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STRUCTURAL DESIGN OF A LIGHT STEEL FRAME HOUSE

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Aos meus Pais

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Resumo

Nos últimos anos surgiram novos sistemas de construção que garantem alto desempenho estrutural e ambiental. Entre outros, a estrutura de aço leve (LSF) está se tornando um sistema estrutural eficaz para edifícios de baixo e médio porte. Várias vantagens, como alta relação resistência / peso, baixo custo de transporte e facilidade de construção, agilizam o uso de elementos de aço enformado a frio em muitos países quer como elementos estruturais quer não estruturais.

Além de oferecer excelentes propriedades estruturais, térmicas e acústicas, este sistema apresenta ainda uma solução sustentável, uma vez que a estrutura de aço é 100% reciclável e, ao mesmo tempo, permite economias significativas em termos de energia. Esta dissertação concentra-se nas propriedades das construções LSF, passando pelas várias características inerentes a este método, como as propriedades dos materiais e as principais seções transversais utilizadas, além de apresentar alguns revestimentos disponíveis no mercado.

Posteriormente, é apresentado um estado de dimensionamento de acordo com as práticas europeias de projeto de uma estrutura de dois andares, feita de aço leve, adotando paredes resistentes ao corte com revestimento de madeira OSB como sistema de resistência lateral à carga.

Palavras-chave: LSF, estruturas de aço leve, aço enformado a frio, dimensionamento de estruturas, Eurocódigos, sustentabilidade.

Abstract

In recent years, new innovative systems to ensure high structural and environmental performance have emerged. Among others, light steel framing (LSF) is becoming an effective structural system for low- and mid-rise buildings. Several desirable features, like high strength-to-weight ratio, low shipping cost and easiness of construction, expedite the use of CFS members in many countries as both structural and nonstructural members.

While providing excellent structural, thermal and acoustic properties, this method also offers a sustainable solution, since the steel frame is 100% recyclable and at the same time, enables significant savings in terms of energy. This dissertation focusses on the properties of LSF constructions, going through the various characteristics inherent to this method like the properties of materials and main cross-sections used, while also presenting some claddings available in the market.

Subsequently, analysis and design according the European design practices of a 2-storey structure made of light steel frame adopting OSB wood-sheathed shear walls as lateral load resisting system, are carried out.

Key words: Light steel frame, structural design, Eurocodes, sustainability.

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Table of acronyms

CEN – Comité Européen de Normalisation

CLT – Cross Laminated Timber

EC0 – Eurocode 0

EC1 – Eurocode 1

EC3 – Eurocode 3

EC5 – Eurocode 5

EC8 – Eurocode 8

LSF – Light Steel Frame

HVAC – Heating, Ventilation and Air Conditioning

HSHW – Health Safety and Hygiene at Work

OSB – Oriented Strand Boards

USA – United States of America

1

Introduction

1.1. Contextualization

Having a home or even just a roof is not only a desire, but also a concern for many people around the world. Faced with this need, the construction industry has sought to adopt innovative technologies and construction methods that allow the reduction of costs, time and simultaneously guarantee the quality required by customers.

Currently, the execution of the structure of buildings is based on the use of reinforced concrete, stone masonry, steel profiles or even wood.

One solution that has been growing in recent years is the light steel construction, commonly known as LSF - Light Steel Frame, due to its constructive system that allows the construction of buildings that present, in terms of safety and comfort, a high quality homes using the latest precision engineering technology.

Instead of a more traditional approach, which consists of the use of concrete and brick, the structure of a house will consist of resistant walls composed of several metal profiles in galvanized steel of small thickness and covered by several types of sheathings.

It should be noted at the outset that the key word in this concept is *Light*, since it is its main characteristic and from here advent all its advantages. Because of this lightness, each element is easy to transport, handle, and assemble, reducing the time of construction, and the equipment needed. Furthermore, since steel is mostly priced by the weight, light elements will be cheaper.

Due to the fact that these metal profiles have a cross-sectional thickness between 0.9 and 3.2 mm, combining the material high strength and a considerable lightness, a better structural performance is obtained [1].

On top of being used worldwide for new construction, the low weight of steel and other materials used, make the LSF constructive method ideal for remodeling old buildings. The use of lightweight materials reduces transportation and lifting difficulties while at the same time eliminating the need to reinforce the old building structure, since it the increased load won't be relevant. In some cases, this advantage makes LSF the only possible alternative for splitting spaces, adding a new floor or replacing wood floors or already degraded roofs.

Another great advantage of this constructive method is the prefabrication by modules, and the walls can be assembled in a factory then exported and assembled in a short time in the desired place. An example of success is the 650 m2 dwelling shown in Figure 1-1, prefabricated in China and assembled in Melbourne (Australia), and it took only sixty days for the fabrication and three weeks for the assembly, for a total of one hundred days of construction [1]. Note that, in traditional construction this would have needed about one year to be built.

Figure 1-1 - LSF prefabricated dwelling [1]

On the other hand, sustainable development is a key challenge in our society and light steel construction offers the guarantee of not harming the environment. In addition to being 100% recyclable, the use of steel in a building reduces drastically the usual debris. Finally, it also allows significant energy savings, both in its construction and in its use [2].

For all these advantages, and because the author considers the LSF as the future of building construction, this dissertation was elaborated.

1.2. Scope and objectives

The main purpose of the dissertation is to acquire knowledge about a constructive method in clear expansion, identifying its current uses and realizing its potentialities and limitations.

In this way, two different branches are identified, which will be dealt with herein, the design features and the normative regulations of light steel structures.

On the side of the conception of LSF, to understand how the construction process works, what are the dimensions and shapes of the metallic profiles, what are the thermal and acoustic insulation, and the coating materials, and how the different elements are assembled.

On the structural design side, what are the specific characteristics of light-gauge steel, how important are the different structural elements, and what requirements must be met in the applicable standards.

1.3. Dissertation layout

This dissertation is divided into 8 chapters. As the title seems to suggest, “Structural Design of an Light Steel Frame Dwelling”, this manuscript implies the knowledge about the behavior, constructive method and design implications of light steel framing buildings, as well as the main properties of light steel sections, in order to be applied to a real case study.

The second chapter is a brief introduction and contextualization to light steel framing structures, how this constructive method has appeared and how it has been developed giving a small historic backward. As well, the main advantages and disadvantages of this system are highlighted.

In the third chapter, an intensive presentation is made about all the materials involved in the structural behavior of an LSF structure. All the properties used in the framing system, including the mechanical and chemical properties of the steel, the coating properties, the most common cross-section shapes as well as their inherent properties and how they were obtained.

In addition, some of the possible claddings are presented, since as it will be explained more ahead, they are part of the structure. The main advantages and disadvantages of each one are explained so that it would be possible to know what innovative solutions exist in the market nowadays.

Subsequently, the possible connections to assemble all the structural elements together are presented, describing how they should be used and how the most common cases to each one should be applied.

The fourth chapter is about the constructive method, where it is introduced some specific terminology of this kind of construction, and it is explained in detail how this type of construction is made.

The fifth chapter is about the reglementary norms, where all the ultimate and serviceability requirements are identified and explained, as well as some phenomena related to light-gauge steel members.

Chapter six is about the case study, where a brief presentation of the building, location and specifying some of its main characteristics, is made. Additionally, the applied loads are identified and calculated according to the European standard (Eurocode 1) [3].

In the seventh chapter, the obtained results are explained and analyzed according to Eurocode (EC3) [4] and it is presented the final designs.

Lastly, in chapter eight, a reflection is made about the results obtained and some suggestions about future works.

2
Historic contextualization

To define the historical background of the LSF we must go back to the United States in the nineteenth century. In those years, the population of the country multiplied by ten being necessary to resort to the materials available locally and practical and fast methods that allowed to increase the productivity in the construction of new houses. This type of constructive system with wood structure is usually called wood framing. Wood was then used as the main structural element of residential buildings and thus remained until today.

By the end of World War II, steel was an abundant resource and metallurgical companies had gained considerable experience in using metal because of the war effort. First used in the partitions of large buildings with iron structure, light steel shaped or cold formed was used in partitions of dwelling buildings and believed that it could totally replace the wooden structures in the houses.

Applications of cold-formed steel are found as early as the year 1850, both in the US and in Britain. However, the US legislation applicable to the LSF only began to take its first steps in the 1940s. Another major push came in the 1980s when several forests were closed to the timber industry. This has led to the decline in the quality of wood used in construction and to large fluctuations in the price of this raw material. In 1991, the price of wood used in construction rose significantly in a few months, which led many manufacturers to switch to steel immediately.

According to the Steel Recycling Institute, to build a wooden dwelling with about 186 m2 would be needed between 40 to 50 trees, however, building that same steel housing would require only 6 scrapped cars. [5]

After this explosive but unstructured start, associations of technicians and builders were created and the LSF started to be seen professionally. Today it is incomparably easier to work light steel than fifteen years ago. The system is in full development and is widely used in the construction of buildings in the most developed countries, such as the USA, Japan, Australia New Zealand, United Kingdom, Northern Europe and South Africa. The Figure 2-1 shows the percentage of steel habitations in these different countries.

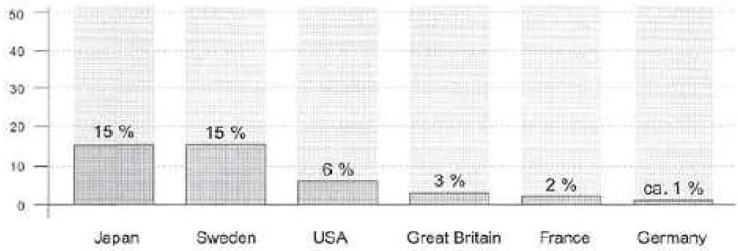


Figure 2-1 - Percentage of the use of steel structures in habitation buildings[21]

In Portugal, although with less impact than in the countries listed above, there was also the same tendency to seek alternatives to traditional construction processes. Since the initial faltering of the LSF in Portugal around 1993, the demand for houses with steel structure has been increasing, so as to find quality solutions at competitive prices. [6]

3

Materials

The correct choice of materials can contribute a lot to the value of a property, not only in terms of finishes, but mainly nonvisible elements such as structural elements as well as thermal or acoustic insulation. In the design of buildings in LSF this selection has an even greater relevance since it is necessary to consider the nature of the materials, considering the compatibility between them, and with the different specialties. Therefore, it is extremely important that there is good communication between the different specialties' technicians and the structural designer, which must master many aspects of civil construction.

Finally, it should be emphasized that beyond the optimization of the choice of materials, it is also very important that the manpower has enough knowledge about this method so that the final product has the best quality possible.

3.1. Steel

By far the largest producer and exporter of steel in the world is China [7]. Here the raw materials of new or recycled mineral are transformed into galvanized steel sheets of thicknesses that can vary between 0,9mm and 3,2mm and with necessary widths to form the profiles and length of several hundred meters rolled in reels.

This steel reels have the following characteristics:

Yield strain (f_y) in the range of 220-460 N / mm^2 (however, it is now possible to use high strength steels - up to 600 N / mm^2),

Ultimate strain (f_u) in the range 300-720 N / mm^2;

The relation between the ultimate strain and the yield strain (f_u / f_y) with values in the range of 1.1-1.9;

The maximum extension in the range 14-25%

However, these values are not the same after bending, since the cold folding operation increases the yield stress of the steel (f_y) to a modified yield stress (f_{ya}), also raising its ultimate strength (f_u) and reducing its ductility.

The value of f_{ya} depends very much on several parameters such as the kind of steel, the kind of stress (compression or tension), relation between f_u / f_y, the relationship between the bend radius and the thickness (r_i / t), the amount of cold work performed, and is estimated in different ways in several standards.

As the main zones affected by the cold forming process are the corners, these will have mechanical properties different from the flat zones. Figure 3-1 illustrates the variation of the mechanical properties along the cross-section of a U-profile.

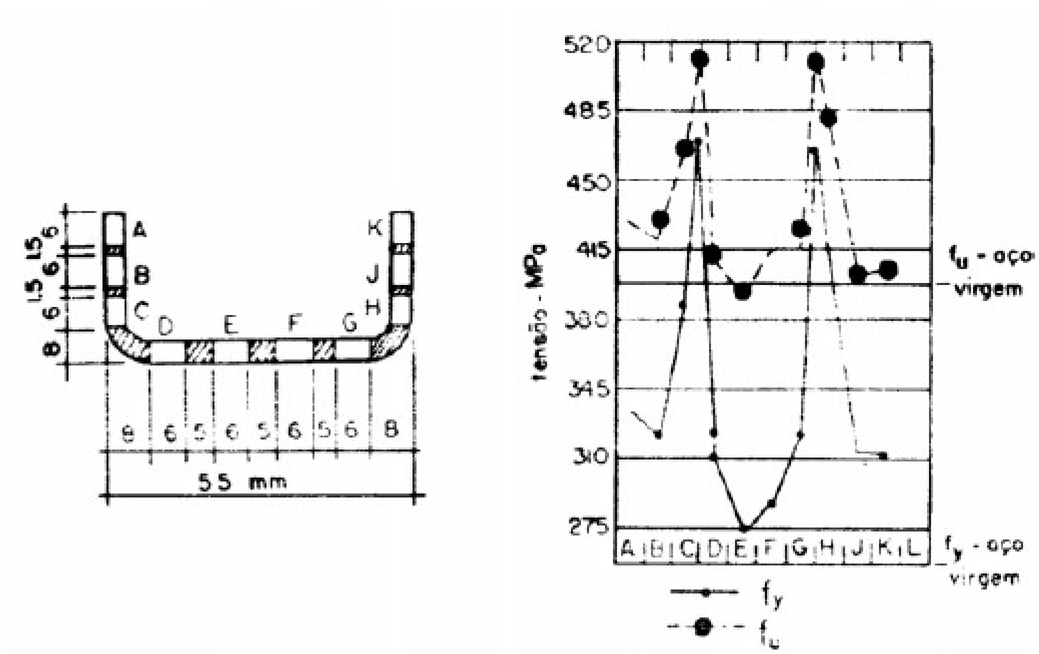


Figure 3-1 - Effect of cold forming in the mechanic properties of a U-profile [6]

In this particular case, the corner region has an increase of 28% for f_u and 70% for f_y, and hence the mean yield strength of the section f_{ya} can in certain cases be used in substitution of f_y. Even if the designer chooses the more conservative option and does not use f_{ya}, he should be aware of this increase of the yield stress. [8]

3.1.1. Protection

Galvanizing is the treatment that not only protects steel from rust and corrosion, but also gives it a bright and attractive appearance is guaranteed through a molten zinc bath ensuring a surface coating with a density that can range from 30g / mm2 to 600g / mm2.

Thanks to this type of protection, it is possible for the steel to remain in service for about 40 years without maintenance and can be freely folded, painted and welded. However, it has the disadvantage that it cannot be used under the ground without being properly covered. [9] Also, if weld is applied, a paint of cold galvanization spray should be applied, since the weld destroys this coating.

3.1.2. Fabrication process

The coiled steel in reels is then exported to the manufacturers of profiles which can mold them through two different methods. Bending and rolling.

Press braking is mainly used to manufacture elements with simpler cross-sections (U, Z or L) and is more advantageous for small-scale production and whose lengths do not exceed 6 meters.

This consists of using a press consisting of an upper part with a convex shape (in V or U) that compresses the plate against a lower surface in the inverse (concave) shape. Since it is a less industrialized process that requires at least two operators, the production speed does not exceed 60 meters per minute. [10]

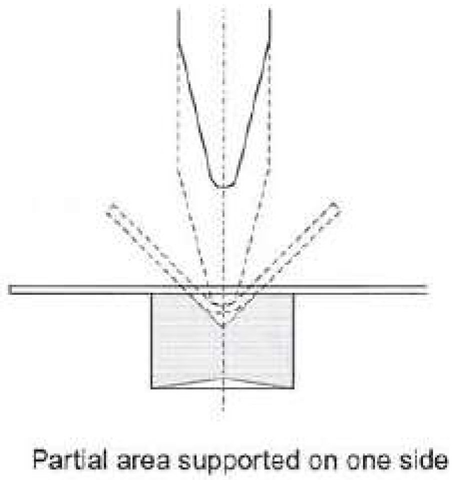


Figure 3-2 - Press braking process [21]

Rolling is the process most used in the manufacture of cold formed sections because it is almost 100% automatic, making it more advantageous in the mass production of batches of profiles with the same section.

The process consists in passing the steel strip through a machine composed of a series of sequentially placed compressor rollers (6 to 15) and with different spacing and geometries, which fold the plate as it is drawn, being practicable an average production of 30 meters per minute. At the end of the line the pieces are being cut to the desired length up to a maximum of 12 meters.

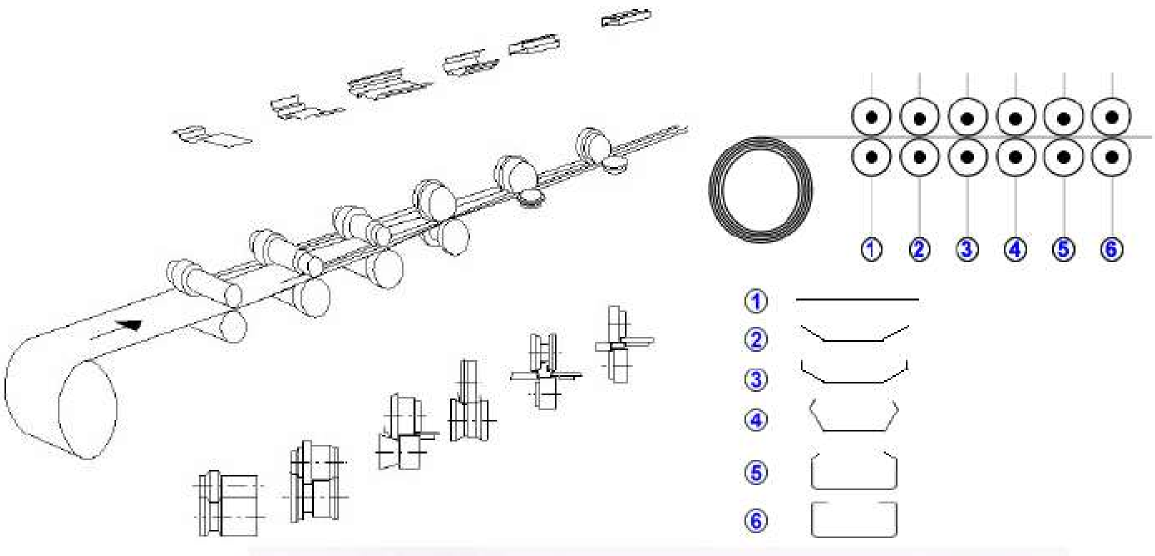


Figure 3-3 - Rolling process [8]

In the same process it is still possible to fabricate closed section profiles using a different machine which after the folding steps still applies a weld seam.

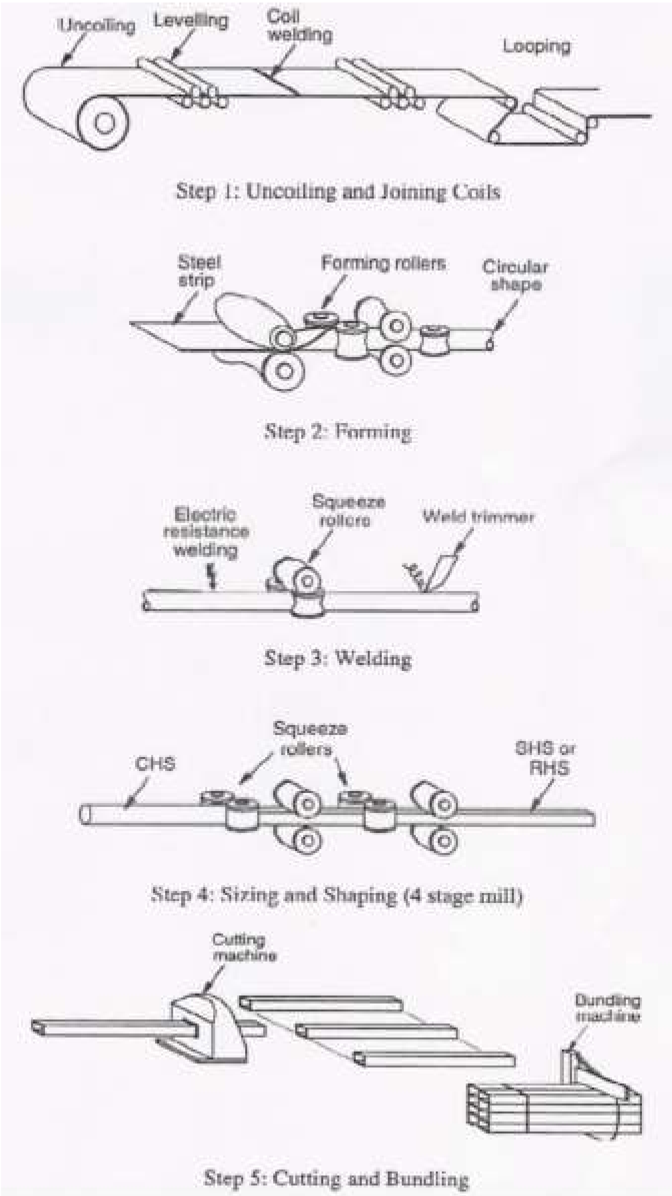


Figure 3-4 - Cold forming a closed cross-section process [8]

3.1.3. Common cross-sections

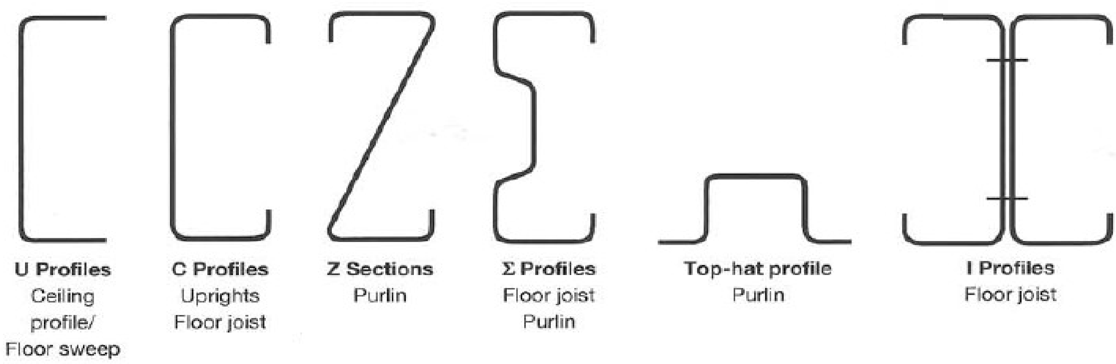


Figure 3-5 - Most common cross-sections shapes [21]

The most popular cross-sections are the U-sections and C-sections. Note that the sections C are also referred to as U-sections with stiffeners. These two sections are the main constituents of the load bearing walls, where U-sections are designated as the tracks where the C-section studs are fixed. These C profiles are also frequently used for the construction of beams, whether front-to-front or back-to-back. (Figure 3-6)

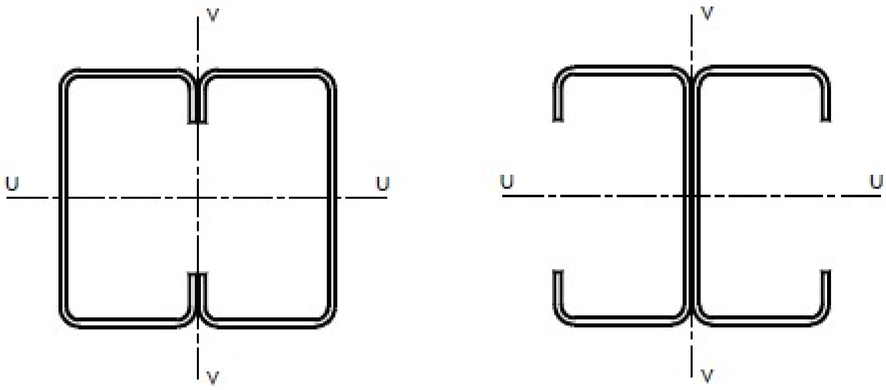


Figure 3-6 - Front-to-front and back-to-back built-up cross-sections [18]

On the other hand, more irregular sections, with more reinforced zones, can be used. This have increased cross-sectional strength, as illustrated in the graph of Figure 3-7. This is the case of the sections in Σ and Ω that by their more irregular geometry can be advantageous in certain cases, being that they are usually more used in the execution of floor beams and ceiling mounts since its more easy to choose the desired height of the web and hence the height of the slab.

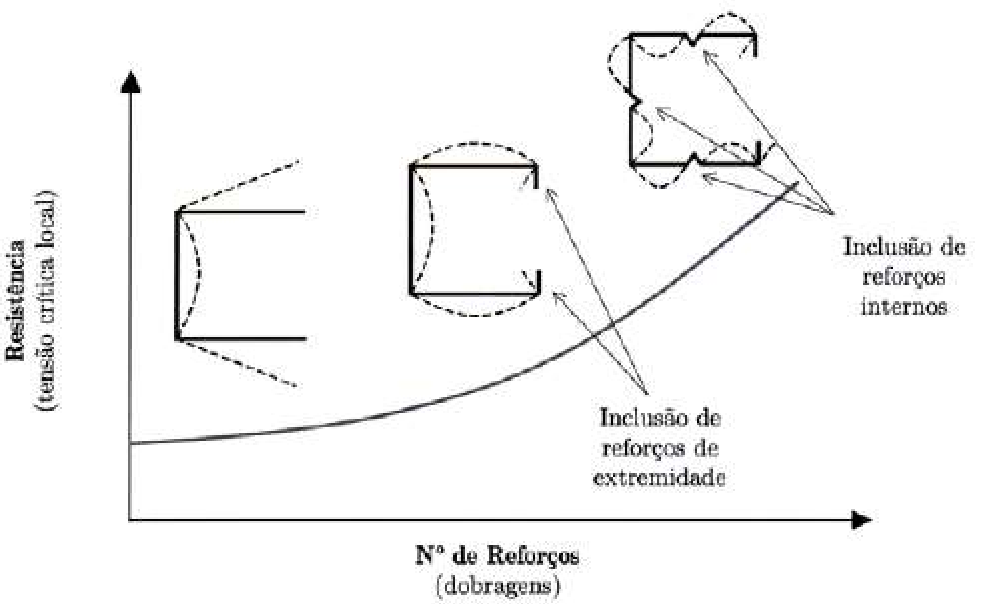


Figure 3-7 - Evolution of the critical tension with the increase of stiffeners [22]

3.2. Insulation and claddings

3.2.1. Facade claddings

As previously mentioned, it is necessary to choose carefully the claddings in the execution of light steel projects. It is in facade claddings that this aspect gains special prominence, since these can be mere claddings of the structural elements or claddings that really contributes to the structural performance of the building.

3.2.1.1. Structural claddings

Structural claddings are usually boards of a material with physical and geometric characteristics that provide them with an ability to withstand expected loads, acting as braces and providing rigidity to the building.

In view of the importance of these claddings, they must be packed, handled and applied with special care. Since they are an integral part of the structure, they are subject to design and must comply with some norms, such as NP EN 300 which establishes the minimum requirements for OSB structural boards. (**Erro! A origem da referência não foi encontrada.**)

Table 3-1 - Requirements specified in NP EN 300 to consider a OSB board as structural element [6]

NP EN 300		
Thickness	11 – 17 mm	18 – 25 mm
Service classes	3	
Resistance		
Longitudinal bending	20 N/mm^2	18 N/mm^2
Transverse bending	10 N/mm^2	9 N/mm^2
Tension perpendicular to the board surfaces	0,32 N/mm^2	0,30 N/mm^2
Young Modulus		
Longitudinal	3500 N/mm^2	
Transverse	1400 N/mm^2	
Maximum swelling percentage 24h	≤ 15 %	

Usually this is only used on the exterior face of the building, on the walls, floors and roofs being screwed to provide a diaphragm effect (4.4.3), however, they can also be placed on the inside when it is desired to create a surface that allows the attachment of heavy objects on the wall as furniture or decorations. [6]

Having said this, the following are some of the main coatings applied to light steel construction.

1) Cross laminated timber (CLT)

Figure 3-8 – CLT board [30]

One of the oldest materials used as a structural coating is glued laminates, invented in Germany in the twentieth century by Friedrich Otto Hetzer. These are made up of successive lamellas of wood glued to each other, having the same characteristics of wood, but eliminating natural defects of the material.

In solid wood the greatest defect lies in the nodes since these are places where important stresses accumulate. However, since the lamellas are placed one above the other, there is a randomness in the dispersion of these nodes, distributing the accumulated stresses through all the board. [11]

They have the advantage of being able to stay in view, which provides a feeling of comfort to the user and allied with the pleasant aspect usually lead to buildings of great beauty. However it falls short to other claddings in some other characteristics.[6]

2) Oriented strand boards (OSB)

Figure 3-9 - OSB boards [31]

The OSB - Oriented Strand Board boards are also composed of wood sheets but shorter, in the order of 10 cm, being arranged in layers with direction perpendicular to the previous layer.

With many advantages at a reduced cost, these are the preferred choice of contractors in Portugal to coat and reinforce the LSF structure.

Firstly, they are quite easy to transport, cut and fix through screws, being that they are prepared to withstand harsh weather during the constructive process. They also allow a simple execution of the materials of finish of facades and has great resistant capacity, being able to support heavy objects such as furniture or decorations. [6]

Also, good thermal insulation levels are associated with the excellent structural behavior, making an efficient contribution in the design of LSF structures allowing stress dispersion and structural stability, being extremely resistant, especially when requested in its plan.

To prove the strength of this material, walls made with this product were shear tested and the conclusions show that this material guarantees a great stability of these walls and endows them with ductility. [11]

The EN 300 standard defines the four classes of OSB according to the environment of use, mechanical characteristics and physical properties, these being:

- OSB / 1 - General purpose signs, including interior decoration and furniture, in dry conditions
- OSB / 2 - Plates for structural purposes, in dry conditions
- OSB / 3 - Plates for structural purposes in a humid environment
- OSB / 4 - Plates for high structural performance in a humid environment

For current LSF structures the most commonly used OSB boards are Class 3 because these ensure proper weather behavior both during construction and in service.

Finally, the ideology of sustainability is given great importance, as, for example, Kronospan, one of the world leaders in the production of OSB, ensures that the wood used comes from plantations designed for this purpose and is exploited in a management context sustainable forest, taking advantage of the highest possible yield of raw materials. These also take on the commitments of the PEFC - *Program for the Endorsement of Forest Certification schemes* and FSC® - *Forest Stewardship Council®*, where they declare to guarantee the lowest environmental impact on soil, air and water, promote recycling as well as accelerate the recycling of wood residues from other products and also handle, use, condition and destroy chemicals in a safe, healthy and environmentally friendly way. [12]

3) Cement plates ("Viroc"®)

Figure 3-10 - "Viroc" boards [11]

Cement plates or better known in Portugal by the main brand "Viroc" are composite panels consisting of a mixture of wood particles and Portland cement, dried and compressed. These combine the flexibility of the wood with the strength and durability of the cement and can be applied as a cladding both inside and outside, and are mainly applied when greater resistance to impact, fire, noise, humidity and fungi is required.

In contrast, they are substantially more expensive than other existing coatings, and more difficult to apply since they are not only heavier but require a larger screwing surface (metal profiles with larger flanges). Finally, because they are more rigid, they are more susceptible to cracking. [13]

3.2.1.2. Non-structural claddings

In contrast to the foregoing, non-structural coatings are all those which due to lack of valences in their mechanical properties do not comply with the applicable standards to be considered as an integral part of the structure. Its main function is to hide the steel frame and to serve as a surface to the finishing (eg paint or tiles) or be the same the finish itself.

In addition to OSB boards whose physical characteristics do not meet the minimum requirements to be considered structural, the main interior linings of LSF structures are gypsum boards, better known as "pladur".

The board is fabricated from a wet gypsum slurry and poured continuously onto paper, receiving a new layer on the upper surface, forming a sheet of gypsum wrapped in paper, which upon drying is cut to the desired size.

Depending on the additives they receive, they can be used in different environments, such as more humid or ones that require higher fire resistance, and can be used on interior walls and false ceilings [14]

The plates can be screwed directly onto the metal frame or onto a pre-OSB layer. The joints are treated by the application of reinforced slurry with a strip of paper or netting, in the end being sanded and being ready for final painting or for another type of finish such as tiles. [6]

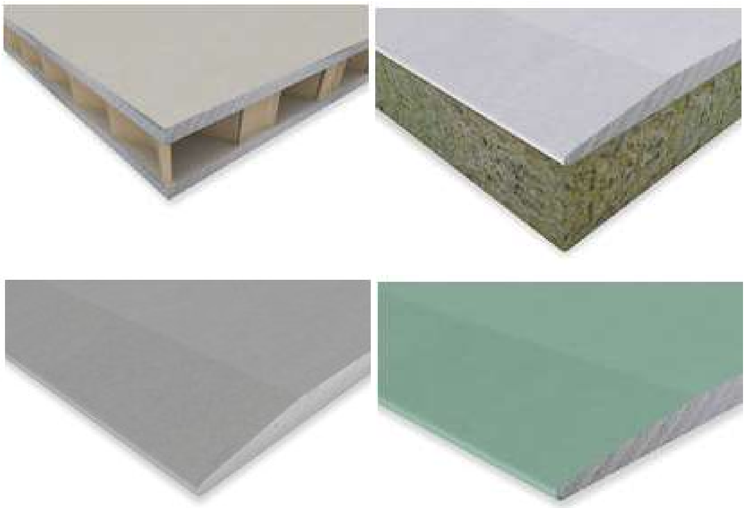


Figure 3-11 – Different kind of gypsum boards [12]

Furthermore, there are still more advanced materials that offer special advantages in terms of thermal, acoustic or fire resistance, these are not their main functions.

An example of this material is the Isolpro boards being a composite of lightweight concrete, polystyrene granules, sand, water, fiberglass and an interior structure in galvanized steel, performing thermal and

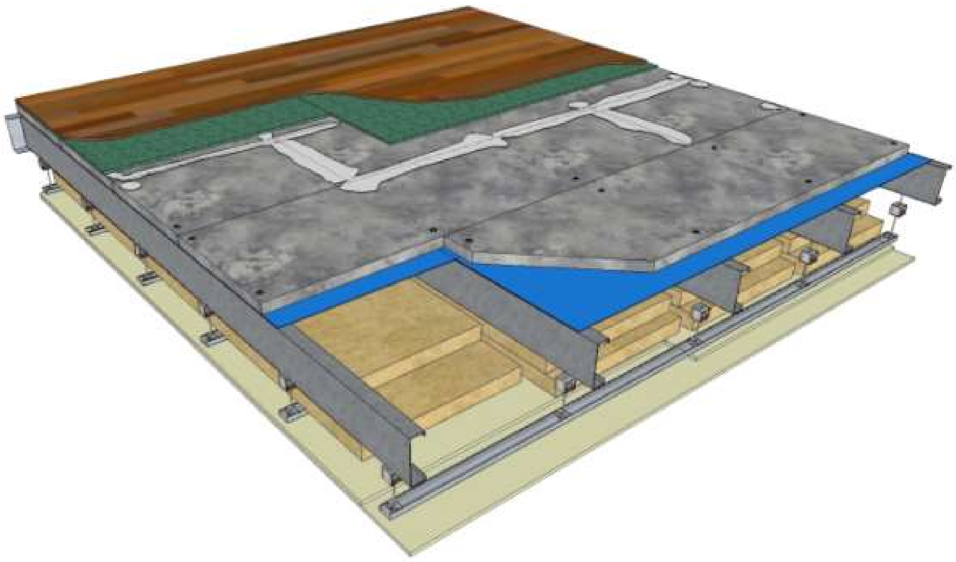


Figure 3-12 - Scheme of Isolpro boards application on a LSF floor [4]

acoustic insulation, and that can be used in both interior and exterior walls, floor coverings and facades, being adaptable to any type of construction system and allowing any finishing. [15]

3.2.2. Thermal insulation

Probably the most important component for the user is the comfort of their home regarding its thermal performance. Therefore, the materials used must provide resistance to temperature variations, however it is accepted that the steel has no property as a thermal insulation. This, for an uninformed customer might raise some doubts about its thermal performance, but the truth is that, for the most part, the surface of a wall in LSF does not have any steel, simply being limited to thermal bridges.

Usually the profiles used are 1.5 mm thick and are spaced 60 cm, in addition there are still thermal bridges of 1.5 mm near the floor and ceiling due to the thickness of the web of the lower and upper tracks. The remaining area should be covered by about 4 to 6 cm of rock wool, providing that the percentage of the surface of the thermal bridges is a maximum of 0.375%, with 99.625% of the area being properly insulated. [16]

Parallel to the insulation guaranteed by rock wool, it is quite common that the facades of buildings are coated with polystyrene (extruded or expanded), either to ensure the waterproofing of the walls, but also to eliminate thermal bridges.

In short, especially due to the "light" component of these buildings, the thermal inertia are practically negligible, which is why the effect of HVAC systems is felt much more quickly, since it is not necessary to heat / cool the walls in order to heat / cool the entire volume of the room. Compared to traditional construction, for the same level of comfort, it is needed less power, and therefore increasing energy savings.

3.2.3. Acoustic insulation

In order to have a good acoustic performance it is necessary to intercalate high density materials with low density materials.

As previously mentioned, the walls of an LSF construction are made up of several layers of different materials. On the one hand, the OSB and the plasterboard provide mass to the walls, while, on the other hand, rock wool is very efficient in acoustic insulation thanks to its mineral structure, high damping and also the reduction of resonance effect in the plasterboard. Finally, it should be noted that extruded polystyrene adds resistance to the passage of sounds.

The joint action of these materials leads to the fact that the dwellings in LSF have an acoustic behavior quite different from the traditional construction, being that the sound produced in the interior is not absorbed by the walls, but reflected by these, preventing three times more the propagation of the noise than on a brick wall.

Lastly, there is the impact sound, which produces a hollow sound, since the sound is transmitted only partially to the other side of the wall. Due to the lower density of the structural elements and coatings, better insulation of percussion sounds is ensured, however, the aerial sounds are more difficult to insulate with a level similar to the first ones. [16]

3.3. **Connections**

In order to match the various materials mentioned above, there are also different types of connections possible and whose selection depends on the material to be joined.

3.3.1. Steel-to-steel connections

There are currently four types of connections possible to solidify steel elements, namely welding, rivets, nails and screws.

Welding should be avoided from being applied as it induces very high residual stresses on the thin sheets and may compromise the strength of the structural profiles.

Rivets, as they require prior drilling, become a very time-consuming solution to the intended work rate in LSF construction.

Nails, when applied by pressure gun, are a faster solution, however their use is not yet widespread in Portugal.

Finally, the most widely used connector is the screws, which should be self-drilling, thus ensuring a good work rate. These should have a "Truss" concave head (commonly referred to as a cheese head) and a length that guarantees at least three threads after the last layer to be fixed. Most commonly used screws have a diameter of 4.2mm or 4.8mm, ie # 8 or # 10 screws.

However, the diameter, quantity and spacing of screws must be properly dimensioned according to the current regulations (EC3-8), as this should never be used less than 4 screws to join profiles, and these should not be spaced less than 13mm from the edge of the profiles or between them. This way, it is always possible to screw it all over the web and flanges, but never on the lip. [16]

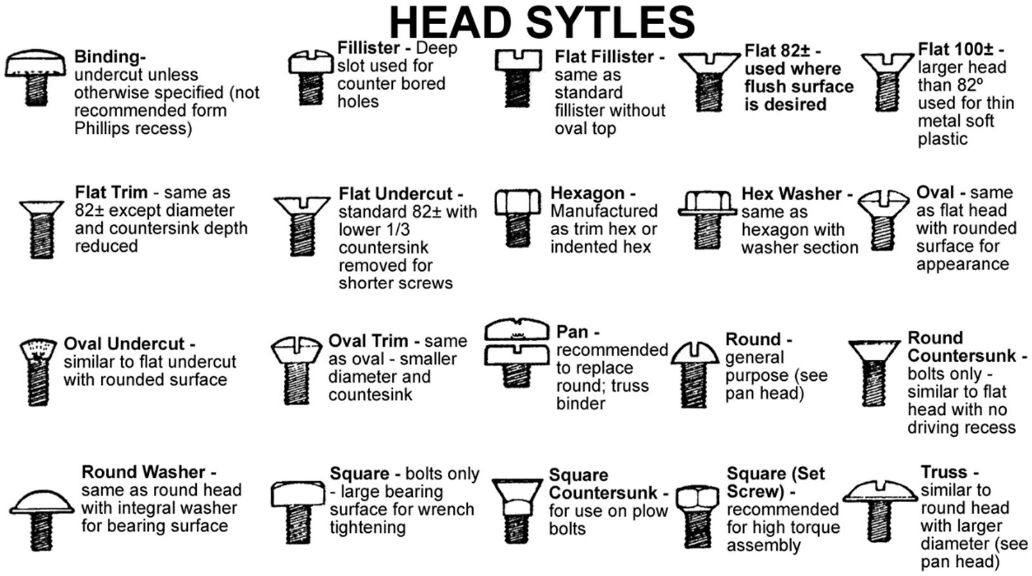


Figure 3-13 - Types of screws heads [32]

3.3.2. Cladding to framing connections

In the connections between the steel profiles and the OSB plates, the screw heads should be at least 7mm in diameter and should be embedded in the OSB, so that there are no protrusions that could cause finishing deficiencies.

Since OSB boards perform structural functions, these screws should be at least 9mm apart from the edges of the OSB boards and spaced by a maximum value to ensure the safety of the cutting walls from horizontal forces. This design can be done according to the tables proposed by AISI. [17]

4

Constructive Method

4.1. Terminology

Firstly, in order to properly understand the construction process, it is necessary to know the various characteristic terms of the metal construction, especially the light steel construction.

That said, Table 4-1 shows some terms and their names taken from the LSF Structure Design Manual. [16]

Table 4-1 - Main terms used in LSF construction [16]

Designation	Definition
Bearing wall	Wall that supports mainly gravity loads from slabs or roofs. This shouldn't be changed after construction.
Bracings	Normally inclined structural element applied between studs to prevent lateral movement and increase the resistance of the structure to horizontal actions (wind and earthquakes). Can additionally provide more attachment points of the diaphragms (plasterboard or OSB)
Buckling	Profile deformation due to excessive compression (studs) and / or bending (beams). Buckling in the studs may occur by flexion or flexion-torsion. In beams, buckling may occur by lateral deformation (lateral buckling). In both cases (studs and beams) local buckling may still occur (deformation of the sections without deformation of the structural element axis).
C-section	Cold formed cross-section with the shape of an "C", composed by one web, two flanges and two stiffeners. This profile is used to support mainly axial loads (studs). The dimensions of the cross-section are measured by the exterior.
Clip	Piece that allows the connection between orthogonal profiles. This piece must always be of thickness equal to or greater (whenever possible) to the profiles to be connected. As a rule, clips are in the form of a short angle section with the same flange or (less common) a short section with a U or C section are used.
Eaves	Part of the console roof extending beyond the facade plane

Flange	Section (or wall) of profiles C and U perpendicular to the web, generally 43 mm wide (maximum 50 mm)
Gable	Exterior walls perpendicular to the façade, usually without windows and without significant loads applied, except for those transmitted by the ridge beam (if any).
Interrupted stud	Studs below the sill of a window or above the same, or more frequently, above the blinders box, which only purpose is to create points to fixate the diaphragm. Doesn't have any applied loads.
King studs	Studs on the sides of the openings that receive loads from that span. These studs don't overcome all the height so they must be fixed to other studs that do overcome all the height.
Lintel	Beam (C-profile or truss) above a door or window span.
Lip	Also known as stiffeners, it is each part of the plate parallel to the web on the open side of a C-profile which makes the flanges stiffer and the cross-section more symmetrical, reducing the distance between the geometric center and the shear center. This allows to increase the resistance of the section to bending and also to local instabilities.
Partition wall	Wall whose only function is to create divisions. No significant loads are applied. Their disposition com be easily changed.
Ported Alignment	The "philosophy" of construction underlying the LSF; alignment, as far as possible, of the sturdy elements of the construction, to transmit the loads to the ground by pure compression on these elements. In vertical elements, the generated moments are negligible. This philosophy deals with the concept of load distributed (albeit only in the "knife load" version (kN / m) as opposed to the logic of the typical concrete pillar-and-beam construction where point loads are generated).
Ridge	Horizontal edge at the meeting of two waters
Shear wall	Wall designed to face loads applied on its own plan (horizontal loads), general rule with resistant diaphragms and/or diagonal bracings.
Span	Distance between two supports of an horizontal element
Strip	Metal sheet without any folds of a given width. These elements can only be subjected to tension, so they are usually used in pairs. (In a panel under shear loads, one of the diagonals is subjected to tension an the other to compression). In general, panels have diaphragms that dismiss the use of these strips, so they are only used when these panel receive loads before the fixation of this diaphragms. (for example, in interior walls where the claddings are only applied when all the structure is finished.)
Structural claddings	The panels claddings when they are effectively working as a diaphragm. Usually in OSB but can be with others board materials. Gypsum when used with only one layer has no structural resistance.
Studs	Vertical structural element that receives loads from beams and transfers them to the inferior floor, overcoming all the floor height.

Tracks (upper and lower)	U-profiles where the studs are fixed at floor and ceiling level. At ground level (foundation slab) they are normally fixed on asphalt screen with chemical bushing.
Truss	Structure formed by bars and nodes in which the nodes rotations are allowed. The loads must be applied on these nodes and so the only stresses on the bars are axial tension or compression.
U-section	Cold formed cross-section with the shape of an "U", composed by one web and two flanges. These profiles are made to support mainly shear loads (beams), or for constructive reasons (tracks). The dimensions of the cross-section is measured by the interior.
Web crippling	Plastic deformation (irreversible) of the web due to the action of excessive concentrated local loads or due to support reactions.

4.2. Foundations

The foundations of an LSF building is basically the same as in any other traditional construction since they are still made of reinforced concrete elements.

However, it is more often used lintel beams around the building, as well as the remaining interior walls that receive load. On the other hand, it is also very common to use ground slabs, since it's a method that requires much less time consuming.

Note that, generally, the foundations of steel structures are smaller, since the permanent loads are also smaller than those found in concrete or masonry construction. Having this said, the foundations must be designed to resist the vertical forces as well as the uplifting induced by the wind forces.

However, these are always limited to a minimum of 20 cm, as the lower tracks are fixed to foundations through various M16 bolts, spaced 50 cm apart, and fixated using chemical bushing.

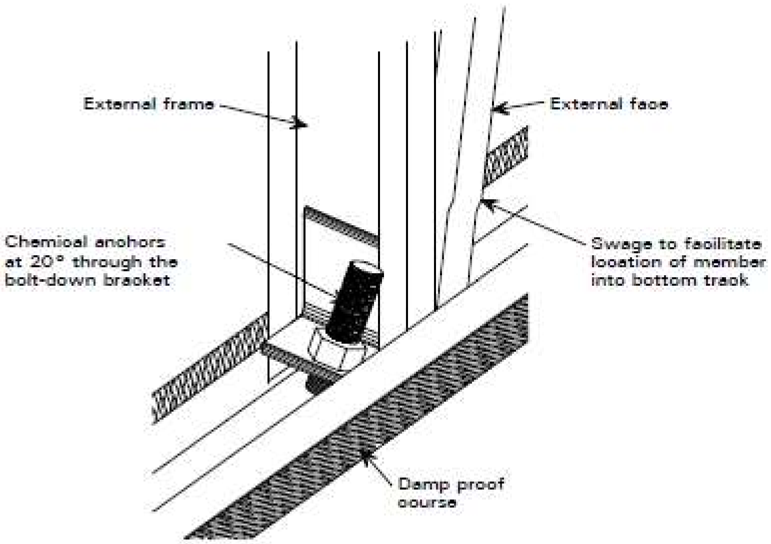


Figure 4-1 - Scheme of track to foundation connection [19]

That said, as these are elements in reinforced concrete, they are no longer dealt with further detail in this dissertation.

4.3. Constructive methods

According to experience of the constructor, the kind of building and the required fastness, there are essentially four light steel construction methods, which can be defined as follows:

4.3.1. Stick construction

In this construction method, discrete members are assembled on site to form columns, walls, joists, beams or braces, to which the cladding, interior linings and other elements are affixed. The profiles usually come from the factory pre-cutted with exact dimensions as well as with open holes for the passage of the installations, however, the connections are made in situ using self-tapping and self-tapping screws.

The main advantages of this method are:

- Any modifications can be accommodated on site, as well as ensuring the tolerances of the construction.
- Relatively simple construction technique.
- The contractor is not required to have a prefab factory as in panel construction or modular.
- Large quantities of structural elements can be accommodated and transported in single loads.

Element construction usually requires a lot of labor on the construction site, however it can be very useful when dealing with more complex constructions where prefabrication is not a viable solution. [18]

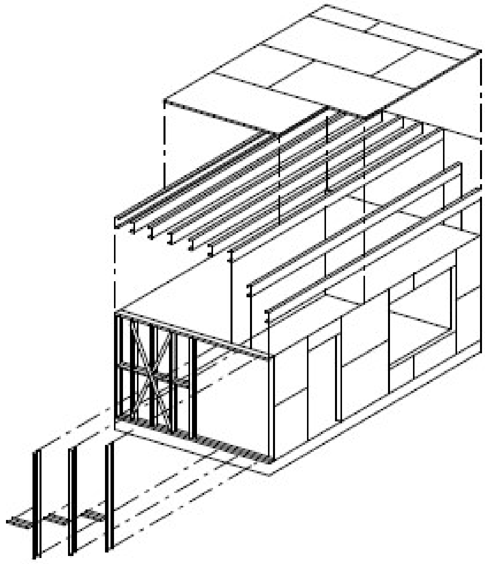


Figure 4-2 - Stick construction [19]

4.3.2. Panel construction

In order to reduce construction time on site, wall and floor panels, as well as roof trusses can be prefabricated on factory and then assembled on site. Considering that these elements are built in the factory, better accuracy is more easily achieved by mitigating possible deficiencies that could be noticed in the finishes.

Therefore, the main advantages of this method are:

- Building erection speed.
- Quality control in production.
- Minimization of costs on site.
- Possible factory production automation.

Because panels are prefabricated in a controlled environment, the geometric accuracy and reliability of these and other components is enhanced. Good preparation for quick assembly of the panels is essential in order to obtain a good efficiency of the construction process on site.

Finally, the size and weight of the panels are determined by transport service, erection facility and on-site assembly. [18]

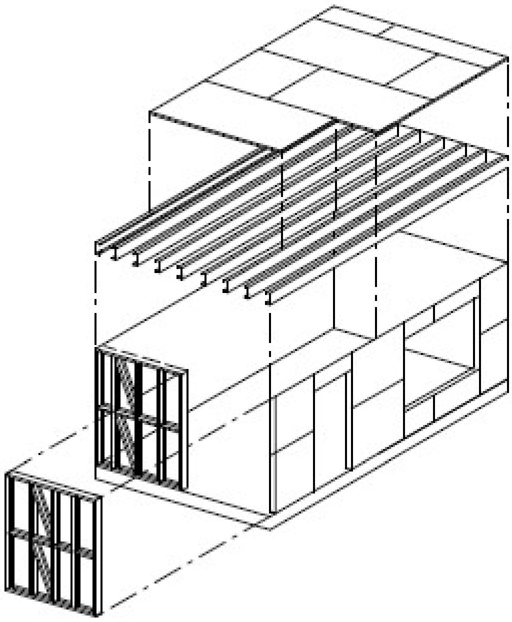


Figure 4-3 - Panel construction [19]

4.3.3. Modular construction

In modular construction, the so-called volumetric units are completely prefabricated on factory and can be delivered on site with all internal trim, fixtures and fittings properly installed. The units are placed side by side or stacked on top of each other to be assembled in their final form. Alternatively, these modules can be inserted into buildings already constructed to create pre-built partitions.

This type of construction has been gaining in popularity especially in the case of mass construction due to economies of scale, quality control and rapid on-site construction. [18]

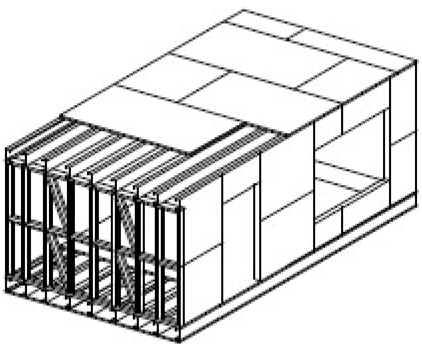


Figure 4-4 - Modular construction [19]

4.3.4. Balloon frame construction

This method is in fact a complement to the methods of elemental construction or panel construction, since in platform construction the walls and floors are constructed sequentially, one floor at a time, and the walls are not structurally continuous. In some forms of construction, loads from the upper walls are transferred from the floors to the lower walls. That is, in platform construction, the floors rest directly on top of the upper wall rail. This type of construction is mainly used in domestic scale buildings.

On the other hand, the "balloon" construction, the wall panels are usually much larger and are also continuous on more than one floor, being the floors fixed to the side of the walls. The loads of the upper walls are transmitted directly to the lower ones. [18]

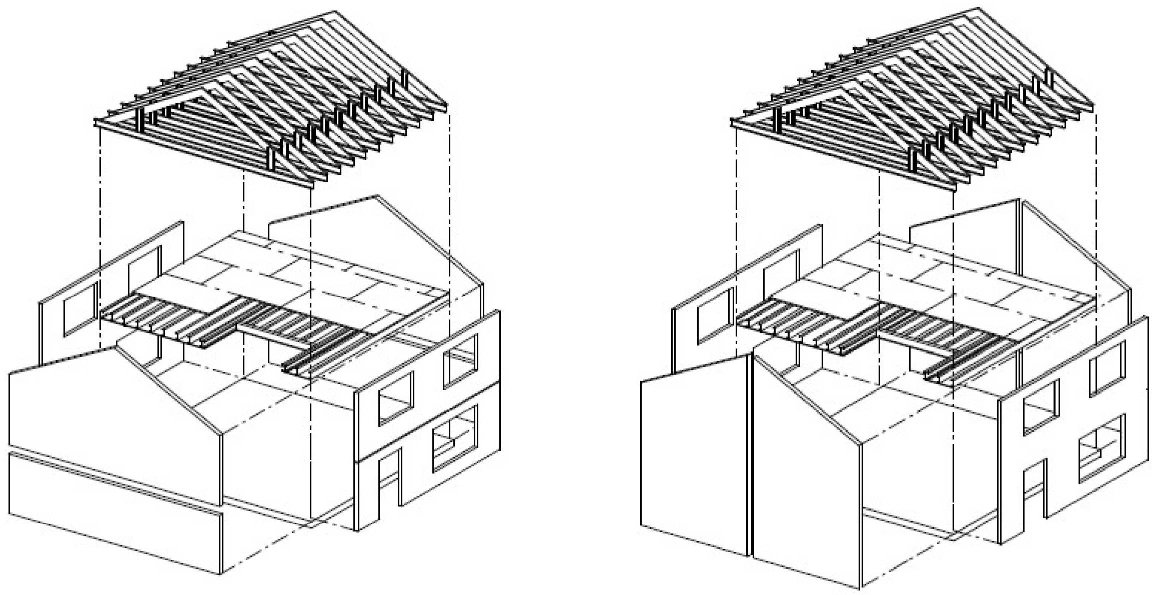


Figure 4-5 - Platform construction (on the left) and balloon frame construction (on the right) [18]

In both construction cases, the exterior cladding and finishes are installed and affixed to the profiles on site.

4.4. **Bracings**

All structures must be of adequate stiffness to prevent excessive displacements when required by horizontal forces such as wind forces or seismic forces.

There are three main ways of ensuring the structure's stability in the vertical plane: “K” integral bracing, X bracing and diaphragm effect action.

Either method offers a viable solution for transferring wind forces to foundations.

4.4.1. K - Bracings

In this method, C-section profiles are fixed diagonally between the vertical uprights within the thickness of the profiles. The diagonal members of the brace should be well fixed to the vertical elements to ensure the transfer of forces both in tension and in compression. [19]

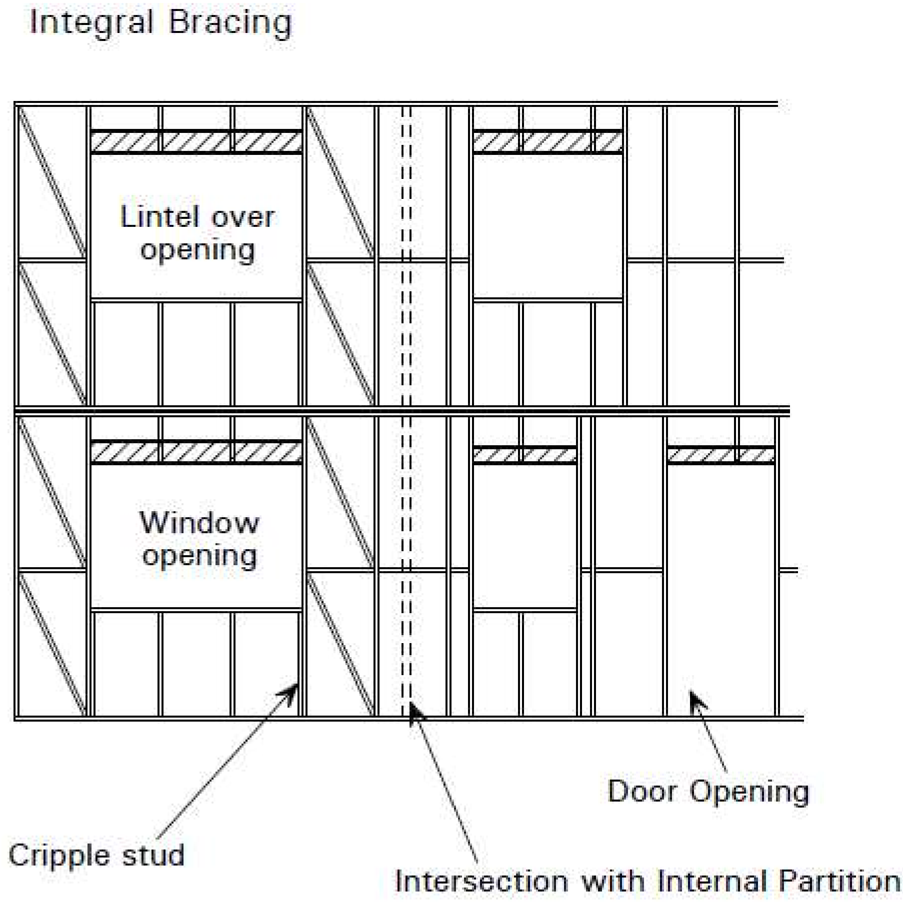


Figure 4-6 - K-Bracings [18]

4.4.2. X - Bracings

X-crossed thin steel sheets are attached to the outer faces of the uprights. These strips act only in tension and can bend unless pre-tensioned during installation. Crossbands must be fixed to all vertical elements. [19]

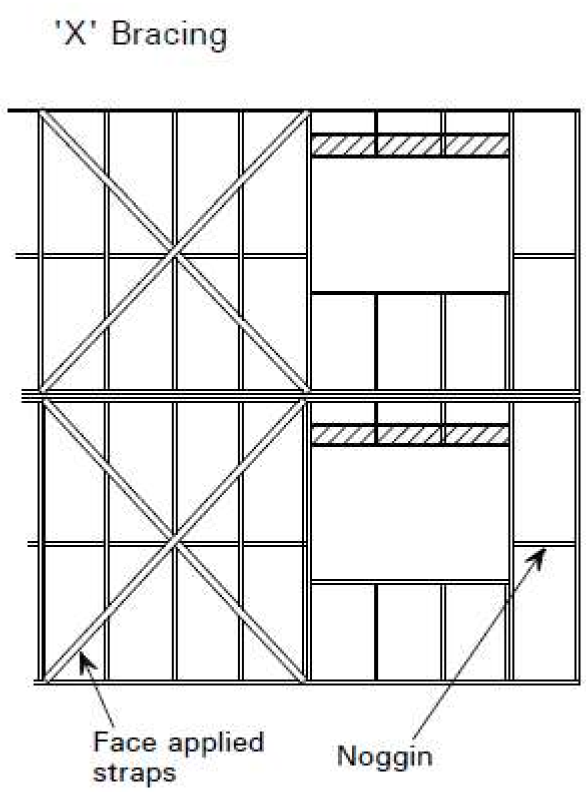


Figure 4-7 - X-Bracings [18]

4.4.3. Diaphragm Effect

Both walls, floors and roofs are often covered with boards of suitable materials (eg chipboard, chipboard, or plasterboard) that act as structural diaphragms for transferring forces to braces or foundations.

This diaphragm effect is achieved by attaching the structural plates to the metal profiles using self-drilling screws never spaced more than 300mm apart.

The diaphragms can also be used in the horizontal plane as the floors must be designed to drive the shear forces to the bracing walls. Note that if there are areas with significant openings (such as windows or doors), it is important to ensure that the force path is maintained. Eurocode 5 proposes a method for detailed diaphragm design which, although directed to wooden structures, its principles can be adopted for light steel structures. [19]

5
Design regulations

The design of steel structures made of cold formed profiles is usually characterized by the complexity resulting from the high slenderness of the cross sections. This high slenderness potentially causes cross-sectional distortion phenomena, in addition to the local section and global buckling, traditionally present and regularly treated for hot-rolled sections.

In regulatory terms, and in the context of the Structural Eurocodes development program and in particular EC3 (Design of Metal Structures), this additional complexity has led to the elaboration of a specific part of EC3 to deal with the design of cold formed profiles, to part 1.3.

Finally, it will be necessary to use Eurocode 5, relative to wood structures, in order to design and do all the required safety checks on the OSB diaphragms, which guarantee the safety to horizontal loads.

5.1. Cold-formed steel specific phenomenon

5.1.1. Local buckling

The high slenderness of the different elements that make up the cross-section leads to local buckling phenomena, which must therefore be explicitly considered in the design of this type of section. In addition, the post bending strength of thin elements is usually stable, so it is possible to consider the post bending reserve when sizing these sections in order to obtain an economical solution.

According to Eurocode 3, part 1.3, the design of cold formed elements is based on the effective section method. This method considers the reduction of section strength due to local buckling by reducing the size of each of the cross-section elements (effective section).

5.1.2. Torsion

The slimness of the cross sections implies a normally very low torsional stiffness. Most sections produced by the cold hanging process are monosymmetric or even asymmetric, and where the shear center does not coincide with the center of gravity of the section, therefore it is essential to consider secondary torsional moments due to the eccentricity between the axis. load action and the shear center. Thus, it may be necessary for such elements to be restricted to torsion either at regular intervals or continuously over their entire length.

5.1.3. Distortion

Cross sections prevented from laterally deforming and / or twisting may still undergo a buckling mode commonly referred to as distortion buckling. This mode of buckling may occur in compressed and / or flexed limbs. EC3-3 considers the reduction of section strength due to distortion buckling by reducing the thickness of the reinforcement. [20]

5.2. **Cross-section classifications**

As is Known, structural steel profiles cross sections, regardless of being welded or hot rolled, are formed by assembling individual plate elements. This, when in compression, can buckle locally

Local buckling in cross sections may limit the load bearing capacity by preventing yield stress from being achieved.

Even though the cross-sections of cold formed sections come from a single piece (plate) which has been bent to form the final section, ii is in the end made up of several slender elements. I tis further evident that because it's walls are so thin, the possibility of this phenomenon is even greater.

According to EC3., cross sections can be classified as the following:

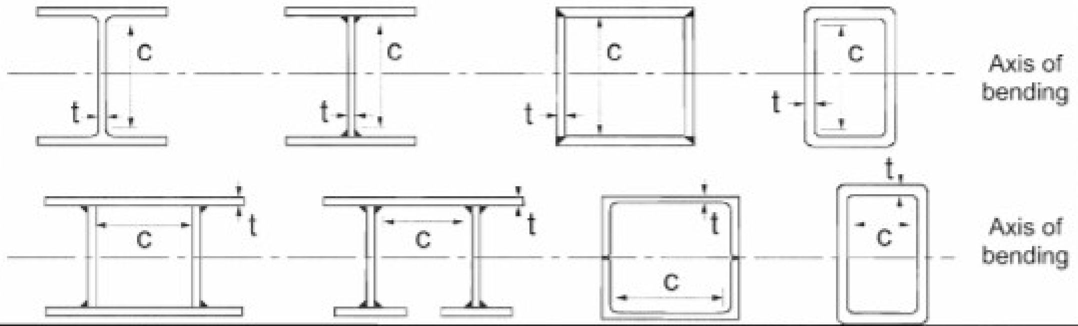
- Class 1 – Cross sections which can develop plastic hinges, with enough rotation capacity to perform a plastic analysis, without reducing its resistance;
- Class 2 – Cross sections which can achieve the resistant plastic moment, but which rotation capacity is limited by local buckling;
- Class 3 – cross sections where the tension in the farthest compressed fiber, calculated using a elastic distribution of stress, can achieve the yielding stress, but where the local buckling can stop the plastic resistant moment from being fulfilled;
- Class 4 – Cross sections where local buckling occurs before the yielding stress in one or more parts of the cross section.

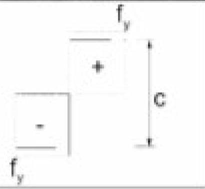
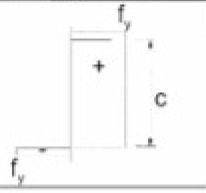
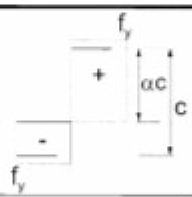
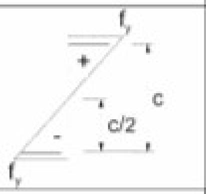
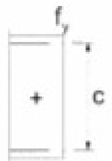
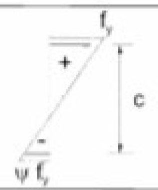
Eurocode 3 also exposes that the class of sections depend on the slenderness of the comprised element (length-thickness ratio), and also on the demand, class of steel, and kind of profile. Through tables 5-1 and 5-2, it's possible to calculate the parameters identified and whenever a cross section is not in the limits of a class 3 then it will be considered as class 4, and so it is required to considerate buckling calculation.

The classes of the web and flanges can be different, but the class of the cross section will be the highest of them all, meaning the worst case.

Table 5-1 Maximum width-to-thickness ratios for compression parts [21]

Internal compression parts



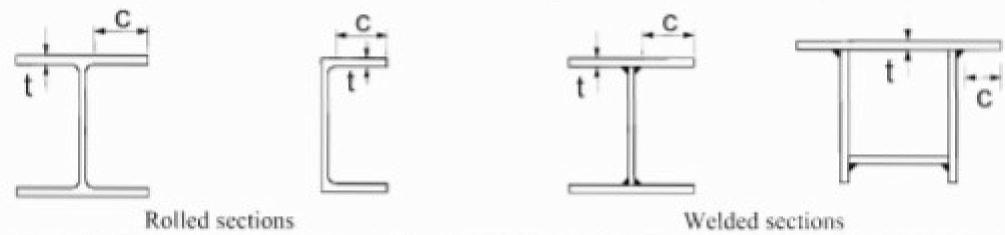
Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \le 72\varepsilon$	$c/t \le 33\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \dfrac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \dfrac{36\varepsilon}{\alpha}$
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 $\psi \le -1^{*)}$: $c/t \le 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

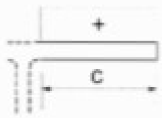
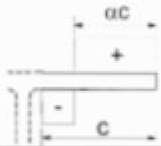
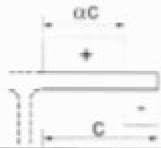
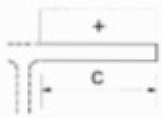
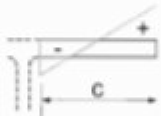
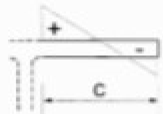
$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

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

Table 5-2 - Maximum width-to-thickness ratios for compression parts [21]

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	$c/t \leq 9\varepsilon$	$c/t \leq \frac{9\varepsilon}{\alpha}$	$c/t \leq \frac{9\varepsilon}{\alpha\sqrt{\alpha}}$
2	$c/t \leq 10\varepsilon$	$c/t \leq \frac{10\varepsilon}{\alpha}$	$c/t \leq \frac{10\varepsilon}{\alpha\sqrt{\alpha}}$
Stress distribution in parts (compression positive)			
3	$c/t \leq 14\varepsilon$	$c/t \leq 21\varepsilon\sqrt{k_\sigma}$ 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

5.3. Effective properties

When the profile is in fact class 4, the effective resistance should be based on the effective cross section.

Effective cross sections are equivalent to cross section class 4 but with reduced dimensions, obtained by disregarding the material in the areas susceptible to buckle. With this new effective cross section, the effective properties can be calculated through a linear-elastic calculus.

The effective length of the comprised elements is calculated through a reducer factor p, defined as a function of normalized slenderness of the element.

5.4. Resistance of cross-sections

In order to ensure safe conditions, the axial force, Ned must meet the following criteria:

$$\frac{N_{ed}}{N_{Rd}} \leq 1.0$$

5.4.1. Axial tension

The design resistance of a cross section for uniform tension $N_{t,Rd}$ should be determined from:

$$N_{t,Rd} = \frac{f_y A_g}{\gamma_{M0}}$$

5.4.2. Axial compression

The design resistance of a cross section for compression $N_{c,Rd}$ should be determined from:

$$N_{c,Rd} = \frac{f_y A_{eff}}{\gamma_{M0}}$$

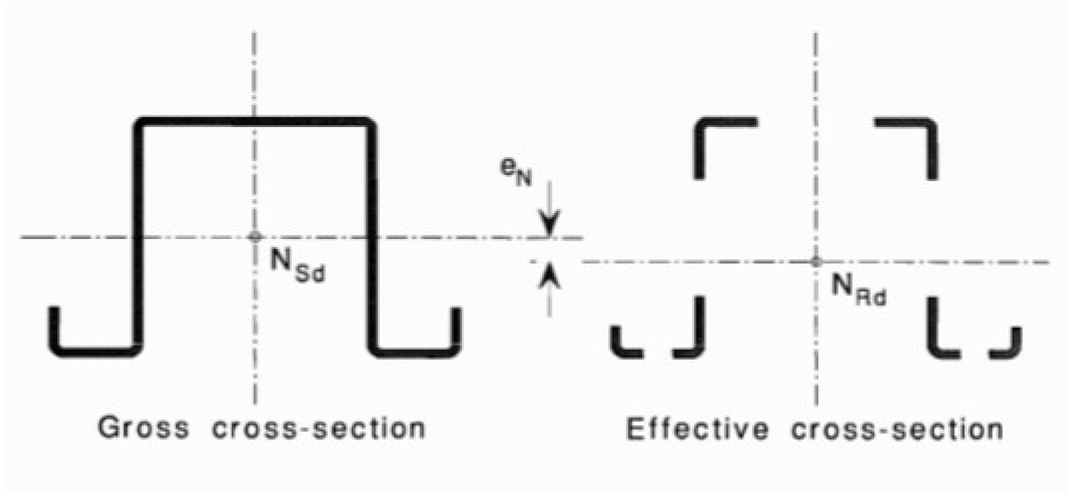


Figure 5-1 Effective cross-section under compression [4]

5.4.3. Bending moment

The design moment resistance of a cross-section for bending about one principal axis Mc,Rd is determined as follows:

$$M_{c,Rd} = \frac{W_{eff}\, f_y}{\gamma_{M0}}$$

For biaxial bending the following criterion may be used:

$$\frac{M_{y,Ed}}{M_{cy,Rd}} + \frac{M_{z,Ed}}{M_{cz,Rd}} \leq 1$$

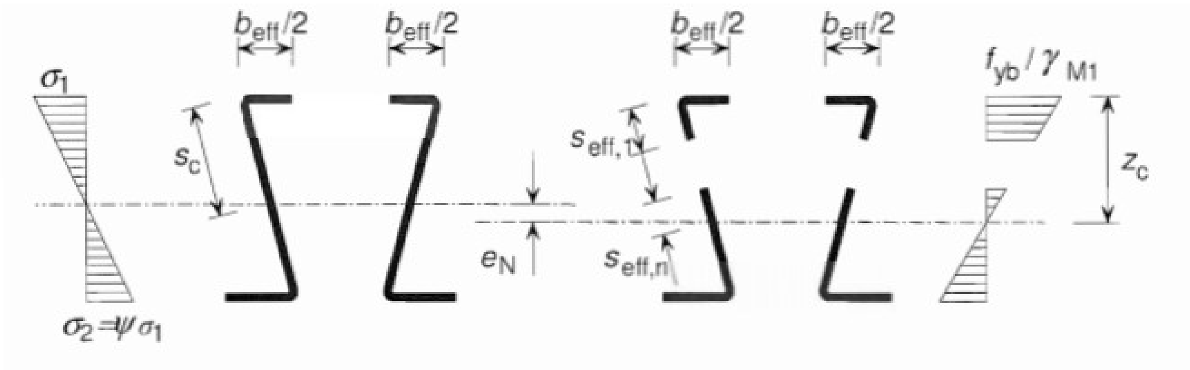


Figure 5-2 - Effective cross-section for bending resistance to bending moments [4]

5.4.4. Shear force

The design shear resistance $V_{b,Rd}$ should be determined from:

$$V_{b,Rd} \leq \frac{\frac{h_w}{\sin \Phi} t f_{bv}}{\gamma_{M0}} 1$$

The value of the shear strength considering buckling – f_{bv} is according to Table 5-3

Table 5-3 - Shear buckling strength f_{bv} [4]

Relative web slenderness	Web without stiffening at the support	Web with stiffening at the support [1)]
$\bar{\lambda}_w \leq 0{,}83$	$0{,}58 f_{yb}$	$0{,}58 f_{yb}$
$0{,}83 < \bar{\lambda}_w < 1{,}40$	$0{,}48 f_{yb} / \bar{\lambda}_w$	$0{,}48 f_{yb} / \bar{\lambda}_w$
$\bar{\lambda}_w \geq 1{,}40$	$0{,}67 f_{yb} / \bar{\lambda}_w^2$	$0{,}48 f_{yb} / \bar{\lambda}_w$
[1)] Stiffening at the support, such as cleats, arranged to prevent distortion of the web and designed to resist the support reaction.		

The relative web slenderness λ_w should be obtained from the following:

For webs without longitudinal stiffeners:

$$\lambda_w = 0{,}346 \, \frac{S_w}{t} \sqrt{\frac{f_y}{E}}$$

For webs with longitudinal stiffeners:

$$\lambda_w = 0{,}346\ \frac{S_d}{t}\sqrt{\frac{5{,}34}{k_t}\frac{f_y}{E}} \qquad but \qquad \lambda_w \geq 0{,}346\ \frac{S_p}{t}\sqrt{\frac{f_y}{E}}$$

where:

$$k_t = 5{,}34 + \frac{2{,}10}{t}\left(\frac{\Sigma\, I_s}{S_d}\right)$$

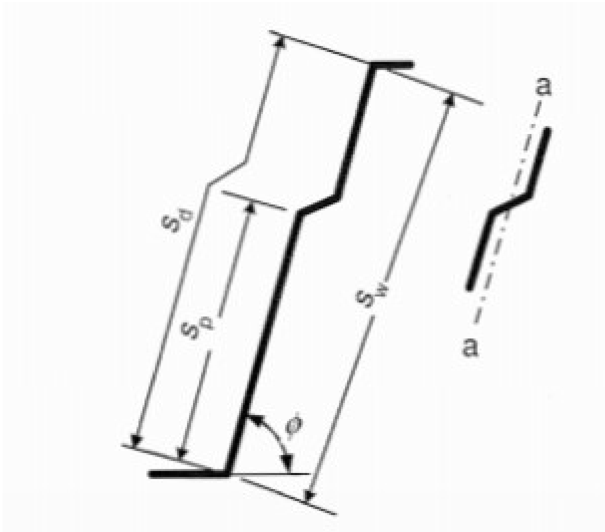


Figure 5-3 - Longitudinally stiffened web [4]

5.4.5. Torsional moment

Where loads are applied eccentric to the shear center of the cross-section, the effects of torsion should be taken into account.

In cross-sections subjected to torsion, the following conditions should be satisfied:

$$\sigma_{tot,Ed} \leq \frac{f_y}{\gamma_{M0}}$$

$$\tau_{tot,Ed} \leq \frac{f_y/\sqrt{3}}{\gamma_{M0}}$$

$$\sqrt{\sigma_{tot,Ed}{}^2 + 3\ \tau_{tot,Ed}{}^2} \leq 1{,}1\ \frac{f_y}{\gamma_{M0}}$$

5.4.6. Local transverse forces

To avoid crushing, crippling or buckling in a web subjected to a support reaction or other local transverse force applied through the flange, the transverse force F_{Ed} shall satisfy:

$$F_{Ed} \leq R_{w,Rd}$$

For a cross-section with a single web, must satisfy the following criteria:

$$\frac{h_w}{t} \leq 200$$

$$\frac{r}{t} \leq 6$$

$$45° \leq \Phi \leq 90°$$

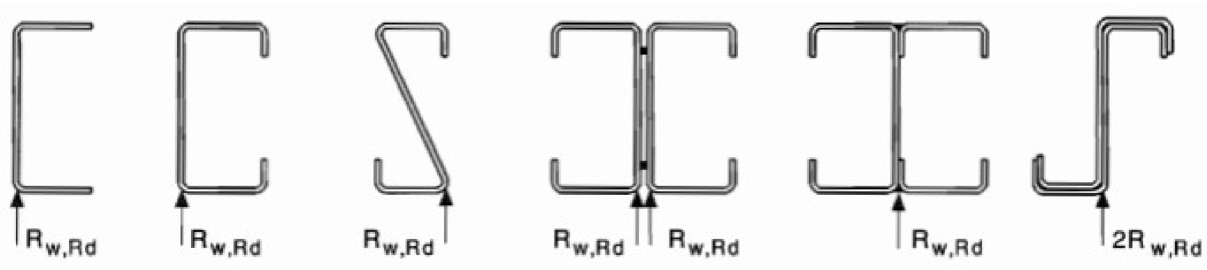


Figure 5-4 - Examples of cross-sections with a single web [4]

When the cross-sections satisfy this criteria, the local transverse resistance $R_{w,Rd}$ may be determined according to the respective case presented on the section 6.1.7.2 of the Eurocode 3-3 [4].

These cases are defined according to the position of the local transverse loads, whether they are single or two opposing local loads, and if the web rotation is prevented.

On the other hand, for cross-sections with two or more webs, including sheeting, like in the Figure 5-5, the local resistance of an unstiffened web may be determined according to section 6.1.7.3., attending that the following criteria is fulfilled:

$$\frac{h_w}{t} \leq 200 \sin \Phi$$

$$\frac{r}{t} \leq 10$$

$$45° \leq \Phi \leq 90°$$

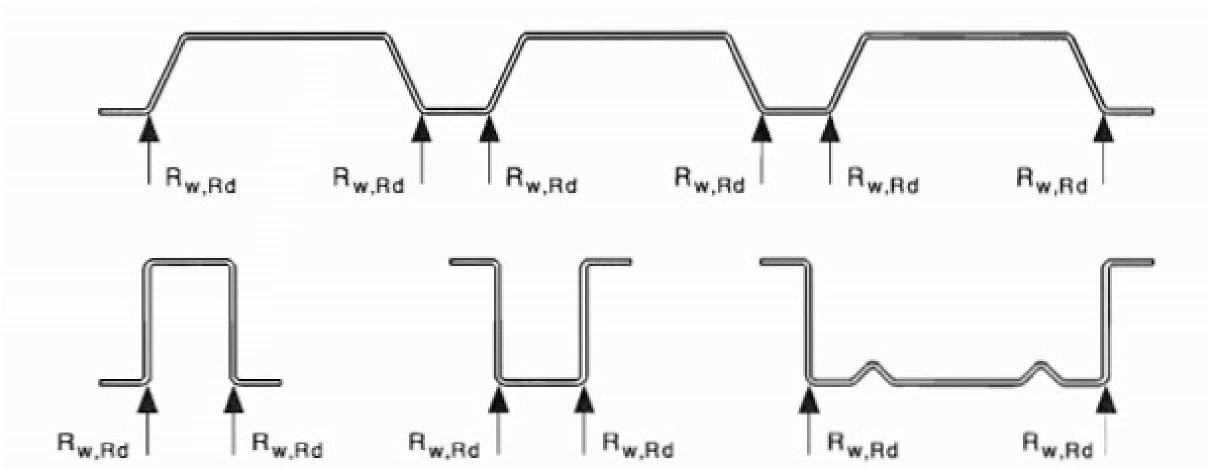


Figure 5-5 - Examples of cross-sections with two or more webs [4]

For elements with two or more webs, the local transverse resistance $R_{w,Rd}$ per web of cross-section should be determined from:

$$R_{w,Rd} = \alpha\, t^2 \sqrt{f_y\, \mathrm{E}}\ \left(1 - 0{,}1 \sqrt{\frac{r}{t}}\right)\left[0{,}5 + \sqrt{0{,}02\ I_a \mathrm{t}}\right](2{,}4 + (\Phi/90)^2)/\,\gamma_{M1}$$

This equation has in consideration the value of the acting local transverse forces (V_{Ed}), and also divides in two categories according to the cases specified before.

5.4.7. Combined tension and bending

Cross-sections subjected to combined axial tension N_{Ed} and bending moments $M_{y,Ed}$ and $M_{z,Ed}$ should satisfy the criterion:

$$\frac{N_{Ed}}{N_{t,Rd}} + \frac{M_{y,Ed}}{M_{cy,Rd}} + \frac{M_{z,Ed}}{M_{cz,Rd}} \leq 1$$

5.4.8. Combined compression and bending

Cross-sections subjected to combined axial compression N_{Ed} and bending moments $M_{y,Ed}$ and $M_{z,Ed}$ should satisfy the criterion:

$$\frac{N_{Ed}}{N_{c,Rd}} + \frac{M_{y,Ed} + \Delta\, M_{y,Ed}}{M_{cy,Rd}} + \frac{M_{z,Ed} + \Delta\, M_{z,Ed}}{M_{cz,Rd}} \leq 1$$

5.4.9. Combined shear force, axial force and bending moment

For cross-sections subjected to the combined action of an axial force N_{ed}, a bending moment M_{Ed} and a shear force V_{Ed} no reduction due to shear force need not to be done provided that $V_{ed} \leq 0{,}5\ V_{w,Rd.}$ If the shear force is larger than half of the shear force resistance, then following equations should be satisfied:

$$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y,Rd}} + (1 - \frac{M_{f,Rd}}{M_{pl,Rd}})(\frac{2\,V_{Ed}}{V_{w,Rd}} - 1)^2 \leq 1$$

5.4.10. Combined bending moment and local load or support reaction

Cross-sections subject to the combined action of a bending moment M_{Ed} and a transverse force due to a local load or a support reaction F_{Ed} should satisfy the following:

$$\frac{M_{Ed}}{M_{c,Rd}} \leq 1$$

$$\frac{F_{Ed}}{R_{w,Rd}} \leq 1$$

$$\frac{M_{Ed}}{M_{c,Rd}} + \frac{F_{Ed}}{R_{w,Rd}} \leq 1{,}25$$

5.5. **Buckling Resistance**

5.5.1. Uniformly comprised elements

5.5.1. General

A comprised element shall verify the following criteria:

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

5.5.2. Flexural buckling

The design buckling resistance $N_{b,Rd}$ for flexural buckling should be obtained from the following expression, using the appropriate buckling curve from the table 6.3 of the Eurocode 3 part 1-3 [4] according to the type of cross-section, axis of buckling and yield strength used.

The design buckling resistance of a comprised element shall be considered equal to:

$$N_{b,Rd} = \frac{\mathcal{X}\,A_{Eff}\,f_y}{\gamma_{M1}}$$

In which $\mathcal{X}$ is a reduction coefficient for the relevant buckling mode, calculated as the next equation:

$$\mathcal{X} = \frac{1}{\Phi + \sqrt{\Phi^2 + \lambda^2}} \quad but\ \ \mathcal{X} < 1{,}0$$

Where:

$$\Phi = 0{,}5\,[1 + \alpha(\lambda - 0{,}2) + \lambda^2]$$

$$\lambda = \sqrt{\frac{A_{Eff}\,f_y}{N_{cr}}} = \frac{L_{cr}}{i}\frac{1}{\lambda_1}$$

$$\lambda_1 = \pi \sqrt{\frac{E}{f_y}} = 93{,}3\varepsilon$$

$$\varepsilon = \sqrt{\frac{235}{f_y}}$$

N_{cr} is the critical axial force associated with the relevant buckling mode, based on the gross cross-section properties, and α is the imperfection factor from the table 6.1 of the Eurocode 3 part 1-1 [21]

5.5.3. Torsional buckling and torsional-flexural buckling

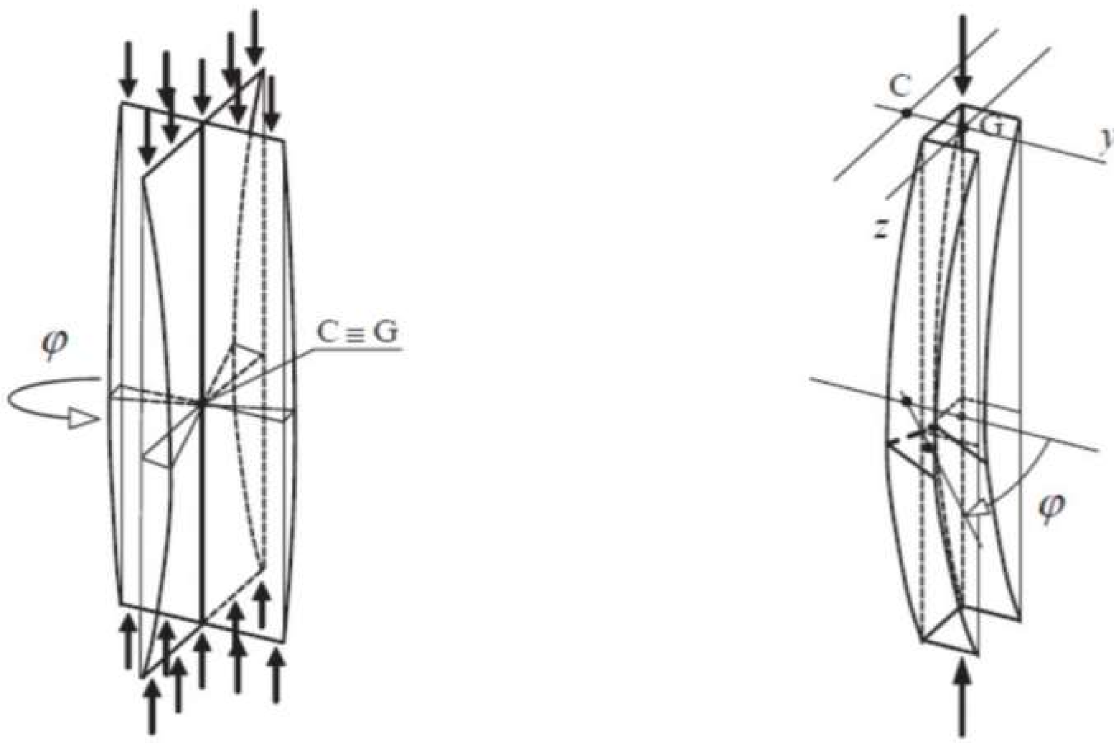


Figure 5-6 - Schemes of torsional buckling(at the left) and flexural-torsional buckling (at the right)

For members with point-symmetric open cross-sections (Z-purlins - Figure 5-7), account should be taken of the possibility that the resistance of the member to torsional buckling might be less than its resistance to flexural buckling.

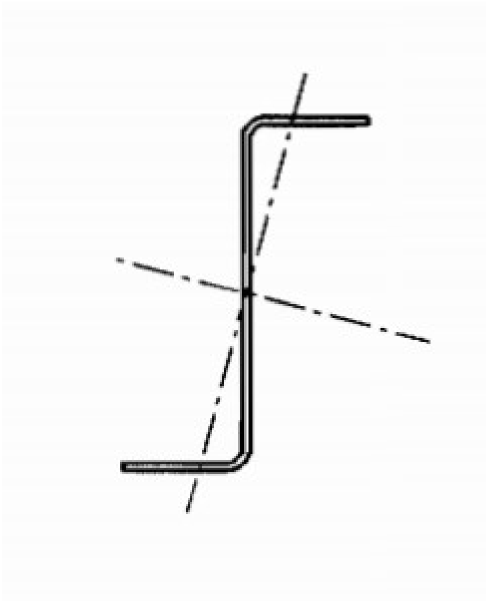


Figure 5-7 - Example of point-symmetric cross-section [4]

For members with mono-symmetric open sections (Figure 5-8) account should be taken of the possibility that the resistance of the member to torsional-flexural buckling might be less than its resistance to flexural buckling.

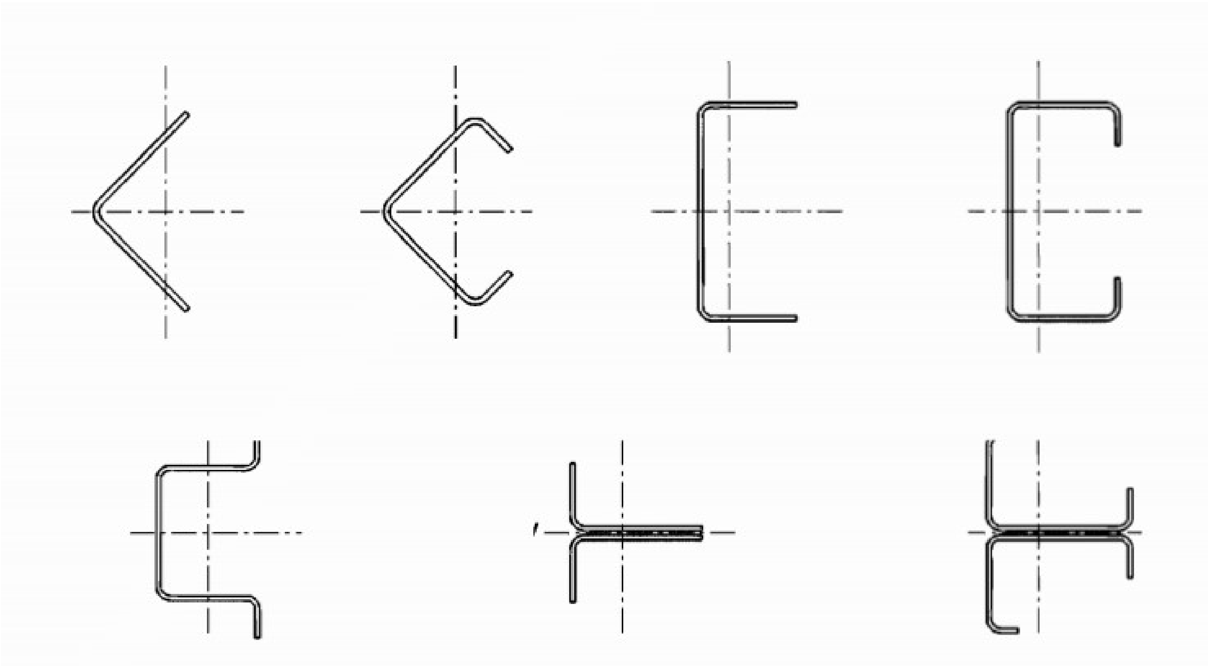


Figure 5-8 - Mono-symmetric cross-sections susceptible to torsional-flexural buckling [4]

And, at last, for members with non-symmetric cross-sections, account should be taken that the resistance to either torsional or torsional-flexural buckling can be less than its flexural buckling resistance.

With this, the design buckling resistance $N_{b,Rd}$ should be obtained considering a slenderness for torsional or torsional-flexural buckling. This slenderness shall take in count the critical axial force, that must be the smallest of $N_{cr,TF}$ (critical axial force associated to torsional-flexural buckling) and $N_{cr,T}$ (critical axial force associated to torsional buckling)

The elastic critical force $N_{cr,T}$ should be determined from:

$$N_{cr,T} = \frac{1}{{i_0}^2}(G\,I_t + \frac{\pi^2\,E\,I_w}{{l_t}^2})$$

And the elastic critical force $N_{cr,FT}$ shall be calculated from:

$$N_{cr,FT} = \frac{N_{cr,y}}{2\,\beta}[1 + \frac{N_{cr,T}}{N_{cr,y}} - \sqrt{\left(1 - \frac{N_{cr,T}}{N_{cr,y}}\right)^2 + 4\left(\frac{y_0}{i_0}\right)^2\frac{N_{cr,T}}{N_{cr,y}}}$$

As it is exposed in 6.2.3 (5) and 6.2.3 (7), of the Eurocode 3 part 1-3 [4]

5.5.4. Lateral-torsional buckling of members subjected to bending

The design buckling resistance moment of a member that is susceptible to latera-torsional buckling should be determined according to Eurocode 3 part 1-1 while using the lateral buckling curve b.

The reduction coefficient for lateral buckling resistance $\mathcal{X}_{LT}$ is calculated as the following expression:

$$\mathcal{X}_{LT} = \frac{1}{\Phi_{LT} + \sqrt{{\Phi_{LT}}^2 + {\lambda_{LT}}^2}} \quad but\ \ \mathcal{X}_{LT} < 1{,}0$$

Where:

$$\Phi_{LT} = 0{,}5\ [1 + \ \alpha_{LT}(\lambda_{LT} - 0{,}2) + {\lambda_{LT}}^2]$$

The relative lateral torsional slenderness λ_{LT} is calculated according to the critical moment M_{cr} which is a parameter that depends on the properties of the gross cross-section, loading conditions, shape of bending moments diagram and lateral supports.

With this, the mathematic expression to calculate M_{cr} is:

$$M_{cr} = C_1 \times \frac{\pi^2 EI_z}{(KL)^2} \times \left[\sqrt{\left(\frac{K}{K_w}\right)^2 \times \frac{I_w}{I_z} + \frac{(KL)^2}{\pi^2} \times \frac{GI_t}{EI_z} + (C_2 Z_g - C_3 Z_j}\right]$$

5.5.5. Bending and Axial Loading

The Interaction between axial force and bending moment may be obtained from a second-order analysis of the member as specified in EN 1993-1-1, which requires:

$$\frac{N_{Ed}}{\frac{\mathcal{X}_Y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\mathcal{X}_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{Z,Ed} + \Delta M_{z,Ed}}{\frac{M_{Z,Rk}}{\gamma_{M1}}} \leq 1$$

And

$$\frac{N_{Ed}}{\frac{\mathcal{X}_Z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\mathcal{X}_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{Z,Ed} + \Delta M_{z,Ed}}{\frac{M_{Z,Rk}}{\gamma_{M1}}} \leq 1$$

Or, alternatively, the following formula may be used, according to Eurocode 3 part 1-3:

$$\left(\frac{N_{Ed}}{N_{b,Rd}}\right)^{0,8} + \left(\frac{M_{Ed}}{M_{b,Rd}}\right)^{0,8} \leq 1,0$$

5.6. **Serviceability limit state**

5.6.1. General

The rules applied for serviceability limit states for cold formed steel structures are the same as any other steel structure.

For buildings, the serviceability limit states must satisfy all the relevant utilization criteria. This can be found in section 3.4 of Eurocode 0, where the most important aspects are concerning to:

-Deformations (appearance, comfort of users and functioning of the structure) that affect or cause damage to finishes or non-structural members;

-Vibrations that cause discomfort to people or that limit the functional effectiveness of the structure;

-Damage that is likely to adversely affect the appearance, durability or functioning of the structure.

Attending to this, the serviceability criteria should be specified for each project and agreed with the client, although, this can be defined in the National annex.

5.6.2. Vertical deflections

Considering as reference the EN 1990 – Annex A 1.4, the maximum values for vertical deflection represented in Figure 5-9 shall be agreed with the client, whenever it's not specified in the national annex.

Figure 5-9 - Definitions of vertical deflections [22]

For the case study, the recommended values to the limits of the vertical deflections are given by the following table:

Table 5-4 - Recommended values for the limits of vertical deflections [21]

Conditions	Limits	
	δ_{max}	δ_2
General rooftops	L/200	L/250
Rooftops frequently used by people beyond maintenance people	L/250	L/300
General floors	L/250	L/300
Floors and rooftops which support plaster or other fragile claddings or non-flexible partitions	L/250	L/350
Floors that support columns (unless that the displacements were considered in the global analysis for the ultimate limit state)	L/400	L/500
When δ_{max} can affect the aspect of the building	L/250	-
Note: generally, L represents the span of the beam. In case of cantilevers, L represents twice the real span of the cantilever.		

5.6.3. Horizontal displacements

The limits for horizontal displacements due to horizontal loads and geometrical imperfections shall be agreed with the client, whenever it's not specified in the national annex. In the next figure it is presented a scheme illustrating the behavior of a two-story structure with horizontal displacement.

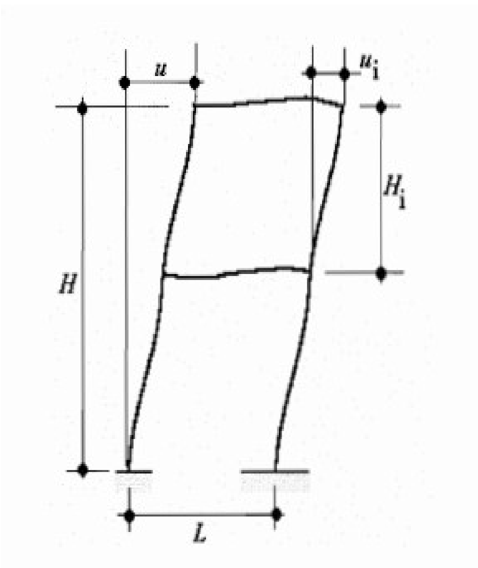


Figure 5-10 - Definition of horizontal displacements [22]

For the case study, the recommended values to the limits of the horizontal displacements are given by the following expressions:

- In multistory buildings:

- In each story:

$$h/300$$

- In the global structure:

$$h_0/500$$

5.7. Dynamic Effects/ Vibrations

To achieve satisfactory vibration behavior of buildings and their structural members under serviceability conditions should be considered the commodity of users and functioning of the structure or its structural members. For this comfort to be achieve, in design, account should be taken to all possible sources of vibrations like walking, synchronized movements of people, machinery, ground borne vibrations from traffic, and wind actions. These and other sources should be specified for each project and agreed with the client.

Therefore, in order to control the structural behavior, the natural frequency of vibrations of the structure should be kept above appropriate values which depend upon the function of the building and the source of the vibration, previously agreed with the client.

Note that if the natural frequency is lower than the appropriate value, a more refined analysis, including the consideration of damping, should be performed.

The Portuguese national annex dispenses the verification to maximum vertical accelerations of a habitation building structure when the natural frequency of vibration is above 3Hz.

5.8. Wall Diaphragms

As explained in 4.4.3, the bracings of the building can be achieved through the use of efficient structural claddings. Since these claddings are mainly wood based panels, the normative requirements that are the base to their design can be found in Eurocode 5, however, the section considering wall diaphragms only has one expression that calculates the design resistant shear (racking) strength of a wall by the use of the following expression:

$$F_{i,v,Rd} = \frac{F_{f,Rd}\ b_i}{s_0} k_d k_{i,q} k_s k_n$$

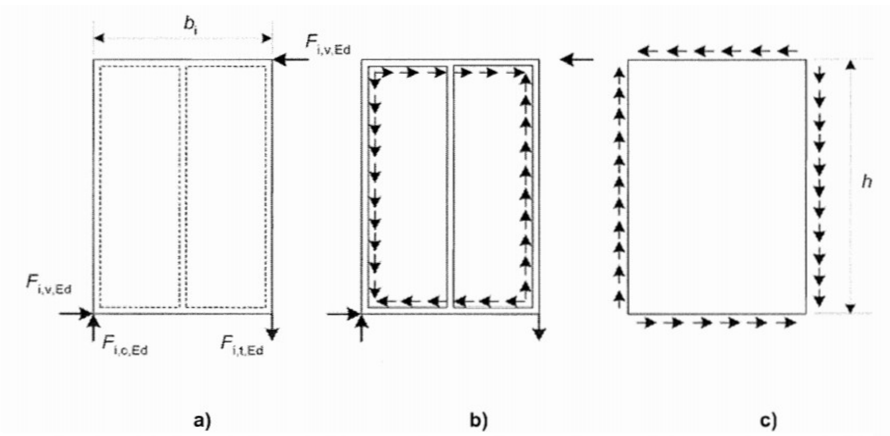


Figure 5-11 - Forces acting on: a) wall panel b) framing c)sheet [33]

And while the sum of all $F_{i,v,Rd}$ in a given direction is bigger than $F_{v,Ed}$, then the safety is guaranteed. However, this expression basically only considers the strength of the fasteners, their spacing and some board materials, instead of the whole wall components as the diaphragm. Even more, none of the parameters of the equation takes in consideration the studs or board material properties.

Because this calculation method is so confusing and looks sort of unreliable even for expert engineers experienced in wood structures, it was tried a different approach which was based on the American code AISI S400 which is the North American Standard for Seismic Design of Cold-Formed Steel Structural Systems, fundamentally a norm specifically to LSF which has methods to properly calculate the resistance of shear walls diaphragms. [17]

This norm presents a very simple design method, which consists in choosing the length of a shear wall from the following table:

Canada (kN/m)						
Assembly Description	**Max. Aspect Ratio (h:w)**	**Fastener Spacing at Panel Edges[2] (mm)**			**Designation Thickness[5] of Stud and Track (mils)**	**Required Sheathing Screw Size**
		150	100	75		
9.5 mm CSP Sheathing	2:1[3]	8.5	11.8	14.2	43 (min.)	8
12.5 mm CSP Sheathing	2:1[3]	9.5	13.0	19.4	43 (min.)	8
12.5 mm DFP Sheathing	2:1[3]	11.6	17.2	22.1	43 (min.)	8
9 mm OSB 2R24/W24	2:1[3]	9.6	14.3	18.2	43 (min.)	8
11 mm OSB 1R24/2F16/W24	2:1[3]	9.9	14.6	18.5	43 (min.)	8

Figure 5-12 - Nominal shear resistance per unit length for seismic and other in-plane loads, for shear walls sheathed with wood structural panels on one side of the wall [17]

These values are based on several tests made in laboratory on walls with the same characteristics expressed on the table.

For example, for a wall sheeted with an 11mm OSB board on just one side, whose board has 2,5m x 1,25m (Aspect ration h:w = 2:1) whose fasteners are number #8 screws spaced by 150mm on the perimeter of the board will have a resistant capacity of 9,9 x 1,25 = 12,375 kN. If more resistance is needed, then the spacing of the screws can be minor or another board can be added if possible.

Finally, this norm presents the following expression to calculate the horizontal deflection:

$$\delta = \frac{2vh^3}{3EA_cb} + \omega_1\omega_2\frac{vh}{\rho G t_{sheathing}} + \omega^{5/4}{}_1\omega_2\omega_3\omega_4\left(\frac{v}{\beta}\right)^2 + \frac{h}{b}\delta_v$$

6

Case Study

6.1. Case study overview

6.1.1. Architeture

The case study refers to two two-story dwellings to be built up in the "Rua Escola da Póvoa" in the Portuguese city of Penafiel.

The houses have a modern architecture based on cubism, resulting in a symmetric geometry of straight lines which formed different sized volumes, where the second story has a 1.5 meters cantilever all around its area.

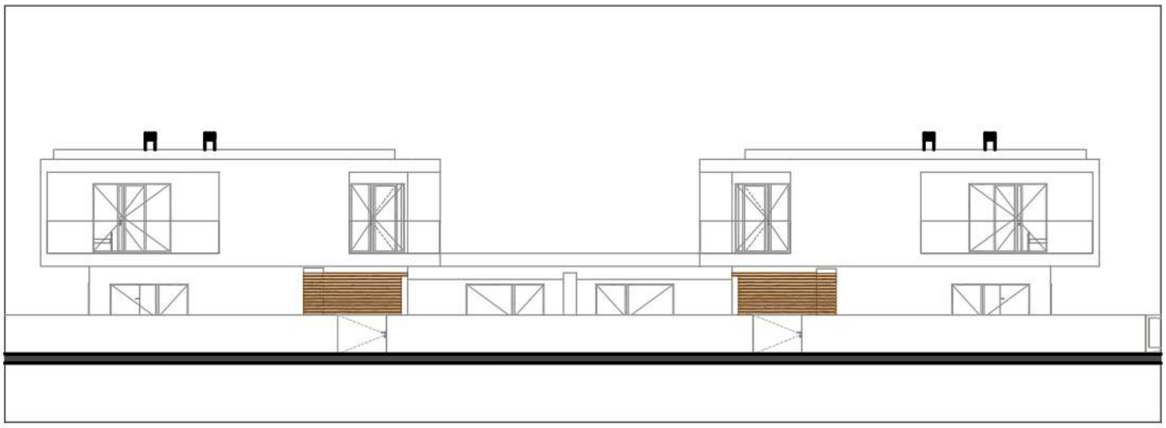


Figure 6-1 - Facade of the dwellings [34]

As it's possible to understand from the plan views below, each habitation is composed by a garage, an office, a kitchen, a small storage, a WC and a living room, in the first floor, while in the second floor there are three suite rooms, each with its own private bathroom, having a total area of 300m^2.

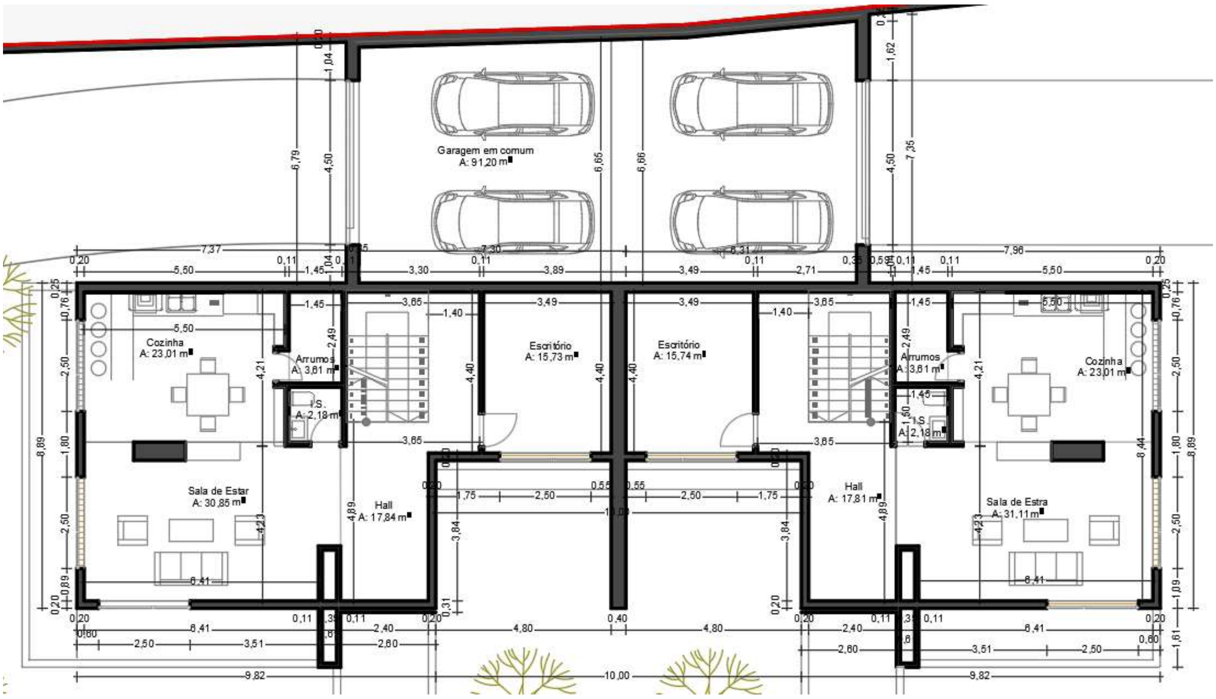


Figure 6-3 - 1st floor plan views [34]

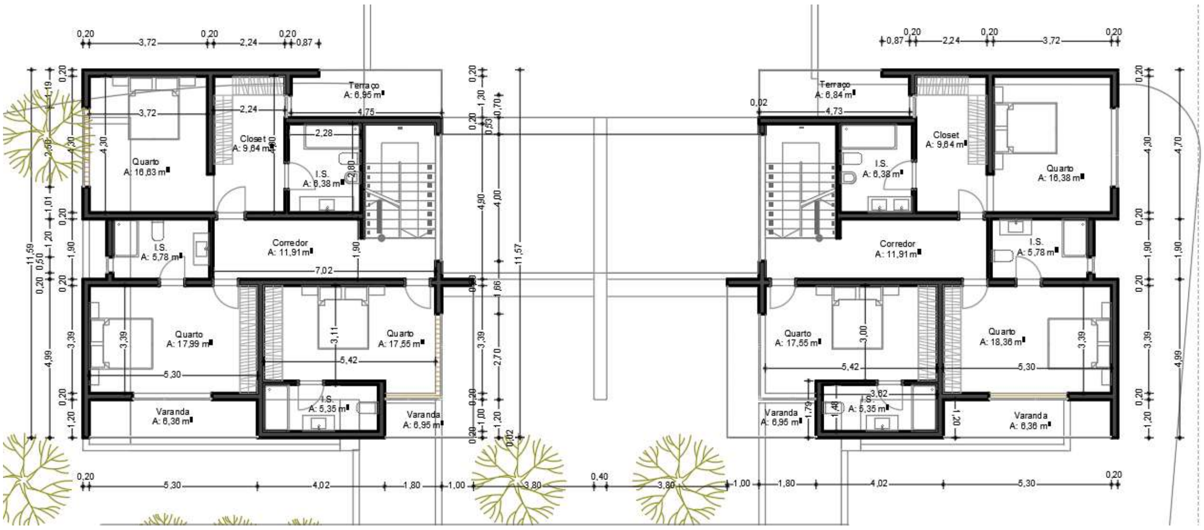


Figure 6-2 - 2nd floor plan views [34]

From the blueprints is also possible to notice that in the first floor there are fewer interior walls than in the second floor, which leads to bigger spans in the floor slab then in the roof slab. Regarding spans, it is also noticeable that the garage beams will have a considerable span of 7 meters.

Furthermore, is possible to have an idea of the thickness of both interior and exterior walls, which shall be around 11 cm and 25 cm respectably. As well, from Figure 6-4 we see that the height of of the garage is 2,60m, while the first floor is at 3,00m and the max height of the building stopping at 6,00 meters.

On the other hand, from all the architecture, it’s noticeable that there are various wide openings in the walls around the house.

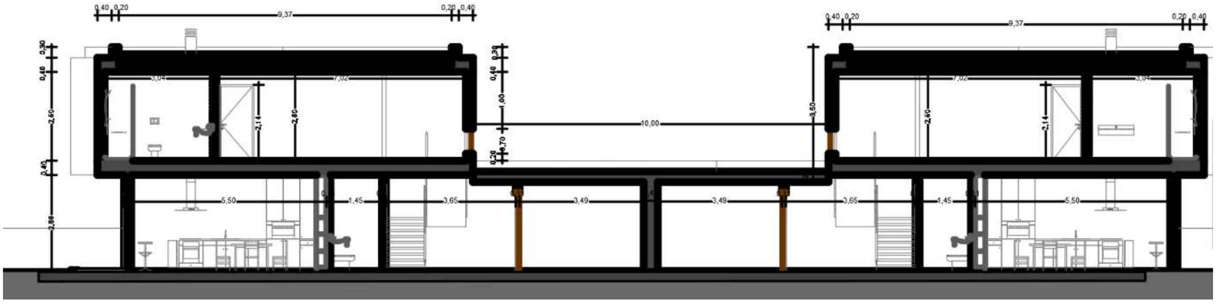


Figure 6-4 - Side View [34]

From client demand, it is supposed for all the structural elements to be in light steel frame due to deadlines limitations, being the foundations and ground floor the only components that are in reinforced concrete.

Any other relevant materials and constructive methods were appointed by the client.

6.1.2. Exterior walls

It is known that the exterior walls need at least 20 cm because of the roller shutter boxes, and that the 60mm Styrofoam, ensuring thermal insulation will be fixed in the exterior, glued to 12mm OSB panels, which are fixed to the frame structure. In the inside of the frame there is 40mm rock wool performing the sound insulation and 9mm OSB panels fixed to the inside of the wall. Finally, there is a 11mm gypsum board as cladding.

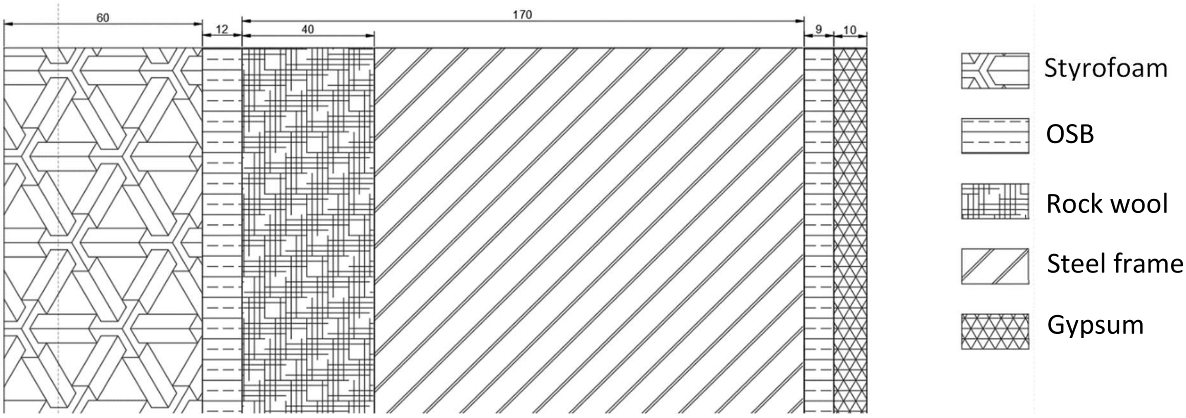


Figure 6-5 - Exterior walls scheme [35]

6.1.3. Interior walls

About the interior walls, this will have a 9mm OSB panel fix to the frame and 11mm gypsum board as cladding in both sides of the profiles, while there is a 40 mm rock wool panel between them.

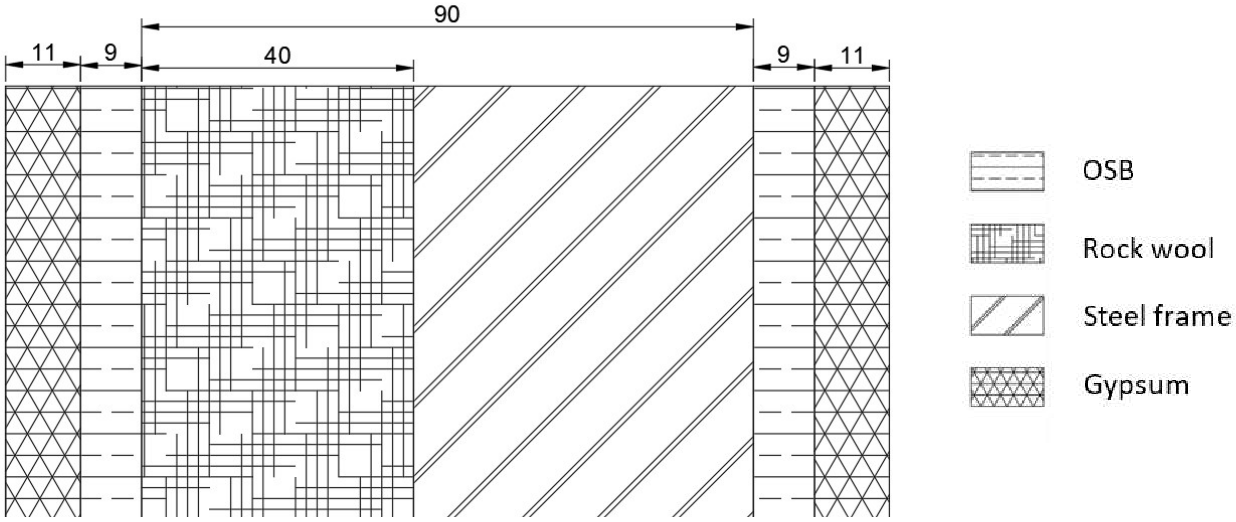


Figure 6-6 - Interior walls scheme [35]

6.1.4. Floor slab

The floor slab will have an 18mm OSB layer above the joists and have the water supply system and electrical system working above it, while the sewage and HVAC systems below, inside a fake ceiling. Covering the pipes above the slab, there will be a 100 mm layer of light concrete and 50 mm layer of mortar, finishing with a layer of parquet or tiles as claddings.

Attached below the beams there will be the fake ceiling which includes a 11mm plasterboard, a 40mm rock wool layer and the fixation structure.

The kind of slab will be placed on the garage, but with an additional final layer of bituminous waterproofing.

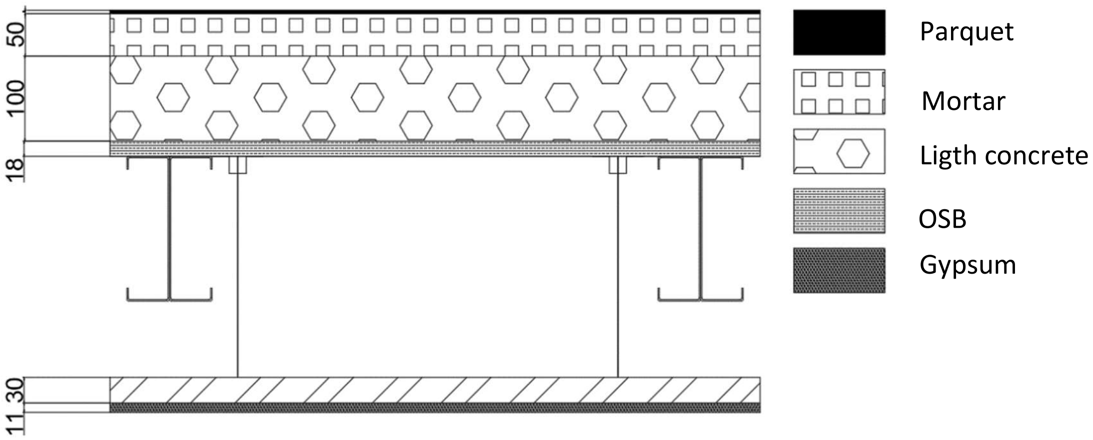


Figure 6-7 - Floor slab scheme [35]

6.1.5. Roof slab

Finally, the roof slab will have a 12mm OSB above joists and a 30mm sandwich panels fixed to perpendicular Z profiles that create a slope.

Below the roof joists there is the same fake ceiling which includes a 11mm plasterboard, a 40mm rock wool layer and the fixation structure.

6.2. Regulatory requirements

The design of the whole framing structure will have in consideration the demands defined in the norms used in structural projects, namely, EC0 (NP EN 1990: 2009, 2009), EC1 (NP EN 1991 – 1-1, 2009 and NP EN 1991 – 1-4, 2005). All safety checks that are required will be based on EC3 (NP EN 1993-1-1: 2005, 2010; NP EN 1993-1-3: 1996, 2009; and NP EN 1993-1-5: 1997, 2009).

- EC0 – Basis of structural design
- EC1 – Actions on structures
- EC3 – Design of steel structures

6.3. Loads

The most relevant actions that will be analyzed and calculated in this case are the permanent actions, overloads and wind loads. Other types of loads related to phenomena such as snow or drastic changes in temperature has no influence on the kind of building and place where is inserted and therefore, not being subjected to any analysis. [22]

The load combinations that will be used are:

Fundamental

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

Characteristic

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

Quasi-permanent

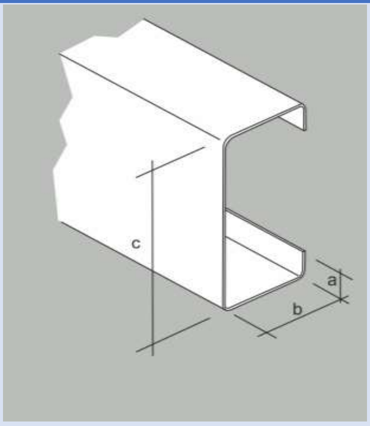
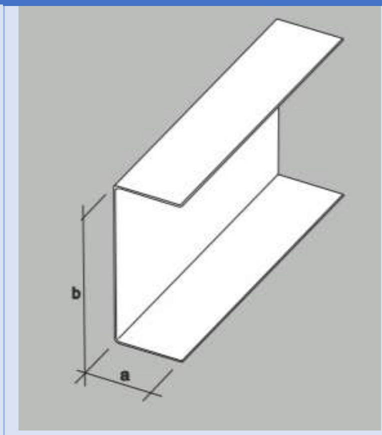
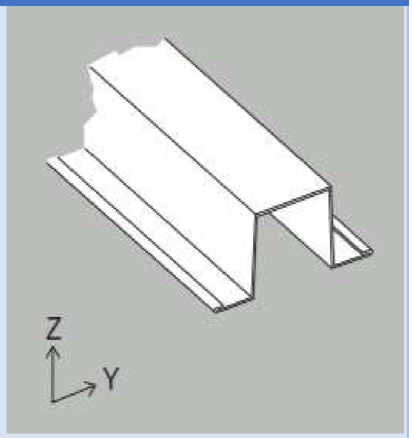
$$\sum_{j\geq 1} G_{kj} + P + \sum_{i\geq 1} \Psi_{20,i} Q_{k,i}$$

6.3.1. Self-weight

The frame self-weight was calculated considering a steel density of 77.01 kN/m^3 and attending to the cross-sections area, and the length of the elements.

The following table displays the most common cross-sections used and their respective area and weight per meter:

Table 6-1 - Weight per meter of most common cross-sections [2]

C170	U170	Hat-140
		
Area = 0,0006 m^2	Area = 0,000405 m^2	Area = 0,000902 m^2
Weight = 0,0462 kN/m	Weight = 0,0312 kN/m	Weight = 0,0695 kN/m

6.3.2. Dead Loads

Adding to the framing structure, there is the imposed dead weight of other materials described in 6.1.2 to 6.1.5.

Table 6-2 – Resume of considered dead loads [23]

	Material	Thickness (mm)	Density – γ(kN/m^3)	Distributed load (kN/m^2)	Total (kN/m^2)
Exterior walls	Styrofoam	60	0,022	0,00132	0,19
	OSB	12	7,00	0,084	
	Gypsum	11	0,76	0,00836	
	Rock wool	40	0,70	0,028	
	OSB	9	7,00	0,063	
	Gypsum	11	0,018	0,00836	
	Gypsum	11	0,76	0,00836	0,17

Interior Walls	OSB	9	7,00	0,063	
	Rock wool	40	0,70	0,028	
	OSB	9	7,00	0,063	
	Gypsum	11	0,76	0,00836	
First floor slab	Parquet	4	16,75	0,067	2,18
	Mortar	50	19,00	0,95	
	Light concrete	100	10,00	1,00	
	OSB	18	7,00	0,126	
	Rock wool	40	0,70	0,028	
	Gypsum	11	0,76	0,00836	
Rooftop slab	Sandwich panel	30	3,43	0,103	0,24
	OSB	15	7,00	0,105	
	Rock wool	40	0,70	0,028	
	Gypsum	11	0,76	0,00836	

6.3.3. Live Loads

The live loads were considered according to EC1 (NP EN 1991-1-1, 2009) – 6.3.1. The building is meant to have domestic and residential usage, being a category A, obtaining from table 6.1 the values defined for overloads on the floors. [3]

For the roofs, section 6.3.4 of EC1 (NP EN 1991-1-1, 2009), the considered loads correspond to categories H and I, defined on tables 6.10 and 6.11 of the norm.

Finally, EC1 part 1 allows that when the exact locations of the partition walls are not known, and the floor has a load transferring capacity, to consider a uniform distributed load on the pavements, 6.3.1.2(8). However, because the internal walls are bearing walls, the exact location is known so it was considered their self-weight acting on them.

Table 6-3 - Resume of considered live loads

Considered loads	kN/m2
Overloads on floors	2,0
Overloads on accessible roofs	2,0
Overloads on non-accessible roofs	0,4

6.3.4. Wind Loads

The selected building is only one of two row houses, but it must be design each house individually, since it is possible that aren't both built at same time. Additionally, this is located near other buildings, so the response when solicited by wind action depends on this variable. If, for one side, the adjacent buildings offer protection, by the other side, it is more difficult to understand and preview the demand that they cause on the structure when under wind load.

The wind loads were calculated using the method applied in the Eurocode EC1-4 (NP EN 1991-1-4, 2010). However, the use of the recommended method is not easy at all to apply, especially because of the irregular shape and geometry with lots of reentrances.

Defining areas for the distribution of pressure coefficients is a lengthy process, with some complexity, particularly in roofing. In the case under study, this aspect becomes even more difficult because the stories are eccentric, having cantilevers, and reentrances all around, doesn't fully fit in any of the situations that the regulation defines.

Considering that there are two roofs at different heights, and that the first floor can be considered as two rectangles, it was adopt a simplification conservative method, where the is one square box for each floor, one on top of the other, like the next scheme shows:

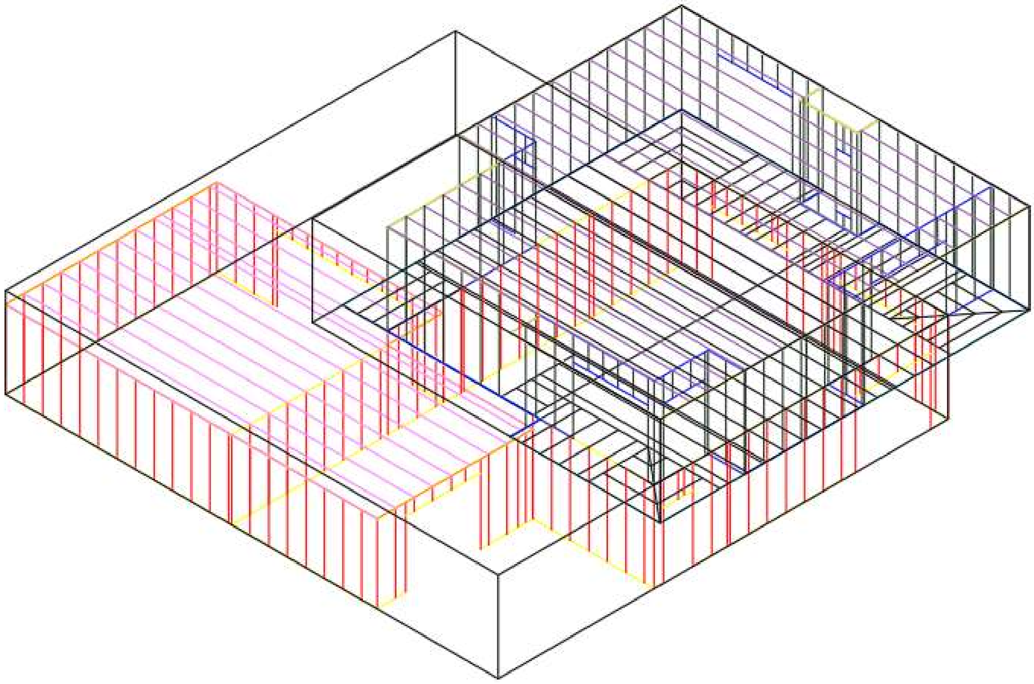


Figure 6-8 - Scheme of simplification model [35]

For the calculation of pressures and their forces, was considered what is prescribed in a note on 7.2.2(3) in EC1-4 (NP EN 1991-1-4, 2010), which considers the simultaneous wind action in the windward and leeward and the lack of correlation between values, therefore advising to multiply the force by a factor that depends on the quotient between height and width.

The considered velocity is 27m/s which corresponds to zone A, with a direction coefficient and season coefficient equal to one. Also, because is a suburban zone, it is classified as category III.

Assuming these factors and calculating all values according to Eurocode, we get a pressure of 713,7 Kg/m^2.

To define the values of pressure, is was considered the wind action with different origins and directions, Figure 6-9 x+, x-, y+, y-.

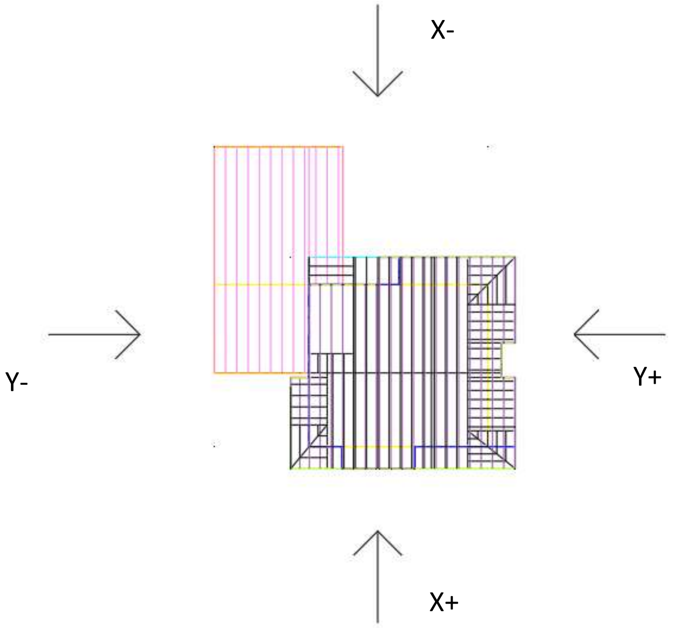


Figure 6-9 - Top view with wind directions [35]

However, the simplifications applied were far from representing reality so, in a attempt to better characterize the wind action and facilitate the calculations, and trying to take advantage of calculation tools offered by the calculation program used, the analysis of the wind loads was also carried out using the ROBOT STRUCTURAL ANALISYS (Autodesk, 2019) wind tunnel simulator. The values obtained using the reference dynamic pressure (qb=713,71N/m2) were in agreement with the ones calculated with the method of Eurocode but with a more coherent distribution.

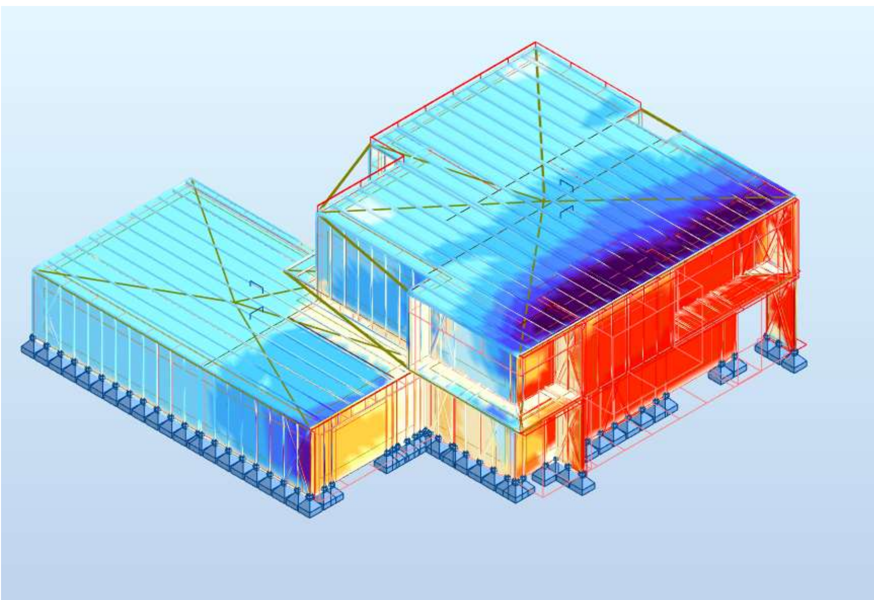


Figure 6-10 - Wind pressure X+ direction [26]

6.3.5. Second order effects

Different possible classifications can be invoked and adopted for structures considering the various aspects of their behavior. In terms of response to lateral actions the following classification can be introduced for the framed structures:

- non-sway structures, in which the influence of vertical loads on lateral displacements is negligible;

- sway structures, in which axial load contribution to lateral displacements is not negligible and stresses in structural members cannot be easily evaluated by considering the axial loads on the undeformed configuration of the frame structure.

The small displacements hypotheses, usually assumed in structural analysis of frames, is carried out by considering all the action applied on the undeformed configuration of the structure, possibly affected by some kinds of imperfections (typically, the lack of verticality) as provided by the relevant Codes of Standards.

The analysis carried out on the undeformed shape is usually called first order analysis and the corresponding stresses neglect the change in eccentricity due to the actual configuration of the frame.

As far as lateral displacements increases, the effects of the eccentricity of vertical loads on the actual value of stresses is more and more relevant and a second order analysis considering loads on the deformed shape is needed for obtaining a careful estimation of the final state of stress of the structure as a whole.

Nevertheless, in non-sway structures the difference of the results stemming out by first order analysis are reasonably close to the ones derived by second order analysis and this condition can be considered satisfied when:

$$\alpha_{cr} = \frac{F_{cr}}{F_{ed}} \geq 10 \quad for\ a\ elastic\ analysis$$

$$\alpha_{cr} = \frac{F_{cr}}{F_{ed}} \geq 15 \quad for\ a\ plastic\ analysis$$

The value of α_{cr} was calculated using the Horne method, present in the Eurocode 3-1-1, given by the following expression and figure:

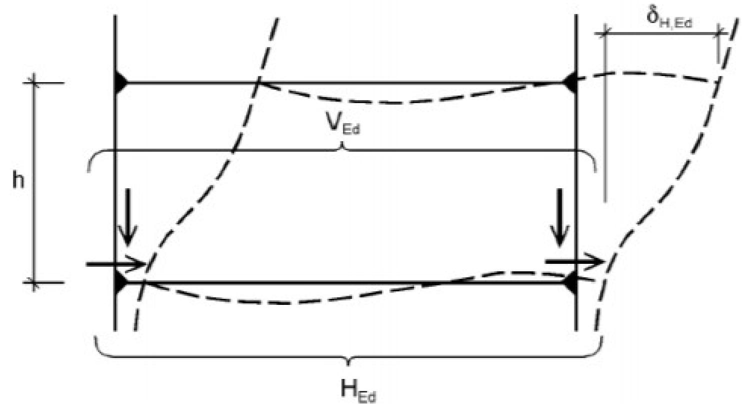


Figure 6-11 - Representation of Horne method [21]

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right)$$

Knowing this, applying all the loads described before in a structural model using the software Autodesk Robot Structural Analysis, we got the following values:

Table 6-4 - Calculation of the critical factor value

		V_{Ed} (kN)	H (m)	H_{Ed} (kN)	$\delta_{H,Ed}$ (m)	α_{cr}
1st floor	Wind X +	847,13	3,0	39,97	0,0032	29,50
	Wind X -	847,13	3,0	39,97	0,0082	17,26
	Wind Y +	847,13	3,0	41,39	0,0039	25,05
	Wind Y -	847,13	3,0	41,39	0,0033	29,6
2nd floor	Wind X +	125.01	3,0	13,18	0,0108	19,52
	Wind X -	125.01	3,0	13,18	0,0033	63,89
	Wind Y +	125.01	3,0	12,41	0,0067	29,63
	Wind Y -	125.01	3,0	12,41	0,0115	17,26

As it is possible to see in the table, in all cases, the value of α_{cr} is bigger then 15, so we can conclude that the structure is a non-sway frame and therefore, it is not needed to proceed with the calculation of second order effects.

6.4. **Modeling**

6.4.1. General

It is extremely important that the model reliably represent the case study as close to reality as possible. The designer's choices made must be taken very cautiously and follow a line of reasoning that allows him to calibrate the model and see if the results obtained are at least plausible or otherwise the calculation method is wrong.

Keeping this in mind and using the calculous software *ROBOT STRUCTURAL ANALYSIS* (Autodesk, 2019) it was possible to model the case study. In this model were only used bar and shell elements, as well as some modeling devices to simplify the structural calculation which are evidenced in the following points.

The first step was to create a CAD file with the disposition of each stud, track and beams. These bar elements were drawn above the architecture CAD file so that the bars represented the center of the walls.

Then the chosen layout was imported to ROBOT software where it was assigned a material and cross-section which center of gravity coincided with the axis of the bar.

Regarding the support conditions, it was considered that the foundations ensured only a simple support while the connection between studs and beams was pinned. This is a conservative simplification since the screws do offer some rigidity connections, but not enough to consider them fixed. It is also understandable that the studs should only have very small bending moments due to wind loads. The maximum value of the bending moment of any element shall be given by:

$$M = \frac{pl^2}{8}$$

Where:

p – uniform distributed load on the element;

l – length of the element

On the other hand, the pavement was model using a shell model which main function was to distribute the loads uniformly by right elements. It also must be understood that this pavement didn't had any rigidity or performed any kind of diaphragm action.

Finally, all the previously calculated loads were applied to the structure and through an iterative process was possible to have the stresses on each element and proceed to do the safety checks.

6.4.2. Modeling the bracings

One of the most difficult elements to model were the shear walls, since it wasn't possible to model a wall or a cladding with the correct stiffness, so instead, it was created a struts and straps model, where basically it was first calculated the board stiffness and then considered two metal elements in X-bracing with the same stiffness which would be added to the model.

So, according to Kechidi [24], the shear resistance of a cold formed steel wood sheathed shear wall panel can be obtained by:

$$P_R = P_s + P_F$$

Where:

P_s is the shear resistance of the board panel, calculate with:

$$P_S = \sum_{i=1}^{2} P_{S,i}$$

P_F is the lateral resistance associated to the studs (framing), given by:

$$P_F = K_F.\mathit{\Delta}$$

This calculation method takes into account the dimensions of the framing structure, the dimensions of the board panel, the mechanical properties of these elements according with their materials, and even the fasteners and their spacing and location.

In the same document, the lateral deformation $\mathit{\Delta}$ is calculated with:

$$\mathit{\Delta} = \frac{P_R}{K_F + K_s}$$

Where K_s is the stiffness of the board panel and K_S is the stiffness of the framing structure.

With these values, and applying a typical state-of-the-practice shear wall model (Figure 6-12), it is possible to model the X-bracing elements with the correct stiffness.

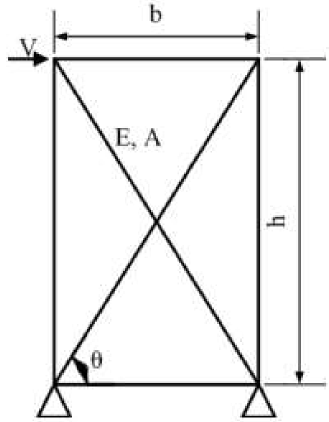


Figure 6-12 - Typical state-of-the-practice shear wall model [24]

The next step is to calculate the axial force and displacement of each diagonal through:

$$F = V/(2\ cos\ \theta)$$

And

$$\delta = \frac{\Delta\ x\ b}{\sqrt{b^2 h^2}}$$

Finally, knowing that the stiffness of the wall diaphragm K_{wall} is:

$$K_{wall} = \frac{F}{\delta}$$

It is possible to know the needed area for each diagonal so that the system as the same stiffness as the real wall diaphragm.

$$K_{wall} = \frac{E.A}{L}\ x\ 2\ <=> A = \frac{K_{wall}\ x\ L}{2E}$$

Where E is young modulus that to use the same material (steel) is 210 GPa and L the length of the diagonal.

6.4.3. Modal analysis

It is reglementary required to analyze every structure under a seismic action, however as LSF structures can be considered composite structures whose rigidity depend on non-steel elements it is extremely difficult to correctly model them. Being so, some conservative simplifications had to be done.

Eurocode 8 establishes for buildings with heights up to 40 m the value of T (period) may be approximated by the following expression:

$$T_1 = C_t \times H^{\frac{3}{4}}$$

With C_1 = 0,05 and knowing that the height of the building is 6 meters, the natural frequency should be around 0,19s. [25]

Considering only the shear walls designed to resist the horizontal loads, the first mode of vibration given by the software had a 0,198s which was, as expected, a global mode of vibration.

It was also noticeable that the main vibration modes were translational in the X direction but there were also some local vibration modes, however this aren't realistic since all the framing system will be covered with several layers of OSB and other claddings which will brace all the frame structure and won't allow this local modes.

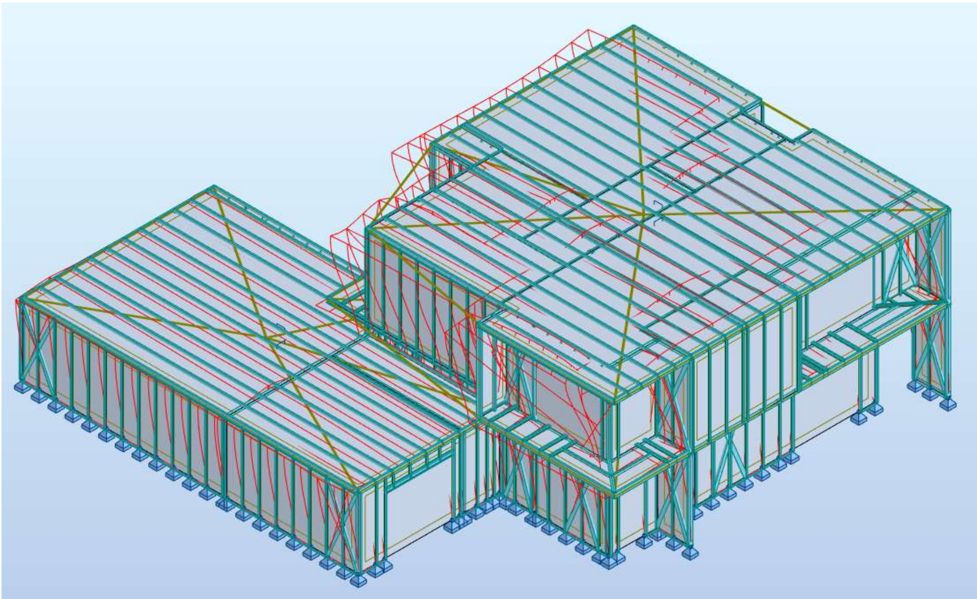


Figure 6-13 - First mode of vibration [26]

Attending to this, and knowing all the previous mentioned layers that constitute the walls it is understandable that the structure will be much more rigid then what was calculated, nevertheless this analysis allowed to ensure an natural frequency of vibration of 5Hz.

7

Design checks

In the first place, to proceed with the safety checks, the effective properties of the cross-section were calculated according to Eurocode 3, as explained in 0, whose results can be found in annex 1.

7.1. Vertical Elements

Since the stud's connections are pinned, these elements will be mainly subjected to axial force due to gravity loads. However, there is still some small bending moments due to wind action and due to the displacement of the neutral axis in class 4 cross-sections.

This bending moment is given by:

$$\Delta M_{i,Ed} = e_{N,i} \times N_{Ed}$$

Where N_{Ed} is the axial force and $e_{N,I}$ is the distance between the centers of gravity of the effective area and the gross cross-section area, when subjected to compression loads only. Nevertheless, these stresses will be very small when compared to axial force.

More is known from 6.1.2, that the height of the web of the studs was pre-defined by the thickness of the walls in the architecture. It was needed at least 170mm for blinders boxes so this is the minimum height of the web for the exterior walls' studs. As for the interior walls, since there were practically no restrictions, it was used the smallest C-section that the supplier's catalogue, which was 90mm.

Being so, the studs were divided in four groups, (first floor exterior walls, first floor interior walls, second floor exterior walls and second floor interior walls), and then, admitting a cross-section for each group it was check all the design requirements.

At the beginning it was considered one C-section stud for each vertical element (bar) and when there was an element which didn't fulfill the needs, there were added more profiles until it was stable.

Whenever it was needed more than one C-profile, these should be connected in a back-to-back shape, however, it is known that, for constructive reasons, there were some cases where this wasn't possible, such as the king studs in windows, that should be in a front-to-front shape.

Knowing this, to take in consideration the weight of the second-floor framing loads, these were the first elements to be designed.

7.1.1. Second floor exterior walls

As there are many interior walls on the second floor supporting the rooftop beams, there weren't much demand on these studs. The most demanded stud for the worst combination had the following stresses:

Table 7-1 - Second floor exterior studs stresses [26]

Stresses	Value
N_{Ed} (kN)	14,4320
$M_{Ed,y}$ (kN.m)	0,0555
$M_{Ed,y}$ (kN.m)	0,0013
$\Delta M_{Ed,y}$ (kN.m)	0,0000
$\Delta M_{Ed,y}$ (kN.m)	0,0971

Attending that the studs were simple C170 cross-section with 3 meter high and spaced at maximum of 0,625 cm, all the safety checks were fulfilled by a large margin as it is resumed in Table 7-2 - Second floor exterior studs safety checks Table 7-2.

Table 7-2 - Second floor exterior studs safety checks

Safety Checks	Value	Verification
$\frac{N_{Ed}}{N_{t,Rd}}$	0,11	<1 Ok
$\frac{N_{Ed}}{N_{c,Rd}}$	0,16	<1 Ok
$\frac{N_{Ed}}{N_{by,Rd}}$	0,18	<1 Ok
$\frac{N_{Ed}}{N_{bz,Rd}}$	0,71	<1 Ok
$\frac{N_{Ed}}{N_{b,FT,Rd}}$	0,55	<1 Ok
$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z,Ed}+\Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}}$	0,35	<1 Ok
$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z,Ed}+\Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}}$	0,70	<1 Ok

7.1.2. Second floor interior walls

Relatively to the interior walls, using a single C90 cross-section, it was possible to guarantee the safety to all studs with some margin since there were many elements to support the loads from the rooftop, that as was mentioned before are very small.

The most demanded stud was under the following stresses:

Table 7-3 - Second floor interior studs' stresses [26]

Stresses	Value
N_{Ed} (kN)	8,180
$M_{Ed,y}$ (kN.m)	0,420
$M_{Ed,y}$ (kN.m)	0,150
$\Delta M_{Ed,y}$ (kN.m)	0,000
$\Delta M_{Ed,y}$ (kN.m)	0,022

With these values wholly under the resistant limits, all safety requirements were fulfilled as its possible to see in Table 7-4 .

Table 7-4 - Second floor interior studs safety verifications

Safety Checks	Value	Verification
$\frac{N_{Ed}}{N_{t,Rd}}$	0,08	<1 Ok
$\frac{N_{Ed}}{N_{c,Rd}}$	0,09	<1 Ok
$\frac{N_{Ed}}{N_{by,Rd}}$	0,15	<1 Ok
$\frac{N_{Ed}}{N_{bz,Rd}}$	0,42	<1 Ok
$\frac{N_{Ed}}{N_{b,FT,Rd}}$	0,30	<1 Ok
$\frac{N_{Ed}}{\frac{\chi_Y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z,Ed}+\Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}}$	0,65	<1 Ok
$\frac{N_{Ed}}{\frac{\chi_Z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z,Ed}+\Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}}$	0,76	<1 Ok

7.1.3. First floor exterior walls

Regarding the first floor exterior studs, because these support all the second floor studs plus the first floor slab which has way more loads inherent than the rooftop, its obvious that the demand will be much higher and so, in some cases a different cross-section, or an increased number of elements will be necessary.

Being so, the first step to design these elements was to identify the maximum resistant capacity of a single C170 profile and for any bar which had a bigger demand, then another element would be added.

Also, during the design it was notice that some elements would need even three C170 profiles to fulfill the requirements.

Being so, the resistant properties of a single, double and triple C170 element with 3 meter high, are:

Table 7-5 - C170 studs resistant properties according to number of profiles

C170	Single	Double	Triple
$N_{t,Rd}$ (kN)	138,033	276,066	414,099
$N_{c,Rd}$ (kN)	90,241	180,482	270,723
$N_{by,Rd}$ (kN)	80,865	161,730	242,595
$N_{bz,Rd}$ (kN)	20,235	40,471	60,707
$N_{b,FT,Rd}$ (kN)	26,303	52,607	78,910

Nevertheless, the combined action of axial loading and bending moment must also comply within the limits specified in 5.5.5, yet it was considered that any element with more than 20kN of axial compression would have a double section and any with more than 40kN would be a triple section.

Table 7-6 - Triple section safety verifications

Safety Checks	Value	Verification
$\frac{N_{Ed}}{N_{t,Rd}}$	0,11	<1 Ok
$\frac{N_{Ed}}{N_{c,Rd}}$	0,17	<1 Ok
$\frac{N_{Ed}}{N_{by,Rd}}$	0,19	<1 Ok
$\frac{N_{Ed}}{N_{bz,Rd}}$	0,75	<1 Ok
$\frac{N_{Ed}}{N_{b,FT,Rd}}$	0,58	<1 Ok
$\frac{N_{Ed}}{\frac{\chi_Y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{Z,Ed}+\Delta M_{z,Ed}}{\frac{M_{Z,Rk}}{\gamma_{M1}}}$	0,40	<1 Ok
$\frac{N_{Ed}}{\frac{\chi_Z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{Z,Ed}+\Delta M_{z,Ed}}{\frac{M_{Z,Rk}}{\gamma_{M1}}}$	0,75	<1 Ok

According to this, considering that there are 163 vertical elements, from which 142 are singles, 20 are doubles and 1 triple sections.

7.1.4. First floor interior walls

The first-floor interior studs have been designed using the same approach as the exterior studs, but assuming the same 90mm web used in the second floor. Although this walls still support a considerable load from the second floor, they present a very good behavior to compression since they have a participation ratio of the effective area of the cross-section of 86%, which is much better than the 63% offered by the C170 cross-section.

With a single C90 section, and considering a 3 meter high stud, it is possible to achieve the following resistant properties:

Table 7-7 - C90 studs resistant properties

C90	Single
$N_{t,Rd}$ (kN)	100,473
$N_{c,Rd}$ (kN)	87,223
$N_{by,Rd}$ (kN)	55,402
$N_{bz,Rd}$ (kN)	19,520
$N_{b,FT,Rd}$ (kN)	27,532

According to this properties, any element with more than 19kN of axial compression would need to be a double C90 section, yet while analyzing the results obtained from the software, there was only one element which need a double section since it had 23kN of axial force.

7.2. Horizontal Elements

As exposed before, there are three main horizontal slabs to design, the floor slab, the rooftop slab and the garage slab. For constructive reasons. in all these cases, it was always tried to use profiles with the same cross-section, which were C170 profiles or built-up sections using these same profiles.

In the floor slab, because there are few interior walls, it was adopted a beam parallel to the façade which will support joist perpendicular to the façade. These joists were allocated by admitting a 0,625m spacing between them and adding some in the needed places like the staircase. Also, because there is a cantilever all around the first floor, the most extreme joists will have smaller perpendicular joists attached to them, being the final configuration shown in Figure 7-1.

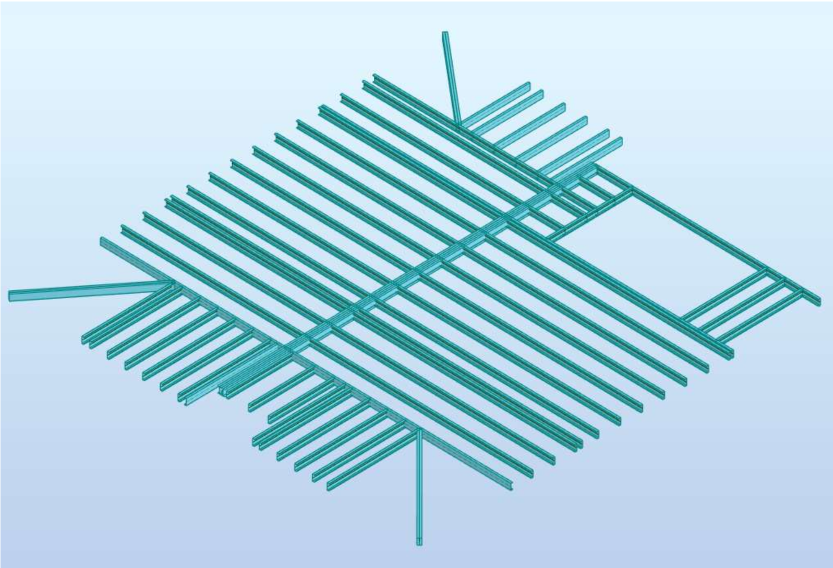


Figure 7-1 - Final configuration of floor slab [26]

For the rooftop slab, as there were many interior walls, there weren't any major constraints or difficulties designing, being the joists spaced by 0,625m for constructive reasons obtaining the following configuration.

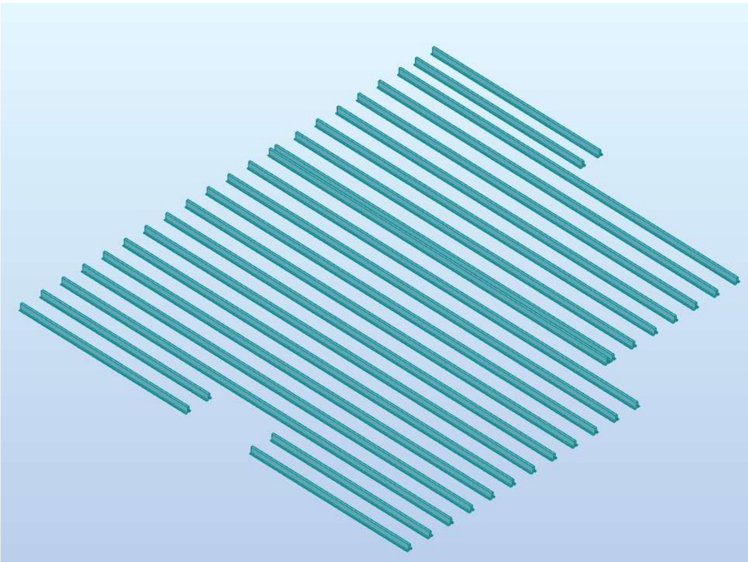


Figure 7-2 - Final configuration of rooftop slab [26]

Lastly, the garage roof slab had especially big span of 7,55m, however this part of the building has a different floor height from the rest of the building, so it is possible to have cross-section with a highest webs which can meet the necessary requirements, without changing the exterior architecture.

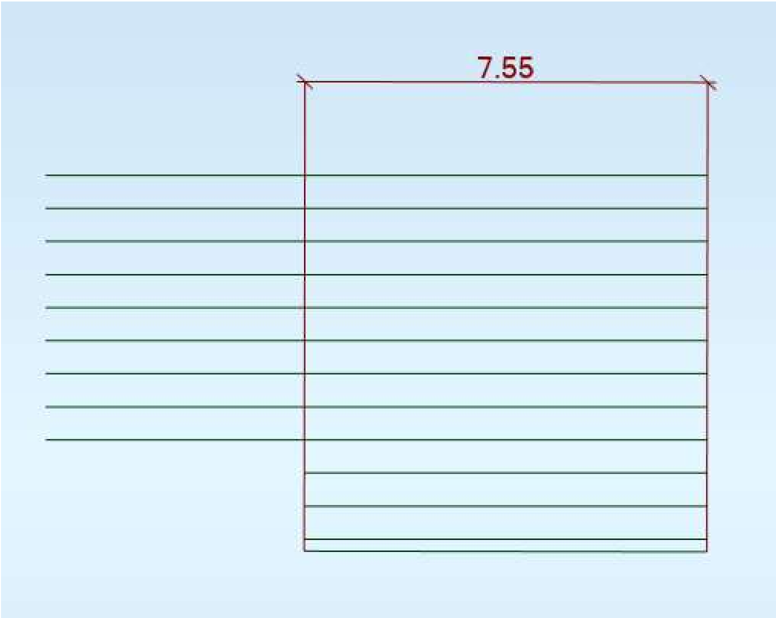


Figure 7-3 - Garage roof slab configuration [26]

7.2.1. First floor main beam

In order to reduce the joists span and trying to take advantage of the few interior walls present in the first floor, it was placed a beam parallel to the façade, in the separation of the living room and the kitchen. In this way, this beam will be supported for the most of its length, being the biggest span of 2,45m.

Since the floor joist will unload on the exterior walls and on this beam, it is perceptible that this will be one of the most important, if not even the most important element on the structure, having considerable shear forces but, lower bending moments, due to the minor spans.

Figure 7-4 - Main beam maximum shear force [26]

Figure 7-4 shows the demand on the beam for the worst combination, which was overloads as base variable action plus wind in Y- direction.

Table 7-8 - Stresses on the main beam [26]

Stresses	Value
V_{Ed} (kN)	41,25
$M_{Ed,y}$ (kN.m)	-15,48
F_{Ed} (kN)	25,26

Since it was desired to have a flat slab, these elements were design to be constructed using a balloon frame system, this is the joists to be fixed in the side of the beam instead of being placed above it.

Being so, the cross-section chosen was a built-up with two front-to-front C170 profiles and two U170 fixed the C section's by the webs, like the Figure 7-5 represents. It was also considered a perfect connection between these elements.

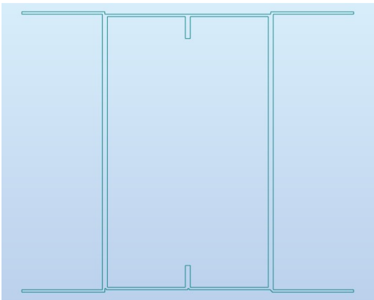


Figure 7-5 - Main beam cross-section and section properties [26]

Considering this, the cross-section didn't verify safety, being the critical check the combined bending moment and support reaction which combined ratios were higher than 1,25 specified in Eurocode 3-1-3, so the thickness of the section was increased to 3,0 mm, obtaining the following properties.

Table 7-9 - Main beam cross-section properties

Properties	Values
Area (mm^2)	1692.00
I_Y (mm^4)	7205573.400
I_z(mm^4)	460729.080
$W_{el,Y}$ (mm^3)	84813.120
$W_{el,z}$ (mm^3)	12331.856

Table 7-10 - Main beam resistant properties

Resistant properties	Values
$M_{cy,Rd}$ (kN.m)	24,516
$M_{cz,Rd}$ (kN.m)	3,254
$V_{b,Rd}$ (kN)	91,768
$R_{w,Rd}$ (kN)	19,758
$M_{b,Rd}$ (kN)	16,841

With this cross-section, it was possible to achieve the following resistant properties, which, as it is resumed in Table 7-11, are enough to verify all the safety requirements.

Table 7-11 - Main beam safety checks

Safety Checks	Value	Verification
$\frac{M_{y,Ed}}{M_{cy,Rd}} + \frac{M_{z,Ed}}{M_{cz,Rd}}$	0,63	<1 Ok
$\frac{V_{Ed}}{V_{b,Rd}}$	0,45	<1 Ok
$\frac{F_{Ed}}{R_{w,Rd}}$	0,61	<1 Ok
$\frac{N_{Ed}}{N_{t,Rd}} + \frac{M_{y,Ed}}{M_{cy,Rd}} + \frac{M_{z,Ed}}{M_{cz,Rd}}$	0,63	<1 Ok
$\frac{N_{Ed}}{N_{c,Rd}} + \frac{M_{y,Ed} + \Delta M_{y,Ed}}{M_{cy,Rd}} + \frac{M_{z,Ed} + \Delta M_{z,Ed}}{M_{cz,Rd}}$	0,62	<1 Ok
$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y,Rd}} + (1 - \frac{M_{f,Rd}}{M_{pl,Rd}})(\frac{2\,V_{Ed}}{V_{w,Rd}} - 1)^2$	No interaction between Shear and bending	Ok
$\frac{M_{Ed}}{M_{c,Rd}} + \frac{F_{Ed}}{R_{w,Rd}}$	1,24	<1,25 Ok
$\frac{M_{Ed}}{M_{b,Rd}}$	0,92	<1 Ok

As well, the deflection was under the limit of L/250, since the largest span was 2,45m the δ_{max} was 9,8mm and as is possible to see in Figure 7-6, for the worst serviceability combination, the maximum deflection was 2,5mm.

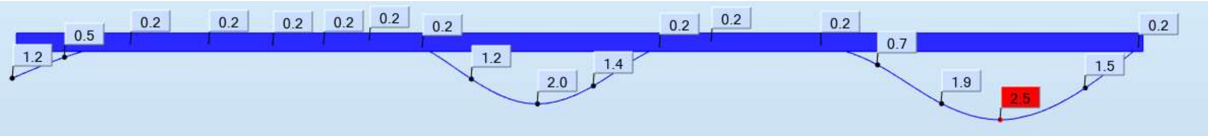


Figure 7-6 - Main beam maximum deflection [26]

7.2.2. First floor corner beams

The highlighted beams in Figure 7-7 are necessary to create the corners in cantilevers around the first floor. These beams, because of their large span in cantilever (2,10m) and considerable acting forces also need a special attention in design.

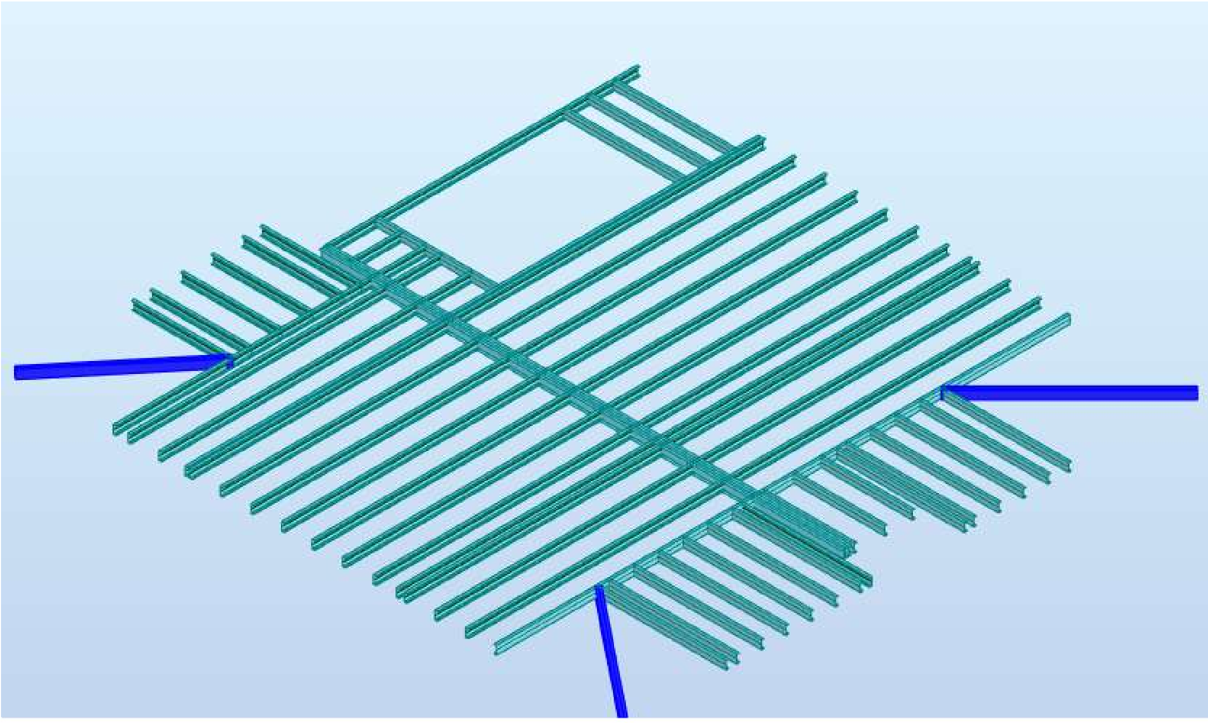


Figure 7-7 -First floor corner beams [26]

Considering a C170 section, was noticeable that in one of the beams, beyond the significant stresses, there were also very big deflections, so it was tried some different cross-sections including a C170 front-to-front built-up cross-section, with 1,5mm and a 3,0mm thickness, and a triple C170 section (front-to-front and a front-to-back) with 1,5mm and 3,0 mm thickness, and although it was possible to verify the safety to all necessary design verifications, however, the deflection was still to high. As it is possible to see in Figure 7-8, the deflection was 23,9 mm while the maximum according to Eurocode to serviceability states was L/250 and because this is a cantilever, L represents twice the real length of the span, so $\delta_{max} = (2100*2)/250 = 16{,}8$mm.

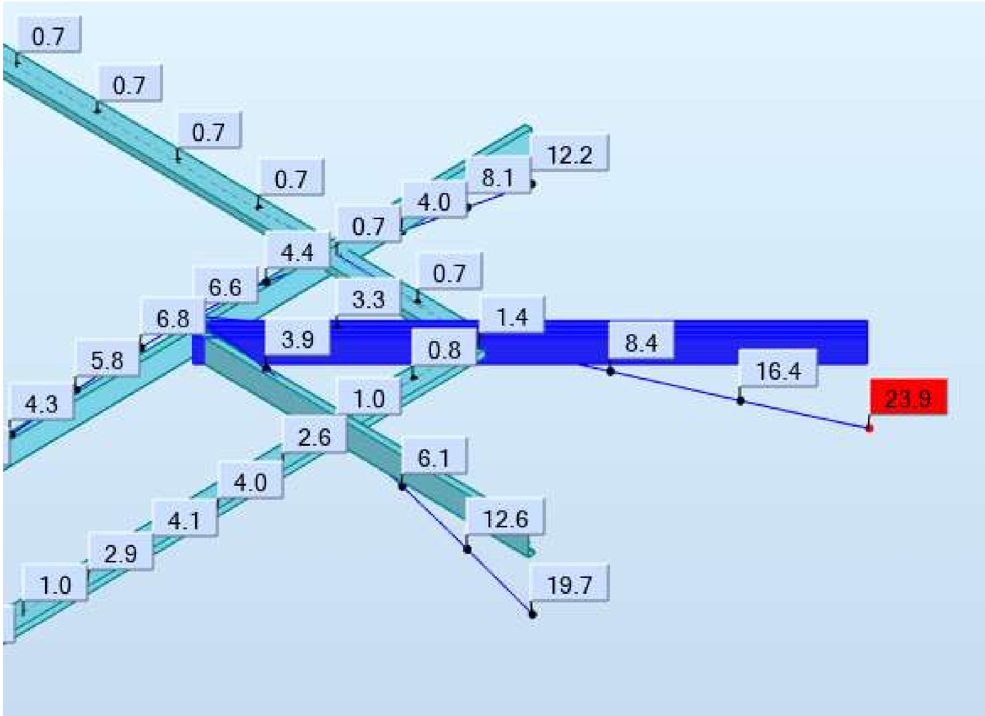


Figure 7-8 - Deflection of the corner beam without support [26]

In order to resolve this problem, the solution found was to add support column on the extremity of the beam. Of course, this solution would need to be discussed with the architect to understand its viability, but for the case in study, looked like a possible solution and did in fact decrease the deflection, while only needing a C170 back-to-back built-up cross-section to fulfill the design requirements.

With this solution, the actuating stresses were:

Table 7-12 - Stresses on corner beam [26]

Stresses	Value
V_{Ed} (kN)	18,94
$M_{Ed,y}$ (kN.m)	10,73
F_{Ed} (kN)	25,4

It was because of the also significant support reaction that the cross-section chosen was a back-to-back and not a front-to-front, since the selected configuration had a better resistance to web crippling. TTable 7-13 shows the cross-section properties, while Table 7-14 shows the resistant properties.

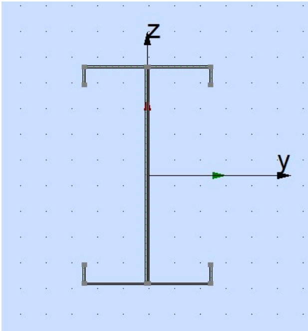


Figure 7-9 - C170 Back-to-back cross-section [26]

Table 7-13 - Corner beam, cross-section properties

Properties	Values
Area (mm2)	882.000
I_y (mm4)	3771637.820
I_z (mm4)	287085.320
$W_{el,y}$ (mm3)	44767.220
$W_{el,z}$ (mm3)	8019.876

Table 7-14 - Corner beam resistant properties

Resistant properties	Values
$M_{cy,Rd}$ (kN.m)	13,032
$M_{cz,Rd}$ (kN.m)	2,504
$V_{b,Rd}$ (kN)	91,768
$R_{w,Rd}$ (kN)	52,617
$M_{b,Rd}$ (kN)	10,821

With this cross-section, it was possible to achieve the following resistant properties, which, as it is resumed in Table 7-11, are enough to verify all the safety requirements.

Table 7-15 - Corner beam safety checks

Safety Checks	Value	Verification
$\frac{M_{y,Ed}}{M_{cy,Rd}} + \frac{M_{z,Ed}}{M_{cz,Rd}}$	0,82	<1 Ok
$\frac{V_{Ed}}{V_{b,Rd}}$	0,21	<1 Ok
$\frac{F_{Ed}}{R_{w,Rd}}$	0,48	<1 Ok
$\frac{N_{Ed}}{N_{t,Rd}} + \frac{M_{y,Ed}}{M_{cy,Rd}} + \frac{M_{z,Ed}}{M_{cz,Rd}}$	0,82	<1 Ok
$\frac{N_{Ed}}{N_{c,Rd}} + \frac{M_{y,Ed} + \Delta M_{y,Ed}}{M_{cy,Rd}} + \frac{M_{z,Ed} + \Delta M_{z,Ed}}{M_{cz,Rd}}$	0,83	<1 Ok

$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y,Rd}} + (1 - \frac{M_{f,Rd}}{M_{pl,Rd}})(\frac{2\,V_{Ed}}{V_{w,Rd}} - 1)^2$	No interaction between Shear and bending	Ok
$\frac{M_{Ed}}{M_{c,Rd}} + \frac{F_{Ed}}{R_{w,Rd}}$	1,24	<1,25 Ok
$\frac{M_{Ed}}{M_{b,Rd}}$	0,98	<1 Ok

Finally, since now this beam is no longer in cantilever but simply supported, the maximum deflection is L/250 = 8,4 mm, and, as it is perceptible from **Erro! A origem da referência não foi encontrada.**, the maximum deflection existing is 2,9mm, so all the design requirements are fulfilled.

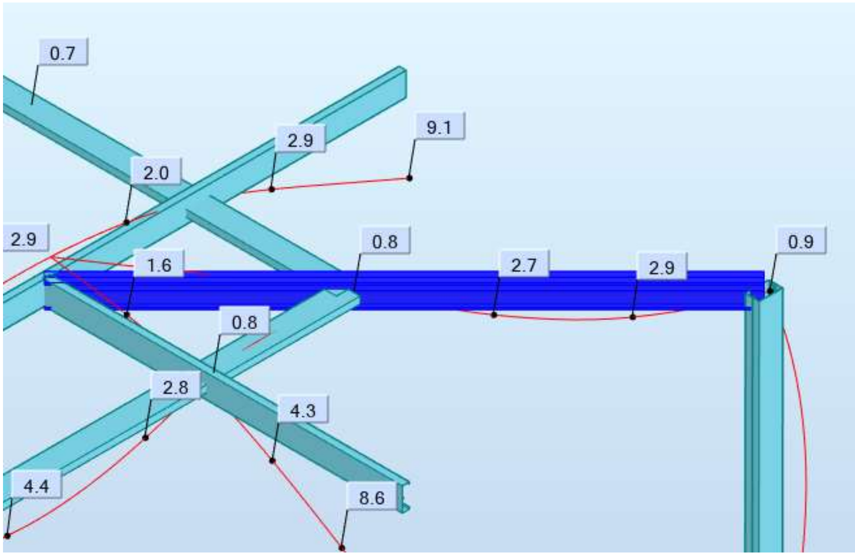


Figure 7-10 - Corner beam deflection with column support [26]

7.2.3. First floor joists

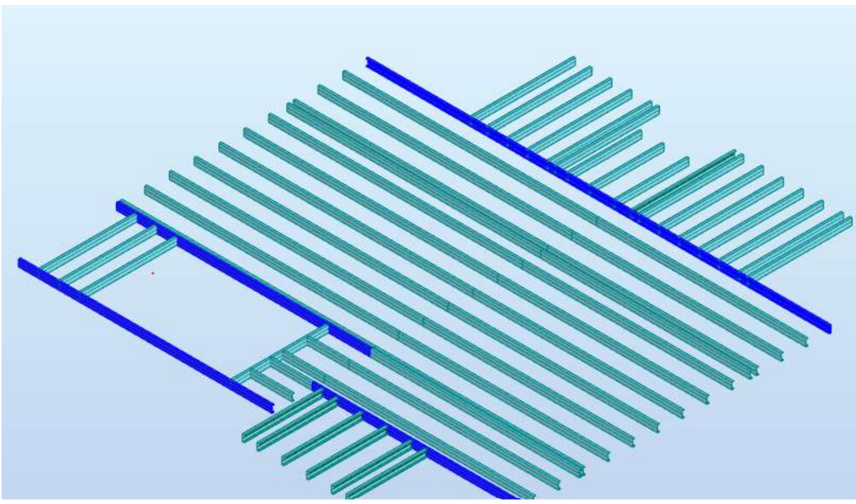


Figure 7-11 - First floor joists [26]

Proceeding now to all the other joists design, and trying to have only one cross-section that satisfies all the requirements to all remaining elements, it was needed a C170 back-to-back built-up cross-section, being that the most solicited joist had the following actuating stresses:

Table 7-16 - Stresses on most solicited joist [26]

Stresses	Value
V_{Ed} (kN)	8,04
$M_{Ed,y}$ (kN.m)	-8,71
F_{Ed} (kN)	17,77

For constructive facilitation, the joists highlighted in Figure 7-11 should have a C170 front-to-front built-up cross-section with a U170 fixed to one of the webs, in order to provide support to the perpendicular joists in cantilever.

Since this is the same cross-section used in 7.2.2, it has the same resistant properties, described in Table 7-13 and Table 7-14. This way, it is possible to maintain the same cross-section for almost all slab, facilitating the construction process.

In the other hand, because the lateral-torsional buckling resistance depends on the joist span, this had to be recalculated. Since these joists had a maximum span of 4,85m, the lateral-torsional buckling resistance was too low, so the adopted solution was to add one of the two, bridging or blocking in the middle of the span, like in Figure 7-12.

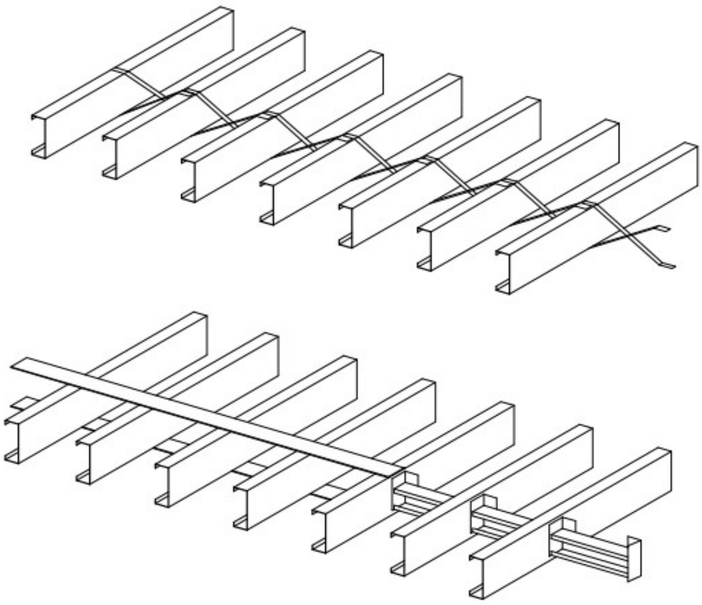


Figure 7-12 - Generic bridging and blocking details [19]

This way, by reducing the buckling length by half, the resistance increased to $M_{b,Rd}$ = 10,254 kN.m. As Table 7-17 shows, all the design requirements were fulfilled.

Table 7-17 - First floor joists safety checks

Safety Checks	Value	Verification
$\frac{M_{y,Ed}}{M_{cy,Rd}} + \frac{M_{z,Ed}}{M_{cz,Rd}}$	0,62	<1 Ok
$\frac{V_{Ed}}{V_{b,Rd}}$	0,09	<1 Ok
$\frac{F_{Ed}}{R_{w,Rd}}$	0,34	<1 Ok
$\frac{N_{Ed}}{N_{t,Rd}} + \frac{M_{y,Ed}}{M_{cy,Rd}} + \frac{M_{z,Ed}}{M_{cz,Rd}}$	0,62	<1 Ok
$\frac{N_{Ed}}{N_{c,Rd}} + \frac{M_{y,Ed} + \Delta M_{y,Ed}}{M_{cy,Rd}} + \frac{M_{z,Ed} + \Delta M_{z,Ed}}{M_{cz,Rd}}$	0,82	<1 Ok
$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y,Rd}} + (1 - \frac{M_{f,Rd}}{M_{pl,Rd}})(\frac{2\,V_{Ed}}{V_{w,Rd}} - 1)^2$	No interaction between Shear and bending	Ok
$\frac{M_{Ed}}{M_{c,Rd}} + \frac{F_{Ed}}{R_{w,Rd}}$	1,01	<1,25 Ok
$\frac{M_{Ed}}{M_{b,Rd}}$	0,85	<1 Ok

Finally, the serviceability states were considered validated since the maximum deflection for middle simply supported joists was δ_{max} = 19,4mm and for elements in cantilever δ_{max}=12,0mm and as it is possible to see in Figure 7-13 the biggest deflection value was 10,0mm.

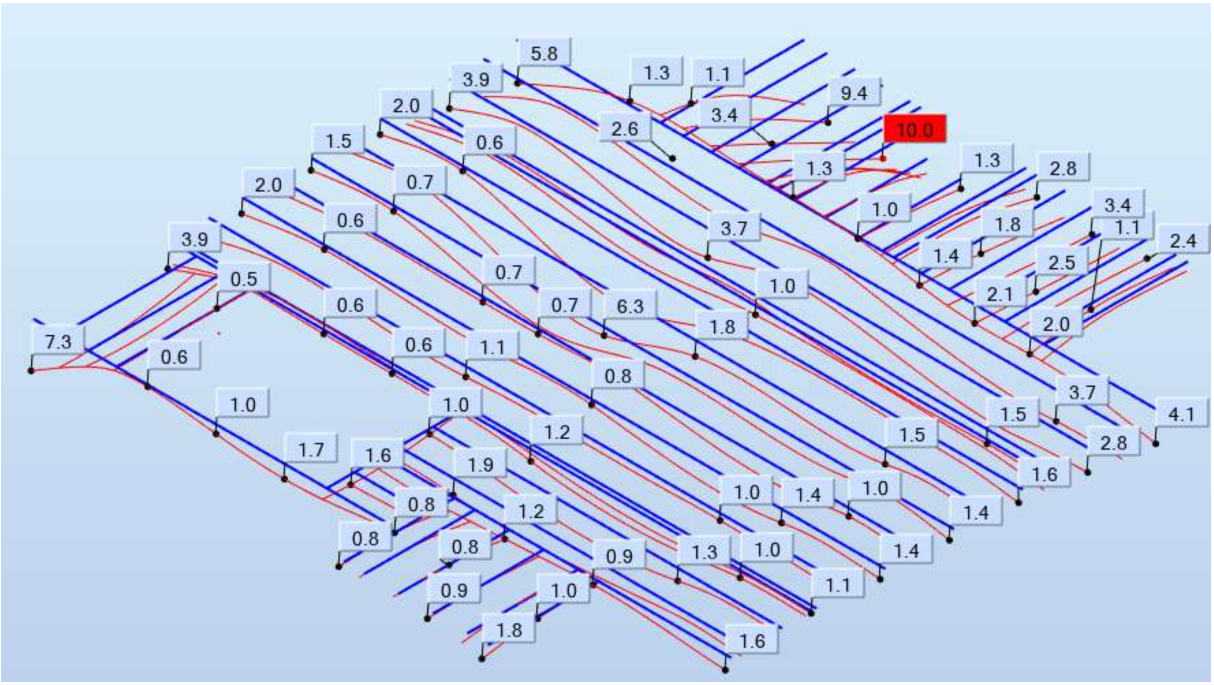


Figure 7-13 - First floor joists deflections [26]

7.2.4. Rooftop beams

Since the rooftop slab only has beams in one direction, it is simpler and easier to design as a whole structure.

Firstly, considering the same C170 cross-section, while verifying the stability for the most solicited beam, it was noticed that this cross-section didn't had the required lateral-torsion resistance, so since all the other verification were complied with some margin, and trying to have a slender slab for architectural purposes, instead of using the same C170 section with built-ups, it was opted to use a 140 high hat-section(Figure 7-14). This cross-section properties are exposed in Table 7-18 and Table 7-19.

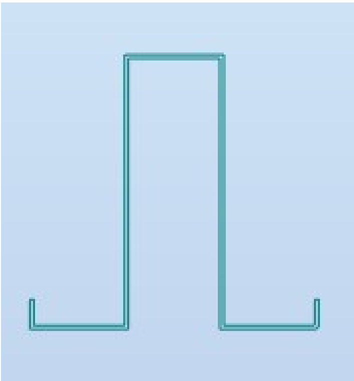


Figure 7-14 - Hat-section [26]

Table 7-18 - Hat section properties

Properties	Values
Area (mm^2)	902.000
I_Y (mm^4)	2385475.000
I_z(mm^4)	1141045.000
$W_{el,Y}$ (mm^3)	29588.060
$W_{el,z}$ (mm^3)	15684.470

Table 7-19 - Hat-section beam resistant properties

Resistant properties	Values
$M_{cy,Rd}$ (kN.m)	8,910
$M_{cz,Rd}$ (kN.m)	4,909
$V_{b,Rd}$ (kN)	101,662
$R_{w,Rd}$ (kN)	33,652
$M_{b,Rd}$ (kN)	6,649

With this cross-section, the most critical beam which had 4,70m span, the actuating stresses were:

Table 7-20 - Stresses on the most solicited rooftop beam [26]

Stresses	Value
V_{Ed} (kN)	2,25
$M_{Ed,y}$ (kN.m)	-2,71
F_{Ed} (kN)	4,49

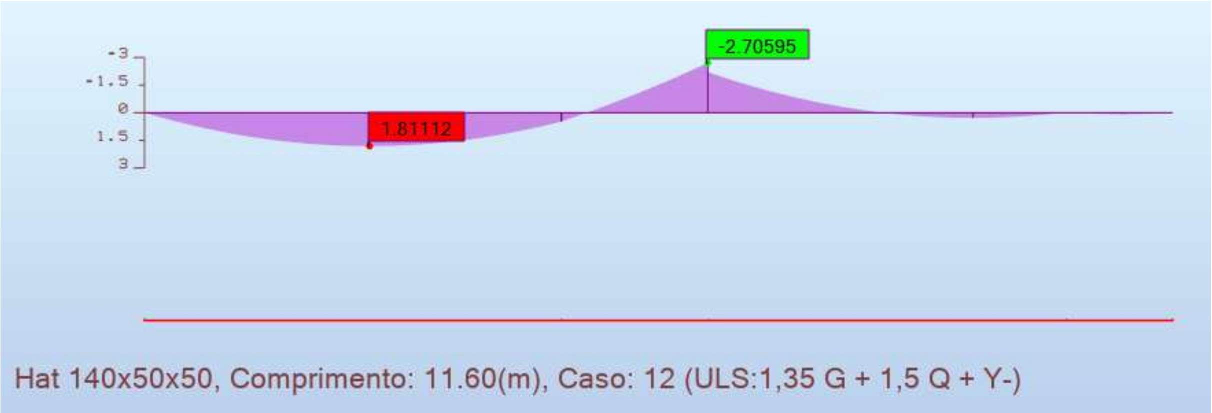


Figure 7-15 - Worst rooftop beam bending moments diagram [26]

Knowing these values, all the required verifications were fulfilled as it is resumed on Table 7-21.

Table 7-21 - Rooftop beam safety checks

Safety Checks	Value	Verification
$\frac{M_{y,Ed}}{M_{cy,Rd}} + \frac{M_{z,Ed}}{M_{cz,Rd}}$	0,31	<1 Ok
$\frac{V_{Ed}}{V_{b,Rd}}$	0,04	<1 Ok
$\frac{F_{Ed}}{R_{w,Rd}}$	0,49	<1 Ok
$\frac{N_{Ed}}{N_{t,Rd}} + \frac{M_{y,Ed}}{M_{cy,Rd}} + \frac{M_{z,Ed}}{M_{cz,Rd}}$	0,31	<1 Ok
$\frac{N_{Ed}}{N_{c,Rd}} + \frac{M_{y,Ed} + \Delta M_{y,Ed}}{M_{cy,Rd}} + \frac{M_{z,Ed} + \Delta M_{z,Ed}}{M_{cz,Rd}}$	0,31	<1 Ok
$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y,Rd}} + (1 - \frac{M_{f,Rd}}{M_{pl,Rd}})(\frac{2\,V_{Ed}}{V_{w,Rd}} - 1)^2$	No interaction between Shear and bending	Ok
$\frac{M_{Ed}}{M_{c,Rd}} + \frac{F_{Ed}}{R_{w,Rd}}$	0,79	<1,25 Ok
$\frac{M_{Ed}}{M_{b,Rd}}$	0,41	<1 Ok

Finally, because the maximum deflection in the worst-case beam was 15,1mm, which is lower than L/250 = 18,8mm, then also the serviceability requirements were fulfilled.

7.2.5. Garage roof

As said before, the garage roof is a specially demanded slab, since it has a very big span (7,55m) and being an accessible rooftop, whose configuration has gravity loads similar to the first floor. Because of these characteristics, the stresses (shown in Table 7-22) on the beams were very high and so it was necessary to use a higher and thicker built-up cross-section.

Table 7-22 - Garage roof beams stresses

Stresses	Value
V_{Ed} (kN)	13,61
$M_{Ed,y}$ (kN.m)	26,0
F_{Ed} (kN)	4,49

To face these stresses, it was used a back-to-back built-up with the highest possible web C-section, the C250, with 2.5mm sheet thickness. With the chosen cross-section it was possible to get the properties exposed in Table 7-23and Table 7-24

Table 7-23 - Garage beams cross-section properties

Properties	Values
Area (mm^2)	1870.000
I_Y (mm^4)	15838908.021
I_z(mm^4)	529746.991
$W_{el,Y}$ (mm^3)	127476.121
$W_{el,z}$ (mm^3)	13754.408

Table 7-24 – Garage beams resistant properties

Resistant properties	Values
$M_{cy,Rd}$ (kN.m)	39,90
$M_{cz,Rd}$ (kN.m)	4,26
$V_{b,Rd}$ (kN)	225,56
$R_{w,Rd}$ (kN)	116,46
$M_{b,Rd}$ (kN)	28,76

This cross-section could only fulfill the lateral-torsional buckling requirements by introducing bridging or blockings each 1,50m. All the ultimate limit states verifications were satisfied as it is demonstrated in Table 7-25, while the maximum deflection admitted was L/250=7550/250=30,2 mm and as it is possible to see in Figure 7-16 the maximum deflection was 28,9mm.

Table 7-25 - Rooftop beam safety checks

Safety Checks	Value	Verification
$\frac{M_{y,Ed}}{M_{cy,Rd}} + \frac{M_{z,Ed}}{M_{cz,Rd}}$	0,65	<1 Ok
$\frac{V_{Ed}}{V_{b,Rd}}$	0,06	<1 Ok
$\frac{F_{Ed}}{R_{w,Rd}}$	0,53	<1 Ok
$\frac{N_{Ed}}{N_{t,Rd}} + \frac{M_{y,Ed}}{M_{cy,Rd}} + \frac{M_{z,Ed}}{M_{cz,Rd}}$	0,65	<1 Ok
$\frac{N_{Ed}}{N_{c,Rd}} + \frac{M_{y,Ed} + \Delta M_{y,Ed}}{M_{cy,Rd}} + \frac{M_{z,Ed} + \Delta M_{z,Ed}}{M_{cz,Rd}}$	0,65	<1 Ok
$\frac{N_{Ed}}{N_{Rd}} + \frac{M_{y,Ed}}{M_{y,Rd}} + (1 - \frac{M_{f,Rd}}{M_{pl,Rd}})(\frac{2\,V_{Ed}}{V_{w,Rd}} - 1)^2$	No interaction between Shear and bending	Ok

$\frac{M_{Ed}}{M_{c,Rd}} + \frac{F_{Ed}}{R_{w,Rd}}$	1,18	<1,25 Ok
$\frac{M_{Ed}}{M_{b,Rd}}$	0,90	<1 Ok

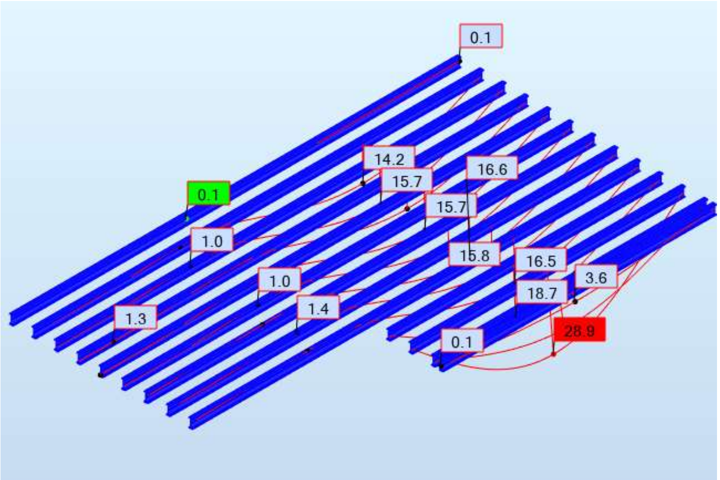


Figure 7-16 - Garage beams deflection [26]

7.3. Bracings

The location of shear walls significantly affects the building behavior when under horizontal forces. It is wanted that the shear walls are located as far as possible from the center of gravity and in a symmetric layout. Even more, this wall should be anchored from the foundations all the way up to the roof.

Attending to this, in the specific case in study is difficult to chose which walls should be shear walls because there are only few walls that fulfill the full height of the building, however these walls were mainly interior walls and as said before, it is wanted that these walls are as far away from the center of rigidity as possible, which means that they should be exterior walls. Since it wasn't possible, it was used all the wall that met the necessary requirements.

When analyzing the layout disposition of the shear walls it was noticeable that most of the walls were to close to the rigidity center and so the first mode of vibration could be torsional, which is not allowed by Eurocode 8.

In order to have a translational first mode of vibration, another wall was added aligned with a second floor exterior wall on the side and direction with less shear walls.

With the finale layout disposition of the shear walls, it was possible to fulfill all the necessary requirements, namely the horizontal displacement of the building that couldn't be higher than 12 mm, and the maximum value obtained was 11mm in X direction and 8,2mm in Y direction.

8
Conclusions

8.1. Developed work

This dissertation main objective was to understand the major characteristics of light steel framing construction and the basis of cold formed steel structures design. It was possible to understand the biggest restrains to the use of LSF as well as the appliances of the legislation.

In short, light steel framing is a relatively recent method of construction that offers various advantages when applied to habitation buildings in matter of time of execution, cost, quality and sustainability. However, this solution is still very little used in Portugal mainly because of the lack of knowledge by the construction agents. On one side, people feel more confident using heavy concrete structures which gives them a feeling of eternity which isn't real at all. On the other side, constructors don't have enough knowledge, qualified information sources nor qualified handwork.

Major focus should be given to the excessive regulation which difficult the use of this method of construction. For example, to licensing any construction is needed a responsibility term given by and certified engineer in both, the design stage and the construction stage. Frequently there are required the same requirements to architects, HSHW technicians, inspectors and others. Due to the quantity of people involved is unlikely that everybody is familiar with this construction method or the applied normative rules. Even more, most European or national norms are very confusing and difficult to implement, as well as they lack some key features of the method such as the diaphragm action provided by the claddings.

Regarding this issue, the American Iron and Steel Institute following the AISI S400, allows the use of a prescriptive method which is much more user friendly while provides the design requirements to most of the common buildings. Of course, that this method has some restrictions regarding the kind of building, the symmetry, height, openings, and others. For example, the case study building couldn't be design using the prescriptive method since it is very unsymmetrical and has a wide cantilever.

On the other hand, to build steel structures is required some knowledge that traditional construction dispenses. The tools used in light steel – screwdrivers, grinders, and drillers- are much more sophisticated than mortar tools – trowel, spoon and plumb line. Also, the hazards inherent to this method of constructions are different, being that cuts due to the sharp edges of the profiles are frequent.

8.2. Future Work

The main subject lacking in this dissertation is the design of the connections. Throughout this work, it was always assumed that all the built-up cross-sections were perfectly connected, but this should be ensured by proper design.

Also, the connection between elements (studs, tracks and beams) should be design according to Eurocode 3-1-3. Relatively to the case study, a special focus should be given to the connections between the extremity beams and the perpendicular beams in cantilever.

One other aspect that could be interesting to analyze is the real behavior of the structure when applied all the claddings on the walls, to understand how rigid the building would be and if the modes of vibrations obtained represent the reality. This, as explained before, would be a very difficult and lengthy process, however very interesting to compare results.

Finally, another very interesting comparison that could be made, would be designing the same building according to the American prescriptive method and identify the major difficulties encountered as well as the main differences in the choices made, understanding how much conservative this method is and by the weight of the framing structure estimated the cost discrepancies.

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Annex

Annex 1

Effective Cross-sections Properties

- C170 Effective cross-section properties

C170	Gross	Compression	Bending Y+	Bending Y-	Bending Z+	Bending Z-
Y_g(mm)	12.70	19.43	12.91	12.91	12.70	19.43
Z_z(mm)	84.25	84.25	83.44	85.06	84.25	84.25
A(mm^2)	441.000	288.310	433.980	433.980	441.000	288.310
I_y(mm^4)	1885818.91	1753980.910	1832138.04	1832138.04	1885818.91	1753980.910
I_z(mm^4)	143542.660	105831.680	142390.340	142390.340	143542.660	105831.680
W_y(mm^3)	22383.610	20818.760	21539.330	21539.330	22383.610	20818.760
W_z(mm^3)	4009.938	3640.489	4000.610	4000.610	4009.938	3640.489

- U170 Effective cross-section properties

U170	Gross	Compression	Bending Y+	Bending Y-	Bending Z+	Bending Z-
Y_g(mm)	8.98	4.36	6.14	6.14	3.91	14.67
Z_z(mm)	85.75	85.75	74.58	74.58	85.75	85.75
A(mm^2)	405.000	166.150	355.070	355.070	347.620	248.000
I_y(mm^4)	1716967.7	971899.600	1073007.0	1073007.00	1294872.00	1572824.00
I_z(mm^4)	86821.880	7468.370	51679.110	51679.110	22046.200	66089.800
W_y(mm^3)	20022.950	11333.770	14387.100	14387.100	15101.000	18345.700
W_z(mm^3)	2155.990	424.040	1198.870	1198.870	841.290	1911.500

- C90 Effective cross-section properties

C90	Gross	Compression	Bending Y+	Bending Y-	Bending Z+	Bending Z-
Y_g(mm)	17.45	20.10	17.45	17.45	17.45	20.10
Z_z(mm)	44.25	44.25	44.25	44.25	44.25	44.25
A(mm^2)	321.000	278.670	321.000	321.000	321.000	278.670
I_y(mm^4)	431213.910	428404.720	431213.910	431213.910	431213.910	428404.720
I_z(mm^4)	116919.950	102062.900	116919.950	116919.950	116919.950	102062.900
W_y(mm^3)	9744.950	9681.460	9744.950	9744.950	9744.950	9681.460
W_z(mm^3)	3765.651	3593.983	3765.651	3765.651	3765.651	3593.983

- U90 Effective cross-section properties

U90	Gross	Compression	Bending Y+	Bending Y-	Bending Z+	Bending Z-
Y_g(mm)	12.77	4.61	8.94	8.94	6.69	15.23
Z_z(mm)	45.75	45.75	53.42	38.08	45.75	45.75
A(mm^2)	285.000	156.990	244.080	244.080	234.220	238.830
I_y(mm^4)	405035.30	230078.560	210603.45	210603.450	298719.130	401390.140
I_z(mm^4)	73037.160	7282.470	45565.670	45565.670	23304.190	64049.390
W_y(mm^3)	8853.230	5029.040	6529.380	3765.651	6529.380	8773.560
W_z(mm^3)	2001.900	419.540	1130.280	1130.280	909.260	1882.920

C250 Effective cross-section properties

C250	Gross	Compression	Bending Y+	Bending Y-	Bending Z+	Bending Z-
Y_g(mm)	9.99	10.83	9.99	9.99	10.83	9.99
Z_z(mm)	124.25	124.25	124.25	124.25	124.25	124.25
A(mm^2)	935.000	861.950	935.000	935.000	861.950	935.000
I_y(mm^4)	7919454.0	7914256.452	7919454.0	7919454.01	7914256.45	7919454.01
I_z(mm^4)	264873.49	256934.641	264873.49	264873.496	256934.641	264873.496
W_y(mm^3)	63738.060	63696.229	63738.060	63738.060	63696.229	63738.060
W_z(mm^3)	6877.204	6820.950	6877.204	6877.204	6820.950	6877.204

Ω140 Effective cross-section properties

C250	Gross	Compression	Bending Y+	Bending Y-	Bending Z+	Bending Z-
Y_g(mm)	72.75	72.75	72.75	72.75	69.35	76.15
Z_z(mm)	57.88	54.17	57.88	57.88	56.28	56.28
A(mm^2)	902.000	680.160	902.000	902.000	791.080	791.080
I_y(mm^4)	2385475.0	2290562.000	2385475.0	2385475.00	2340686.00	2340686.00
I_z(mm^4)	1141045.0	1010515.000	1141045.0	1141045.00	1066634.00	1066634.00
W_y(mm^3)	29588.060	27161.150	29588.060	29588.060	28469.420	28469.420
W_z(mm^3)	15684.470	13890.240	15684.470	15684.470	14006.980	14006.980