	8	4	6,91	5,42	1,69	4,24	2,39	13,4	22,8	5,29	1,82	36,2	8,52	2,29	9,43	3,95	1,17
» 60 × 8 »	60	8	8	4	9,03	7,09	1,77	4,24	2,50	17,0	29,2	6,89	1,80	46,2	10,9	2,26	12,2	4,86	1,16
» 60 × 10 •	60	10	8	4	11,1	8,69	1,85	4,24	2,61	20,1	34,9	8,41	1,78	55,1	13,0	2,23	14,8	5,67	1,16
» 65 × 5 •	65	5	9	4,5	6,34	4,97	1,76	4,60	2,49	14,5	24,7	5,22	1,98	39,2	85,3	2,49	10,3	4,14	1,27
» 65 × 6 •	65	6	9	4,5	7,53	5,91	1,80	4,60	2,55	17,1	29,2	6,21	1,97	46,3	10,1	2,48	12,1	4,74	1,27
» 70 × 5 •	70	5	9	4,5	6,84	5,37	1,88	4,95	2,66	18,3	31,2	6,10	2,14	49,5	10,0	2,69	13,0	4,87	1,38
» 70 × 6 UNI EU 56	70	6	9	4,5	8,13	6,38	1,93	4,95	2,73	21,6	36,9	7,27	2,13	58,5	11,8	2,68	15,3	5,59	1,37
» 70 × 7 »	70	7	9	4,5	9,40	7,38	1,97	4,95	2,79	24,8	42,3	8,41	2,12	67,1	13,6	2,67	17,5	6,27	1,36
» 70 × 8 •	70	8	9	4,5	10,6	8,36	2,01	4,95	2,85	27,8	47,5	9,52	2,11	75,3	15,2	2,66	19,7	6,91	1,36
» 70 × 10 •	70	10	9	4,5	13,1	10,3	2,09	4,95	2,96	33,3	57,2	11,7	2,09	90,5	18,3	2,63	23,9	8,10	1,35
» 75 × 5 •	75	5	10	5	7,36	5,78	1,99	5,30	2,82	22,5	38,5	7,00	2,29	61,0	11,5	2,88	16,1	5,69	1,48
» 75 × 6 •	75	6	10	5	8,75	6,87	2,04	5,30	2,89	26,7	45,6	8,35	2,28	72,2	13,4	2,87	18,9	6,54	1,47
» 75 × 7 •	75	7	10	5	10,1	7,94	2,09	5,30	2,95	30,7	52,4	9,67	2,27	83,0	15,7	2,86	21,7	7,34	1,46
» 80 × 6 UNI EU 56	80	6	10	5	9,35	7,34	2,17	5,66	3,07	32,7	55,8	9,57	2,44	88,5	15,6	3,0.8	23,1	7,55	1,57
» 80 × 7 •	80	7	10	5	10,8	8,49	2,21	5,66	3,13	37,6	64,2	11,1	2,44	102	18,0	3,07	26,5	8,48	1,57
» 80 × 8 UNI EU 56	80	8	10	5	12,3	9,63	2,26	5,66	3,19	42,4	72,2	12,6	2,43	115	20,3	3,06	29,9	9,36	1,56
» 80 × 10 »	80	10	10	5	15,1	11,9	2,34	5,66	3,30	51,1	87,5	15,4	2,41	139	24,5	3,03	36,4	11,0	1,55
» 80 × 12 •	80	12	10	5	17,9	14,0	2,41	5,66	3,41	59,0	102	18,2	2,39	161	28,4	3,00	42,7	12,5	1,55

(segue)

(seguito della tabella 6)

Designazione (• profilati non normali)	a	e	r	r_1	Area S	Massa lineica p	Posizione del centro di gravità			Asse xx = asse yy				Asse zz			Asse vv		
	mm	mm	mm	mm	cm^2	kg/m	d cm	z_1 cm	v_1 cm	I_{xy} cm^4	$I_x = I_y$ cm^4	$W_x = W_y$ cm^3	$i_x = i_y$ cm	I_z cm^4	W_z cm^3	i_z cm	I_v cm^4	W_v cm^3	i_v cm
L 90 × 6 •	90	6	11	5,5	10,6	8,30	2,41	6,36	3,40	47,0	80,3	12,2	2,76	127	20,0	3,47	33,3	9,8	1,78
» 90 × 7 UNI EU 56	90	7	11	5,5	12,2	9,61	2,45	6,36	3,47	54,3	92,5	14,1	2,75	147	23,1	3,46	38,3	11,0	1,77
» 90 × 8 •	90	8	11	5,5	13,9	10,9	2,50	6,36	3,53	61,2	104	16,1	2,74	166	26,0	3,45	43,1	12,2	1,76
» 90 × 9 UNI EU 56	90	9	11	5,5	15,5	12,2	2,54	6,36	3,59	68,0	116	17,9	2,73	184	28,9	3,44	47,8	13,3	1,76
» 90 × 10 •	90	10	11	5,5	17,1	13,4	2,58	6,36	3,65	74,4	127	19,8	2,72	201	31,6	3,43	52,5	14,4	1,75
» 90 × 12 •	90	12	11	5,5	20,3	15,9	2,66	6,36	3,76	86,3	148	23,3	2,70	234	36,8	3,40	61,7	16,4	1,74
» 100 × 6 •	100	6	12	6	11,2	9,26	2,64	7,07	3,74	64,9	111	15,1	3,07	176	24,9	3,86	46,2	12,4	1,98
» 100 × 7 •	100	7	12	6	13,7	10,7	2,69	7,07	3,81	75,1	128	17,5	3,06	203	28,8	3,86	53,1	14,0	1,97
» 100 × 8 UNI EU 56	100	8	12	6	15,5	12,2	2,74	7,07	3,87	85,0	145	19,9	3,06	230	32,5	3,85	59,8	15,5	1,96
» 100 × 10 »	100	10	12	6	19,2	15,0	2,82	7,07	3,99	104	177	24,6	3,04	280	39,6	3,83	72,9	18,3	1,95
» 100 × 12 »	100	12	12	6	22,7	17,8	2,90	7,07	4,11	121	207	29,1	3,02	328	46,3	3,80	85,7	20,9	1,94
» 100 × 15 •	100	15	12	6	27,9	21,9	3,02	7,07	4,27	144	249	35,6	2,98	393	55,5	3,75	104	24,4	1,93
» 110 × 6 •	110	6	12	6	13,0	10,2	2,89	7,78	4,09	87,4	149	18,4	3,39	237	30,5	4,27	62,1	15,2	2,19
» 110 × 7 •	110	7	12	6	15,1	11,8	2,94	7,78	4,16	101	173	21,4	3,39	274	35,2	4,26	71,4	17,2	2,18
» 110 × 8 •	110	8	12	6	17,1	13,4	2,99	7,78	4,22	115	195	24,4	3,38	310	39,9	4,26	80,5	19,1	2,17
» 110 × 9 •	110	9	12	6	19,1	15,0	3,03	7,78	4,28	128	217	27,3	3,37	345	44,4	4,25	89,5	20,9	2,16
» 110 × 10 •	110	10	12	6	21,2	16,6	3,07	7,78	4,34	140	239	30,1	3,36	379	48,7	4,23	98,4	22,6	2,16
» 120 × 8 •	120	8	13	6,5	18,7	14,7	3,23	8,49	4,56	150	255	29,1	3,69	405	47,8	4,65	105	23,1	2,37
» 120 × 9 •	120	9	13	6,5	21,0	16,5	3,27	8,49	4,62	167	285	32,6	3,68	452	53,2	4,64	117	25,4	2,36
» 120 × 10 UNI EU 56	120	10	13	6,5	23,2	18,2	3,31	8,49	4,69	184	313	36,0	3,67	497	58,6	4,63	129	27,5	2,36
» 120 × 12 »	120	12	13	6,5	27,5	21,6	3,40	8,49	4,80	216	368	42,7	3,65	584	68,8	4,60	152	31,5	2,35
» 120 × 15 •	120	15	13	6,5	33,9	26,6	3,51	8,49	4,97	260	445	52,4	3,62	705	83,1	4,56	185	37,1	2,33
» 150 × 12 UNI EU 56	150	12	16	8	34,8	27,3	4,12	10,6	5,83	433	737	67,7	4,60	1170	110	5,80	303	52,0	2,95
» 150 × 15 »	150	15	16	8	43,0	33,8	4,25	10,6	6,01	528	898	83,5	4,57	1430	134	5,76	370	61,6	2,93
» 150 × 18 •	150	18	16	8	51,0	40,1	4,37	10,6	6,17	615	1050	98,7	4,54	1670	157	5,71	435	70,4	2,92
» 180 × 15 •	180	15	18	9	52,1	40,9	4,98	12,7	7,05	936	1590	122	5,52	2520	198	6,96	653	92,6	3,54
» 180 × 18 UNI EU 56	180	18	18	9	61,9	48,6	5,10	12,7	7,22	1097	1870	145	5,49	2960	233	6,92	768	106	3,52
» 180 × 20 •	180	20	18	9	68,3	53,7	5,18	12,7	7,33	1199	2040	159	5,47	3240	255	6,89	843	115	3,51
» 200 × 16 •	200	16	18	9	61,8	48,5	5,52	14,1	7,81	1381	2340	162	6,16	3720	263	7,76	960	123	3,94
» 200 × 18 UNI EU 56	200	18	18	9	69,1	54,2	5,60	14,1	7,93	1533	2600	181	6,13	4130	292	7,73	1070	135	3,93
» 200 × 20 »	200	20	18	9	76,3	59,9	5,68	14,1	8,04	1679	2850	199	6,11	4530	320	7,70	1170	146	3,92
» 200 × 24 •	200	24	18	9	90,6	71,1	5,84	14,1	8,26	1953	3330	235	6,06	5280	374	7,64	1380	167	3,90

Annex 3. Subgrade modulus for different kind of soils.

Nature du sol	*C* (t/m^3)
1 terrain légèrement tourbeux et marécageux	500- 1 000
2 terrain essentiellement tourbeux et marécageux	1 000- 1 500
3 sable fin	1 000- 1 500
4 remblais d'humus, sable et gravier	1 000- 2 000
5 sol argileux détrempé	2 000- 3 000
6 sol argileux humide	4 000- 5 000
7 sol argileux sec	6 000- 8 000
8 sol argileux très sec	10 000
9 terrain compacté contenant de l'humus du sable et peu de pierres	8 000-10 000
10 même nature que ci-dessus avec beaucoup de pierres	10 000-12 000
11 gravier fin et beaucoup de sable fin	8 000-10 000
12 gravier moyen et sable fin	10 000-12 000
13 gravier moyen et sable grossier	12 000-15 000
14 gros gravier et sable grossier	15 000-20 000
15 gros gravier et peu de sable	15 000-20 000
16 gros gravier et peu de sable mais très compacté	20 000-25 000

Annex 4. Recommended values for φ factors for buildings

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991-1-1)			
Category A : domestic, residential areas	0,7	0,5	0,3
Category B : office areas	0,7	0,5	0,3
Category C : congregation areas	0,7	0,7	0,6
Category D : shopping areas	0,7	0,7	0,6
Category E : storage areas	1,0	0,9	0,8
Category F : traffic area, vehicle weight ≤ 30kN	0,7	0,7	0,6
Category G : traffic area, 30kN < vehicle weight ≤ 160kN	0,7	0,5	0,3
Category H : roofs	0	0	0
Snow loads on buildings (see EN 1991-1-3)*			
Finland, Iceland, Norway, Sweden	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l.	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H ≤ 1000 m a.s.l.	0,50	0,20	0
Wind loads on buildings (see EN 1991-1-4)	0,6	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0
NOTE The ψ values may be set by the National annex. * For countries not mentioned below, see relevant local conditions.			