UNIVERSITAT POLITÈCNICA DE VALÈNCIA

School of Aerospace Engineering and Industrial Design

Dimensioning of structure of "El Praíco" sports hall, located in Alhama de Murcia, España

End of Degree Project

Bachelor's Degree in Mechanical Engineering

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Overall, thankful for the journey the last four years have been.

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Abstract:

The purpose of this diploma work is the dimensioning of a steel structure, containing a sports hall of public use in Alhama de Murcia, Spain. The design of the structure is based on the architect's report and project documentation of the structure, which is still being built. It's an industrial unit with sports facilities inside.

The diploma work also contains an explanation of the main materials used (steel, concrete, insulations), from a perspective of both its mechanic properties and their environmental impact and sustainability. The loads acting on the structure are calculated following the criteria established in Eurocodes and the National annex of Spain and analysed on the structure.

Key words: dimensioning, steel, concrete, industrial unit, public, analysis, sports hall.

<u>Note:</u>

This diploma is not project documentation.

The construction design must be verified by an authorized designer who has the authority to create the project and build the buildings from the state authorities.

1. Introduction.

The scope of this diploma work involves a study of the fundamental concepts of structural mechanics, material science and their applications to the design of a public use building made for sport's activity, aiming to provide a descriptive and cohesive guide of steps to beware in the dimensioning of structures to ensure both the safety of users and the structural integrity of the building.

Structural mechanics is a branch of engineering that deals with analysis and design of structures to support or resist loads, for which understanding the implications of loads is crucial for ensuring the safety, reliability, and efficiency of all engineering structures. A structure must satisfy equilibrium conditions, meaning the resultant force and resultant moment acting on it equal zero, and all members that are connected remain connected through deformation.

The dimensioning of structural elements involves the determination of the appropriate size, shape, and material properties of structural components to withstand the applied loads without failure, based on an understanding of material science, load analysis, and design codes and standards.

For the development of this diploma work, an important value is placed on sustainability and environmental impact, of the selection of materials and the overall construction process.

Sustainability in construction addresses the need to balance environmental, economic and social factors in the development of infrastructure. The construction industry significantly impacts natural resources, energy consumption, and greenhouse gas emissions. Integrating sustainable practices helps mitigate these effects, promoting the use of ecofriendly materials, energy efficient design and waste reduction techniques.

An approach that encourages sustainability ensures structures are not only durable, helping humans by satisfying our needs for infrastructure, but also adaptable, helping nature by taking less resources from it. The impact of sustainable practices on the well-being of communities is notorious, supporting local economy, polluting less, and creating safer jobs in an industry renowned for work-related accidents and injuries.

All this is essential to give a more responsible and forward-thinking approach to construction, ensuring that current needs are met without sacrificing the resources that future generations have to meet their own.

In the case for this diploma thesis, the structure being dimensioned is a small sports hall, with one basketball court inside that is also adapted for other sports such as volleyball and football. The surface of the building is $1245m^2$, in one story.

The building's main use is reinforcing the use of the buildings that surround it:

- First, it is located next to the tennis courts: the sports hall will provide a secondary bathroom and changing rooms, as well as a place to store equipment if needed.
- It is next to Valle de Leiva high school: the sports hall will provide a court for the school's sports activities, since the one in their lot is too small.
- Additionally, the sports hall serves a purpose on its own, being available for the public to go and practice sports when it is not being used by the school.

The estimated budget for the construction is one and a half million euros, out of which the biggest part will be dedicated to enclosures and partitions and the structure itself.

It is located in an urban area, with access to all necessary amenities, such as water connections, sewage, etc, in the streets that go around the hall. The hall will make use of the parking already constructed for the tennis court, therefore won't have one of its own. Its main entry, to the north side, is accessible for wheelchair users, with a width of 180cm. There are also three more doors, one to access the storage room from the outside, and two to the sides of the court, of 280cm, 175cm and 82cm.

The building will consist of a steel structure with welded nodes, similar to that of a industrial unit, which will then be covered on the outside with steel plaques, and on the inside with phenolic wood. Therefore, the standards to be met are all specified in Eurocode 3- Steel Structures [1].

1.1. Used symbols.

E_d: design value of internal forces.

R_d: design value of resistance.

$E_{d,dst}$: design value of the destabilising actions.

$E_{d,stb}$: design value of the stabilising actions.

G_k: value of the permanent loads.

Q_k: value of the imposed loads.

Ψ: prescribed safety factors.

ξ: reduction factor for unfavourable permanent actions G.

C_d: design value of the serviceability criteria.

q_k: value of the general effect of imposed loads.

H: altitude.

γ: specific weight of snow.

s_k: characteristic value of the snow load on the ground.

s: snow load on the roof.

C_e: exposure coefficient.

C_t: thermal coefficient.

μ: snow loads shape coefficient.

$V_{b,0}$: fundamental value of basic wind velocity.

V_b: basic of wind velocity.

V_m: mean wind velocity.

C_r: roughness factor.

C_o: orography factor.

q_b: velocity pressure.

ρ: air density.

q_p: peak velocity pressure.

I_v: turbulence intensity.

w_e: wind pressure on external surfaces.

w_i: wind pressure on internal surfaces.

c_{pe}: external pressure coefficient.

c_{pi}: internal pressure coefficient.

A_{ew}: area of external walls.

F_w: wind force.

$c_s c_d$: structural factor.

A_{ref}: area of reference.

h_p: height of parapets on the roof.

ΔT_u: uniform temperature component of temperature distribution.

ΔT_{MY}: linearly varying temperature difference about the z-z axis.

ΔT_{MZ}: linearly varying temperature difference about the y-y axis.

ΔT_E: nonlinear temperature difference component.

T_{max}: maximum annual temperature of air.

T_{min}: minimum annual temperature of air.

T_{out}: shade air temperature.

f_{ac}: correction factor for air conditioning.

f_h: correction factor for heat.

f_{ac}: correction factor for union of pillars and foundations.

a_b: basic seismic acceleration.

μ: behaviour coefficient under seismic action.

F_b: seismic base shear force.

S_d: ordinate of the design spectrum.

T_1: fundamental period of vibration.

m: mass of the structure.

2. Design approach.

Since the building we're working on is for public use, the main goal is to ensure it provides a safe and durable solution to the needs it was built to satisfy.

For this, the design approach should meet the requirements of Serviceability Limit States (SLS). To comply with SLS limitations, the structure must be of service and perform its function during its whole working life; excessive deflection should not limit its use or have negative and unacceptable aesthetic defects, and all cracks should not be big enough to cause durability problems. SLS concerns the correct functioning of the structure under normal use, the comfort of the people using it and the structural appearance, considering deformations and vibrations, whereas Ultimate limit states focus on the extreme conditions to withstand without collapse.

It is not uncommon for structures to meet all ULS requirements while not addressing the SLS. If the criteria of SLS is not properly applied, structures can experience deflection or movement, manifesting as sagging beams and cracked walls and ceilings, as well as bouncy flooring. It also involves the study of material durability since degradation of building materials leads to issues like corrosion of metals.

According to Eurocode 1 [2], the values of dead load, live load, wind load, snow load, and thermal effect and seismic load, when necessary, are considered, making equations that give each type of load a constant depending on its importance.

According to Eurocode 0 [3], Ultimate limit states are those that:

1. Concern safety of people and/or the structure.
2. Concern protection of the contents.
3. States prior to structural collapse.

They should be verified in places where loss of equilibrium of the rigid body, failure by deformation, or failure by fatigue may occur. The verification involves two main equations:

A. The design value of the destabilising actions should be minor or equal to the design value of the stabilising actions.

$$E_{d,dst} \leq E_{d,stb} \quad (01)$$

B. The design value of the effect of internal force, moment or a vector that represents internal forces of moments should be minor of equal to the design value of the corresponding resistance.

$$E_d \leq R_d \quad (02)$$

The general format of prescribed load combinations for ULS, as stated in Eurocode 0, is:

$$E_d = \gamma_{sd} E\{\gamma_{g,j} G_{k,j}; \gamma_p P; \gamma_{q,1} Q_{k,1}; \gamma_{q,i} \Psi_{0,i} Q_{k,i}\} \; j \geq 1; i > 1 \quad (03)$$

$$E_d = E\{\gamma_{G,j} G_{k,j}; \gamma_P P; \gamma_{Q,1} Q_{k,1}; \gamma_{Q,i} \Psi_{0,i} Q_{k,i}\} \; j \geq 1; i > 1 \quad (04)$$

The combination to be considered depends on the design value of the leading variable action and the design combination values of all the variable actions.

a) Permanent and temporary load combinations (basic combinations).

$$E_d = \sum_{j\geq 1} \gamma_{Gj} G_{kj} + \gamma_{Q1} Q_{k1} \sum_{i\geq 2} \gamma_{Qi} \Psi_{0i} Q_{ki} \quad (05)$$

b) Combination with accidental action.

$$E_d = \sum_{j\geq 1} \gamma_{GAj} G_{kj} + A_d + \Psi_{11} Q_{k1} + \sum_{i\geq 2} \Psi_{2i} Q_{ki} \quad (06)$$

c) Combination with action of earthquake.

$$E_d = \sum_{j\geq 1} G_{kj} + \gamma_1 A_{Ed} \sum_{i\geq 1} \Psi_{2i} Q_{ki} \quad (07)$$

Serviceability limit states are those that:

1. Concern functioning of the structure under normal use.
2. Concern comfort of people.
3. Concern appearance of the structure (deflection and cracking).

They are verified regarding the vibrations, deformations and damage that affect comfort, appearance and functioning.

It should be verified that the design value of the effects of actions specified in the serviceability criteria are minor or equal to the limiting design value of the relevant serviceability criteria, being:

$$E_d \leq C_d \quad (08)$$

The prescribed combinations as stated in Eurocode 0 are:

a) Permanent and temporary load combinations (basic combinations).

$$E_d = \sum_{j\geq 1} G_{kj} + Q_{k1} + \sum_{i\geq 2} \Psi_{0i} Q_{ki} \quad (09)$$

b) Combination with accidental action.

$$E_d = \sum_{j\geq 1} G_{kj} + \Psi_{11} Q_{k1} + \sum_{i\geq 2} \Psi_{2i} Q_{ki} \quad (10)$$

c) Combination with action of earthquake.

$$E_d = \sum_{j\geq 1} G_{kj} + \sum_{i\geq 2} \Psi_{2i} Q_{ki} \quad (11)$$

Eurocode also provides the safety factors to be used in the equations, depending on the types of loads on the structure.

Table 1: values of the prescribed safety factors (cited from Eurocode 0).

Type of load	Ψ_0	Ψ_1	Ψ_2
1. Live (imposed) load in buildings			
Category A: living areas	0.7	0.5	0.3
Category B: officies	0.7	0.5	0.3
Category C: buildings for gathering of people	0.7	0.7	0.6
Category D: shops	0.7	0.7	0.6
Category E: archives	1.0	0.9	0.8
2. Traffic loads in buildings			
Category F: vehicles weight to 30 kN	0.7	0.7	0.6
Category G: vehicles weight from 30 kN to 160kN	0.7	0.5	0.3
Category H: roofs	0	0	0
3. Climate loads			
Snow load (Finland, Iceland, Norway, Sweden)	0.7	0.5	0.2
Snow load (CEN countries over 1000 m altitude)	0.7	0.5	0.2
Snow load (CEN countries under 1000 m altitude)	0.5	0.2	0
Wind load	0.6	0.2	0
Temperature	0.6	0.5	0

3. Description of the building.

3.1. Location.

The sports hall is in the crossroads of street Cervantes and Joaquín Blume, in Alhama de Murcia, Murcia (location in figure 1), in a lot owned by the town council, next to sports facilities “El Praíco”. The rectangular lot is $3000m^2$, limiting north with high school Valle de Leyva, south with street Miguel de Cervantes, west with the changing rooms of the sports facilities and east with street Joaquín Blume.

The view from the lot is appealing, having the Carrascoy mountain range, the valley, and the town to the south, and the Espuña mountain range to the opposite side.

Figure 1: Location of the town.

3.1.1. Topography of the lot.

Two geotechnics reports were done on the lot and its surroundings by the company CEICO [4].

The first one (2009) involved seventeen standard penetrations testing in three different holes done with a borehole from 8 to 17,4m deep, and two dynamic probe tests. After this, all laboratory testing was conducted. This study also used information from studies done to the surroundings of the lot.

In concluded that the lot has a first layer of filler to an average depth of 2,2m, although in some areas it is as deep as 11m, due to a previous construction that took place in the same space and was later demolished. The second layer is gravel and silt, sometimes compacted, with an average depth of 3,5m. the last layer is a mixture of silt and clay with gravel. This all makes the lot be notably heterogeneous.

The second study, in 2016, was mainly focused on the lot next to the one where the sport hall is located. It involves data obtained from doing fourteen SPT in two holes drilled with boreholes to a depth of 9 and 20m, all previous data from the last study, 2 dynamic probing super heavy and laboratory testing. This study detected a layer of filling, one of silts and gravel to a depth of 6,5m, one of compacted silt up to 8m and mixed clay and silt from there on.

Due to the complexity of the terrain, and the vast differences in different points of the lot, it is recommended to count with geologists involved in both studies to solve any upcoming concern can might arise during the design and construction process, and for the technical directors in the construction to carefully study the terrain as it gets revealed in the excavation process, to verify its similitude to the tests performed.

3.2. Use of the building.

The building is design to satisfy the need for a sports hall, of public use, that also supports and completes the use of the already existing sports hall from the high school next to it and the tennis courts also located right outside. The stands for visitors can sit up to 110 people. The maximum capacity of the building is around 2000 people.

It is a big diaphanous container, big enough to contain a sports court, optimal for basketball, volleyball, and indoor football. It also has locker rooms, storage rooms for the sports equipment, stands for people to watch the games and a WC. The latter has also an entry through the outside of the container, which makes it accessible for people using the courts outside, even when the sports hall’s main entry is closed.

The goal for the design of the structure, was to make a building that looked like it belonged next to the buildings its surrounded by, and, despite it being such a big structure, for it not to draw all the attention, while prioritizing functionality.

The lot is classified for suburban terrain for public sports facilities. The terrain is considerably flat, with two slightly higher points in the south. All needed amenities, such as water, sewage, gas, and electricity networks connections are in the two main streets. It also has some greenery and trees.

3.3. Plans and design.

It's a metallic structure with a rectangular floorplan measuring $44{,}27x27{,}60m$ with a height of $10{,}70m$. The enclosure wall is reinforced concrete and two metres high, from where the metallic pillars start.

The structure has triangular trusses, to allow for a better and more efficient entry of daylight. Daylight is fundamental in a building like this, specially to ensure that there will not be issues with the lights being blinding or defaulting the sports activity complicated. This type of structure also allows for a better circulation of air, making it possible to have natural air on demand, since they allow the access in them for cleaning, opening and closing, maintenance, etc. Throughout the width of the sports hall, one of the trusses is vertical and contains a window, the other is inclined and serves the purpose of a coverage and roofing. Between the trusses, a system of slightly inclined top chords supports the roofing, giving the right slope needed for water evacuation. Through its length, there are trusses of similar characteristics, located above the stands, to allow access for maintenance.

All these elements are supported by double pillars made of laminated UPN steel profiles, to ensure structural efficiency. In the façade, a series of parallel profiles allow for the attachment of the enclosure. Both the enclosure and the roofing are in situ sandwich panels with multiple layers: a pre-lacquered steel sheet, polyurethane foam, air chamber and a pre-lacquered, corrugated steel sheet. On the inside, this is finished with a phenolic wood panel, a second air chamber and 7 centimetres of high density rockwool insulation; the wood panel is grooved in some places for acoustic improvements.

From the inside, the hall looks like a wood box, with transversal grooves allowing for entry of daylight. From the outside, it is steel box, with two different types of varying size, disposition, and corrugation, but same colour.

3.3.1. Foundation and structural system.

The foundation is based on the geotechnic reports previously done, and made of a reinforced concrete wall footing, on a cyclopean concrete base, to a depth of 2,2 metres. The perimeter of the building has a reinforced concrete wall that supports the steel pillars. The concrete also contains water-repellent additive. All inside walls are made of reinforced concrete.

3.3.2. Plans.

The following pages contain the site plans and views of the sports hall from the architect's report [4].

The figures are:

Figure 2: Location of the lot.

Figure 3: Floor plan of the building.

Figure 4: Top view of the building.

Figure 5: Front plane of the building.

Figure 6: Back plane of the building.

Figure 7: Left side plane of the building.

Figure 8: Right side plane of the building.

Figure 2: Location of the lot.

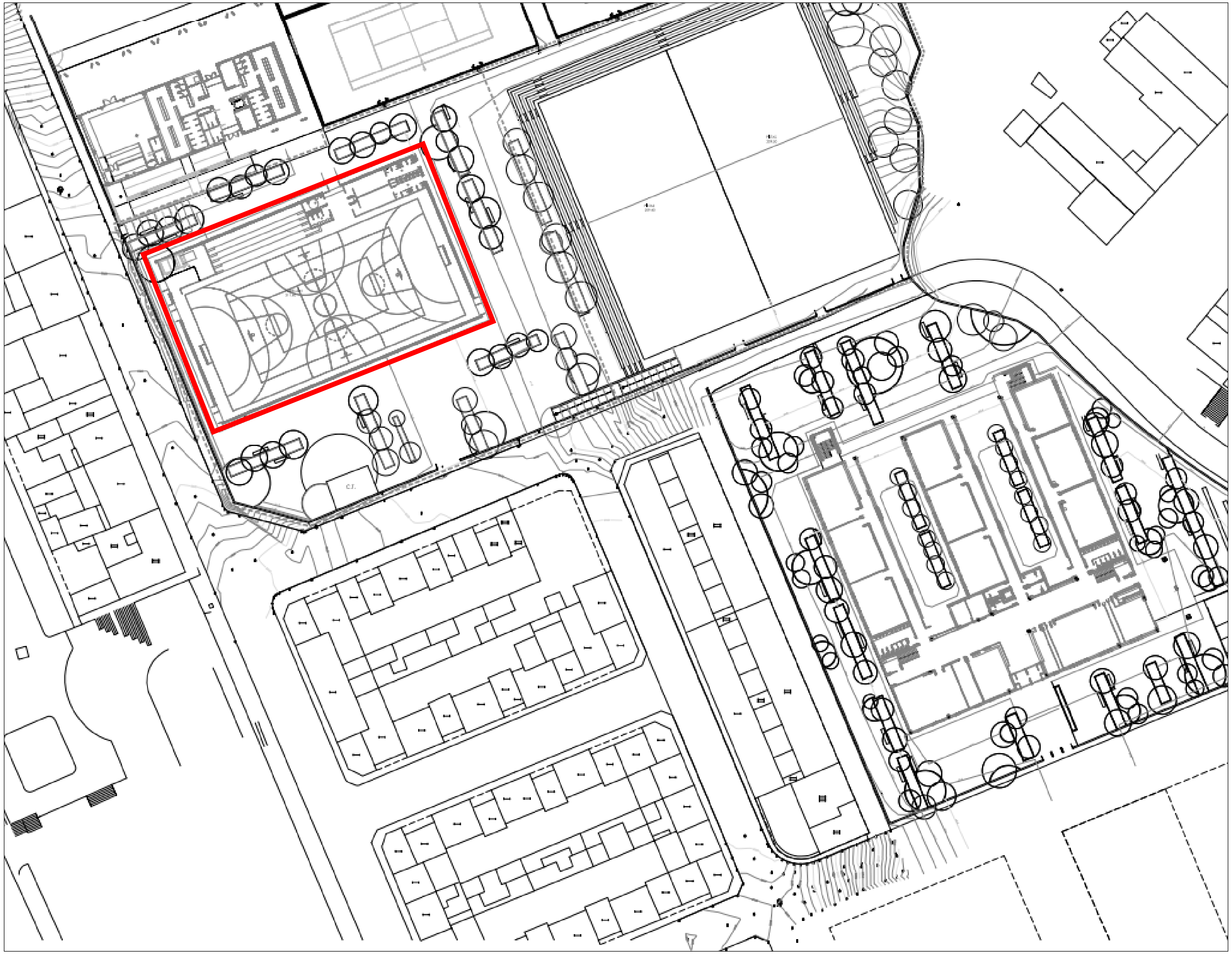


Figure 3: Floor plan.

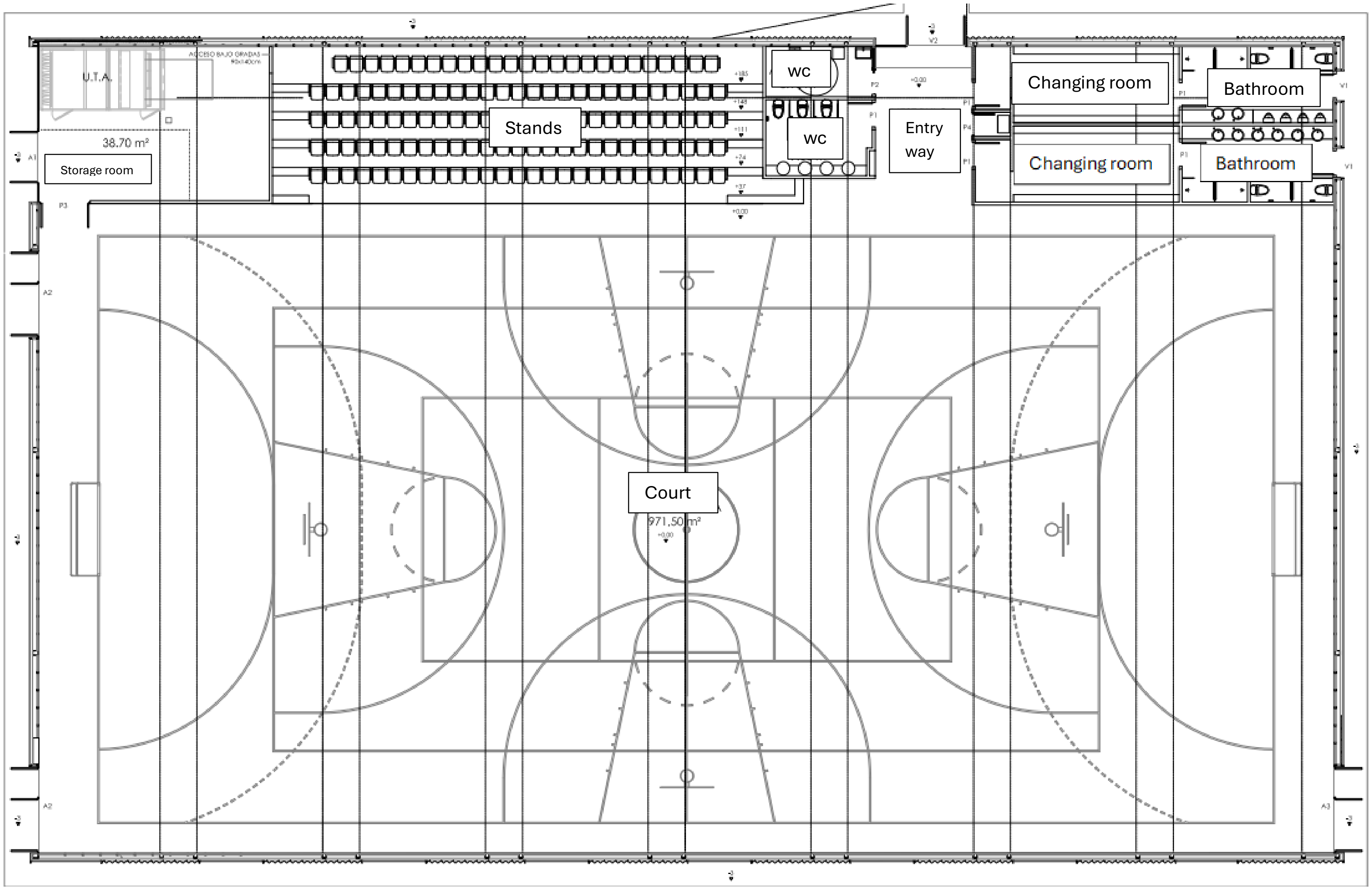


Figure 4: Top view of the building.

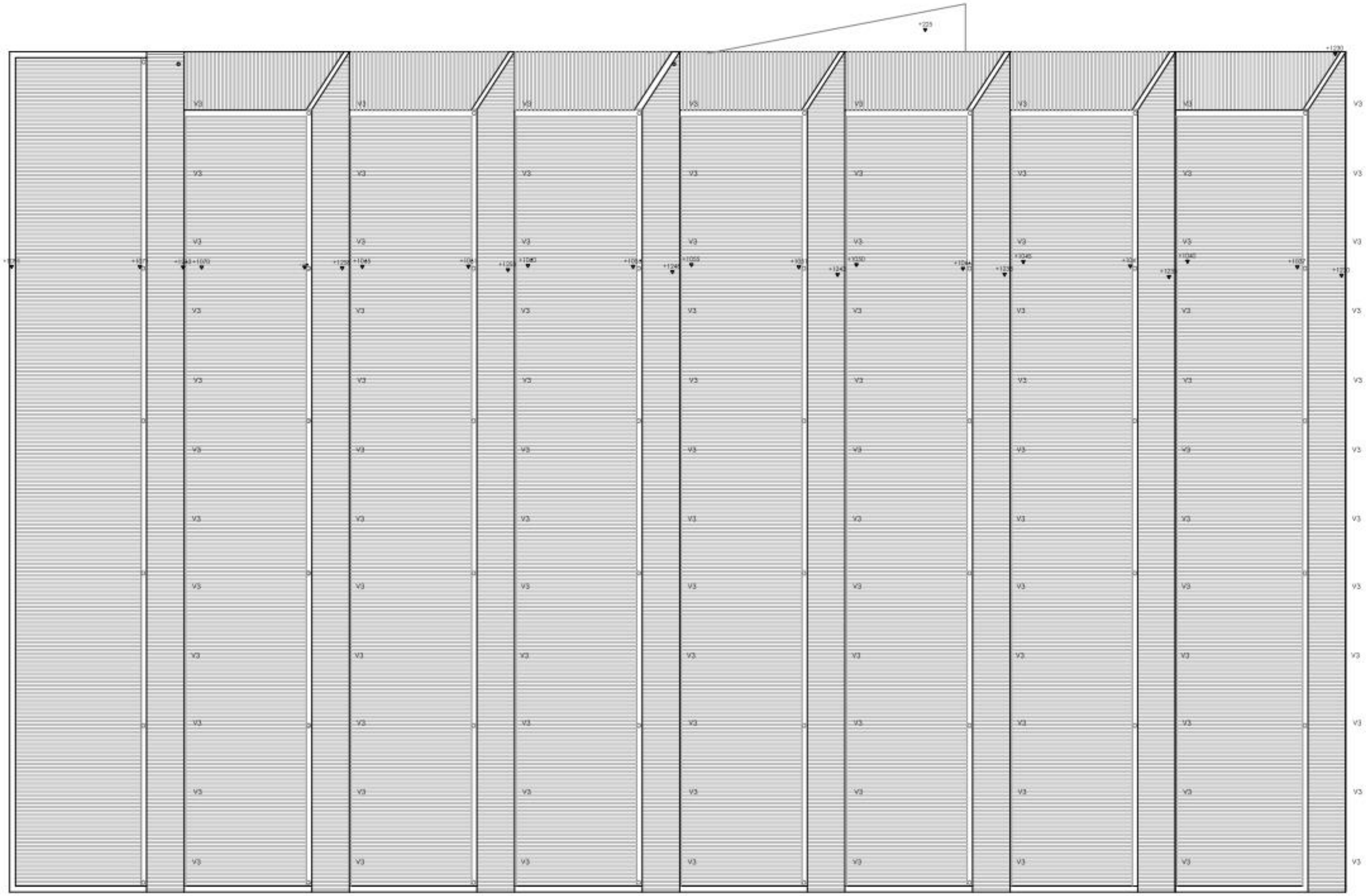


Figure 5: Front plane of the building.

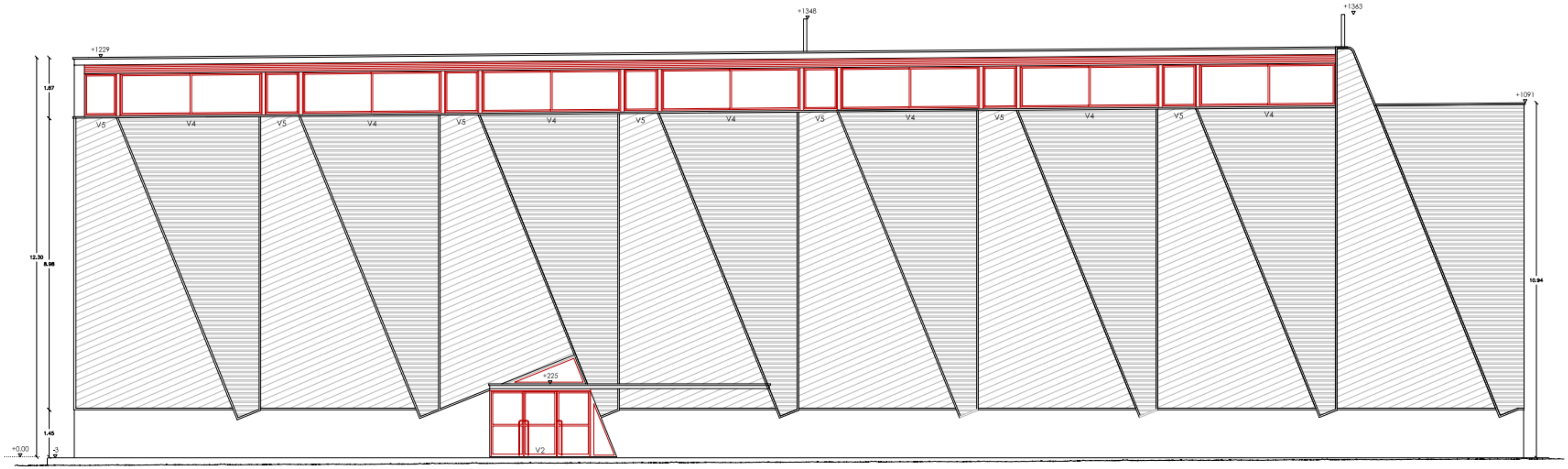


Figure 6: Back plane of the building.

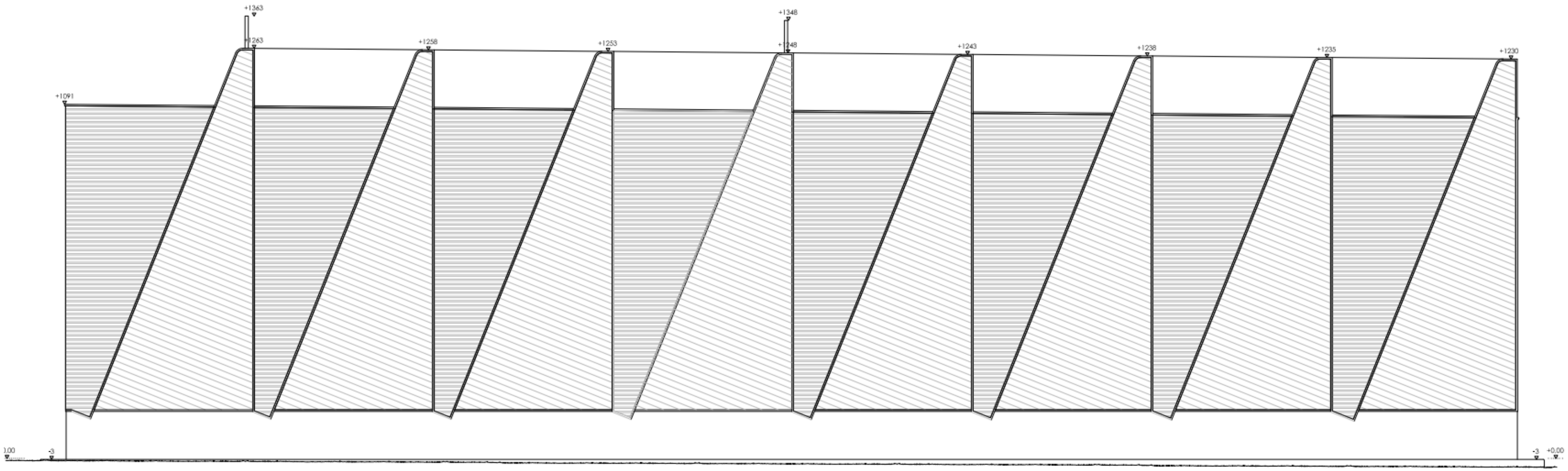


Figure 7: Left side plane of the building.

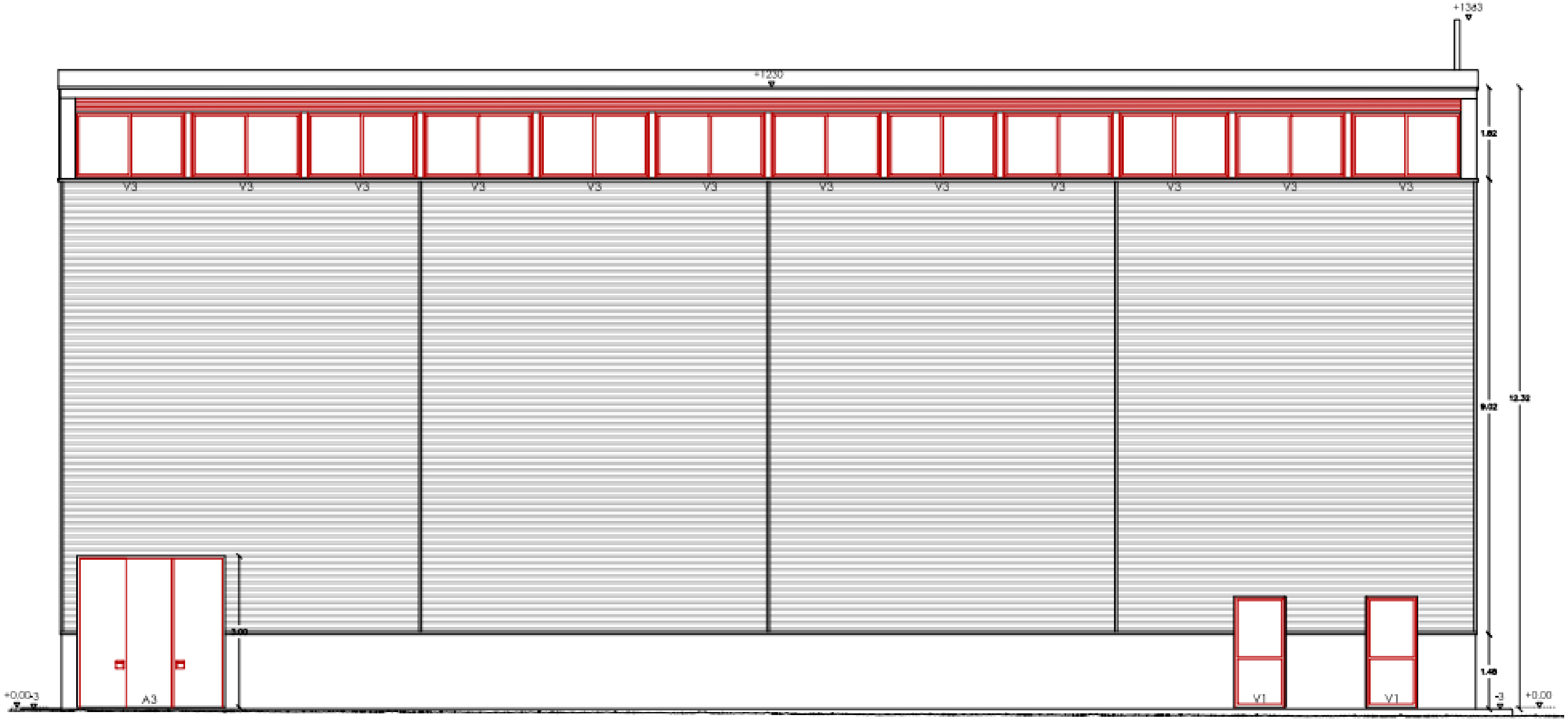
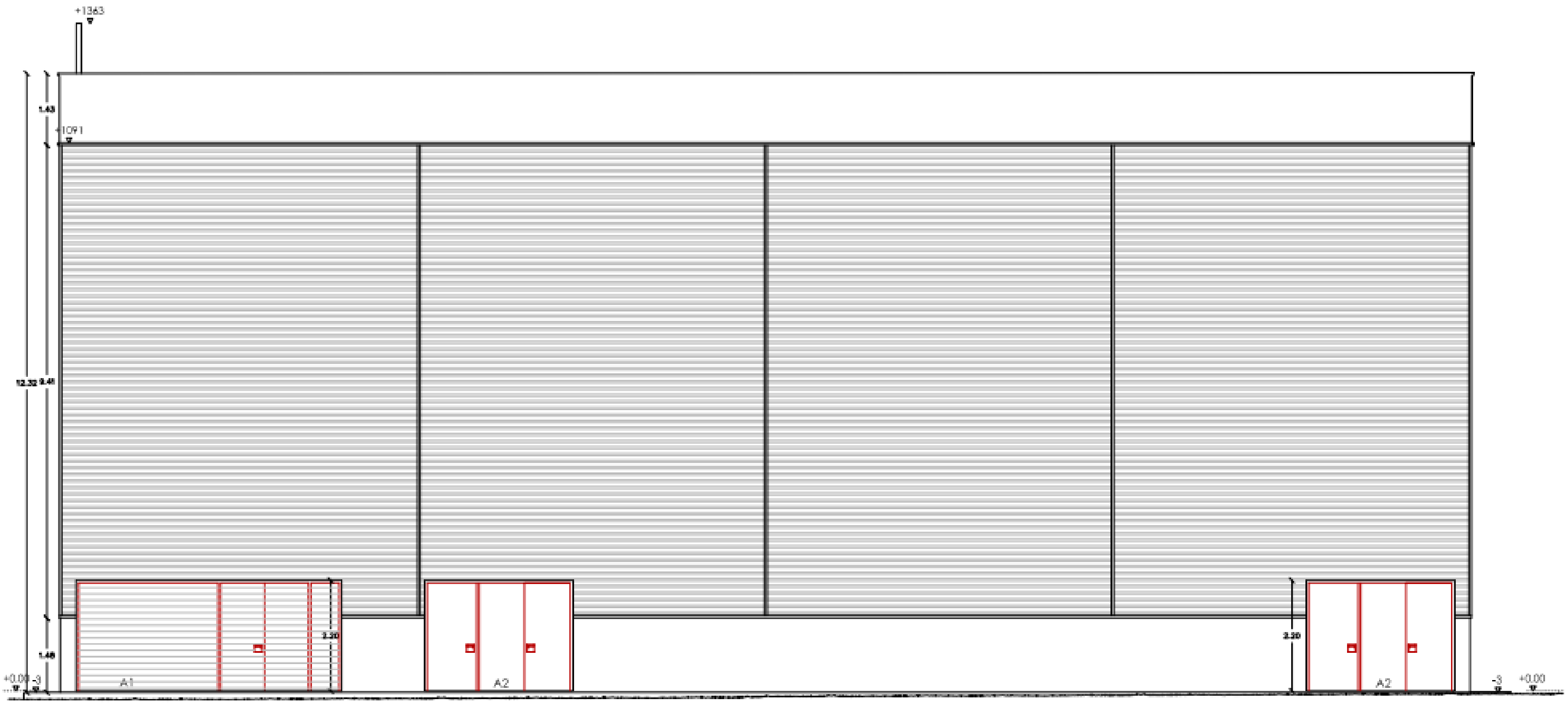


Figure 8: Right side plane of the building.



3.4. Materials: mechanical properties and environmental impact.

With the ever-growing demand of new structures and buildings that meet the changing needs of society, it's important as engineers to make smart decisions when it comes to the materials used to construct.

The construction sector in Europe employs around 20 million people directly and 40 million indirectly, representing more than 10% of Europe's gross domestic product [5]. This industry uses the biggest quantity of raw materials and energy consumption, given that transport and processing of construction materials take up 10% of Europe's energy consumption and the whole industry is responsible for 50% of the global carbon dioxide emissions [6].

The materials chosen for construction can not only reduced this consumption, but also, when a smart decision based on location and climatic conditions is made, it can help reduce the energy consumption of buildings during their use-life, which constitutes 40% of all energy consumption in the continent [7]. The potential to achieve energy savings is the highest in this sector.

Sustainable construction faces many challenges, due to the interrelations existing between environmental impact (potential impacts and ecology), social needs (comfort and aesthetics), and economic aspects (durability and overall costs of construction). Furthermore, being sustainable in this industry entails making decisions keeping the mind the whole life cycle of a building, from its construction to its potential demolishment or adaptation to new needs, and the impact the construction has with its surrounding environment.

Lifetime of a structure is influenced by cultural and market forces. When a structure no longer serves an important function, it is likely to be destructed, and if it is not aesthetically pleasing, it may also be destructed.

Construction materials are manufactured from a combination of raw materials with an expenditure of energy and its associated wastes. Manufacturing is a crucial factor in computing environmental impact, and the most widely citing, involving ecologically sourcing of materials, reduce of waste, etc.

On this structure, the main materials are:

3.4.1. Steel:

Steel is, basically, an iron alloy, made of around 98% iron and 0.05 to 2% carbon, and small amounts of other components, due to the manufacturing process, the inability to remove them, or the decision to add them.

Properties:

Its properties depend on its chemical composition, heat treatment and manufacturing process. For example, the addition of alloys such as manganese increase its strength but also adversely affects its ductility, and steel subjected to wire drawing in its manufacturing process improve its hardness, tensile resistance (from 20 to 40%) and it's easier to work with, due to less inner stresses [8]. This makes it incredibly versatile, and one of the most utilized materials.

Steel still exhibits metal properties of iron, such as its durability, high tensile and yield strength, and heat conductivity. Its main property is ductility, inversely proportional to the percentage of carbon content.

Environmental impact:

Steel construction has proven to offer a great improvement in sustainable development.

It minimises impact on the local community, due to reduced noise, dust, and pollution on site. Also, steel maximises prefabrication, since most elements are carried to the construction site ready to install, while still providing efficient, safe, fast construction. The concept of prefabrication also helps greatly with reducing waste, because elements are mass produced and optimized through computer aided design. Wastage rates in steel construction vary from 1 to 4% [7].

Prefabricating in specialized factories greatly improves the quality of life of the workers, being that conditions in factories are way safer, have more investment in new technologies, and the workers in those positions receive special training.

For sustainable development, it is required that the end-of-life impact of a building is minimized, and steel products have great potential for being re-used thanks to the standardization of components and connections. Even in cases where it cannot be re-used, steel can be recycled, minimizing the use of resources, energy, and waste. All new steel has a percentage of recycled content, varying from 10 to 100% [7].

The solar reflectance index indicates how cool a surface can remain (the higher the SRI the cooler the surface). It is helpful in determining how ecofriendly a structure is and what energy demand it might have and can be controlled if we control the emittance. Emittance, or emissivity of a surface, measures the surface capacity to emit heat, ranging from 0 to 1. Opaque non-metal materials range around 0.85 and 0.95, while steel can go from 0.85 to as low as 0.1 depending on polishing [5], which makes it very adaptable to any application and the SRI it might require.

3.4.1.1. Rebars.

This steel bars are used as a tension device in reinforced concrete and masonry structures. Concrete has low tensile strength, heavily improved when rebar is cast into it, increasing the overall tensile strength of the structure, since the rebar carries tensile loads.

The ribs on the rebar's surface help it better bond with the concrete and reduce slippage. Still, to avoid it getting pulled out by high stresses, rebar is deeply embedded into adjacent structural members (which increases friction, holding the bar in place) or bent and hooked at the ends to lock it in place (taking advantage of the high compressive strength of the concrete).

Because concrete and steel have a similar coefficient of thermal expansion, the reinforcement in steel ensures the concrete experiences minimal differential stress when exposed to temperature changes. If this was not the case, it would be conflictive, causing additional longitudinal and perpendicular stresses.

Common rebar is made of tempered steel, prone to rusting. However, concrete provides a pH value higher than 12, which stops the corrosion reaction. If the layer of concrete is not thick enough, carbonation from the surface can affect the steel, and if the layer is too thick, it can cause cracks, leading to oxide jacking. Oxide jacking is potentially fatal to a structure, causing internal pressure that can end in collapse. Because of this, important emphasis is placed on monitoring this corrosion.

3.4.2. Concrete

Consists of a hard inner substance known as aggregate, made from different types of sand (fine aggregate) and gravel (coarse aggregate), bonded together by cement and water. Cement requires a source of calcium, usually limestone, and one of silicon, usually clay or sand; some small amounts of bauxite and iron ore can then be added for specific properties. On concrete, some chemicals called admixtures can be added, controlling properties like setting time and plasticity.

In this build, concrete is generally used as reinforced concrete (with steel rebars) and cyclopean concrete (with stones).

<u>Properties:</u>

The use of concrete in this structure makes it challenging to adjust to the SLS since the nonlinear behaviour of this material complicates making reliable predictions. With this material, it's crucial to beware the effects of cracking, creep and shrinkage. For this, we rely on experimental studies that analyse the behaviour of concrete under load during time.

Compressive strength is the most important property of concrete. Its tensile strength is around 10% of the compressive strength.

Creep is the gradual increase in strain with time in an element subjected to prolonged stress, being much larger than the elastic strain; when the element is released, there is immediate elastic recovery and slow creep recovery, much less than the original state. The influence and magnitude of creep depends on concrete mix, aggregates, humidity, duration of the loading and age of the concrete.

Shrinkage is an important parameter in the study of concrete, indirectly proportional to the content of aggregates (aggregates that change volume during the cooling of the concrete produce a large shrinkage strain). The first strain to appear is the drying shrinkage strain, caused by the hardening of freshly applied concrete; this one is a slow process and it's irreversible. Secondly, there is autogenous shrinkage strain, caused by the hardening of concrete during the first days of casting; this is a fast process.

Environmental impact:

According to the Environmental Research Group at the University of British Columbia (UBC) [9], it takes around 1550kg of raw materials and 1758kWh to make a ton of finished cement, an ingredient that takes up around 12% of the weight of a typical concrete mix. Cement production is one of the most energy intensive of all industrial manufacturing process. The world's cement consumption is 1.6 billion tons, causing 7% of all loading of carbon dioxide to the atmosphere [10].

Most of this energy is consumed operating the rotary cement kilns. Switching to dry-process kilns can save up to 50% of energy [9], because energy is not needed to drive off moisture; however, the high temperatures of cement kilns allow for a safe disposal of chemicals that otherwise would be hard to process.

The other close to 80% of the weight of concrete, the aggregates, involve the mining, processing, and transport of great quantities of sand, gravel, and crushed rocks, needing considerable amounts of energy and impacting the ecology of forest area and riverbeds.

Some of the environmental impact of concrete could be salvage by replacing part of the conventional Portland cement for fly ash, a fine powder recovered from the gases created by coal-fired electric power generation. Power plants produce millions of tons of this product, that usually gets thrown out at landfills.

It consists mostly of silica, alumina, and iron, that, when mixed with lime and water forms a compound like conventional cement. It has reduced internal friction, increased the mobility of the concrete, and allowed for longer pumping distances. Less water is needed resulting in less segregation of the mixture, giving a smoother surface with sharper details.

Overall, replacing 15 to 35% of the cement for fly ash increases concrete strength, improves sulphate resistance, decreases permeability, reduces the water ratio, and improves the workability of concrete [9]. This benefits in two ways: it makes use of the solid waste of power plants, avoiding the dumping in landfills, and it reduces the energy use.

Another common way to reduce environmental impact is by reusing concrete by crushing it and adding it into aggregate that can then be used in the concrete mix. However, this method gives irregularities in the aggregate, so that more cement is needed to fill all nooks. It can be counterproductive, since it requires more cement, which is the most energy intensive component of concrete.

3.4.3. Rockwool

Rock wool (also known as man-made mineral fibre or mineral cotton) is a type of thermal insulation made from rocks and minerals. The individual fibres that compose it are good conductors of heat, but the sheets and rolls of this material are outstanding at blocking noise and heat. Due to their high melting point of around 1000ºC, it is often used because it can prevent the spread of fire.

<u>Properties:</u>

It is considered 100% fire resistant, non-combustible. When exposed to temperatures of up to 1000ºC, it can maintain stability for up to two hours. It is also resistant to moisture. Its thermal performance is the most attractive quality of this material, it has a high R-value meaning it does not need to be installed in a thick layer to have a strong impact on heat transfer. It also absorbs sound, dampening vibrations and echoes within rooms.

Some downsides are that it is heavier than other types of insulation, and it has low vapour permeability, meaning it can trap moisture. It itself cannot go mouldy, since it is made of minerals, but it can transfer that humidity to the surrounding elements.

<u>Environmental impact:</u>

It is manufactured using furnaces to melt stone particles, so its good environmental performance is unexpected. It belongs to the Man-Made Vitreous Fibres (MMVF) which were once considered carcinogen and associated to toxic emissions. Currently, the more common types of rockwool used as insulation are considered by the International Agency for Research on Cancer to be "not classifiable as carcinogenic in humans."

A study [11] done on the company Flumroc, rock wool manufacturers from Switzerland, showed that roughly 1% of deconstructed rock wool (roughly 300 tonnes a year) is recycled, while the rest is disposed in an inert material landfill. The fabrication of rockwool is tied to 60% of all greenhouse gas emissions related to this material, however, the air pollution caused by rockwool is not only this, but also sulphur dioxide and nitrogen oxide.

However, a study [12] done analysing 20 different types of insulation, not only shows how rockwool comes at the top three for economy performance (economy performance assesses the combined effect of material cost and the annual cost of heating the insulated building), but also ranked rockwool third after cellulose fibre and air on environmental performance alone, and third ranked overall.

3.4.4. Polyurethane foam

PUF is an insulating material that can also support loads, used generally to enhance stability in a confined space with an insulated structure. It's one of the most efficient thermal insulation materials, and it's highly durable, ensuring good performance for the whole life of the building.

Properties:

The structure made by highly polarized urethane bonds determines the mechanical properties of the overall material. Its mechanical properties in tension are identical to those in compression. In compression test, strains of over 90% have been measured without indication of fracture but depends greatly on the density of the foam, since compressive strength is a function of its density; it can be concluded it has excellent mechanical strength.

Its thermal conductivity value is very low, making it insulate a lot with minimum width, and this value does not significantly change in the density range relevant in construction, between 8 to 35 kg/m^3, the most common being 30 [13]. Thanks to its stability and insulation properties, this material performs excellent when exposed to tensile, shear and bending stresses.

Environmental impact:

Polyurethanes are now manufactured with zero ozone depletion and low global warming potential and include recycled content to varying degrees. They are very durable, and in case of demolishing of a building, all undamaged boards can be reused, and clean waste can be crushed and added to new boards, improving their moisture resistance. The waste can also be used as oil binders or as insulating mortar.

In the case that composition is known, and no impurities are found, it can be chemically recovered via glycolysis. If impurities are found, it can be burned in plants with heat recovery systems, transforming it into new primary energy.

3.4.5. Phenolic wood

It is a mixed material consisting of a core made with resin and high-density thermosetting cellulose fibres, over which a finishing layer of wood is added.

Properties:

Its use is widespread in sports facilities and public buildings, since their main characteristics involve being very resistant, reacting good to fire, have a flexural tensile strength, and thermal conductivity. Its resistance to moisture makes this material waterproof, not deforming or swelling when exposed to water.

All these aspects make it ideal for the use in construction with concrete elements.

Environmental impact:

Going along the lines of sustainable construction, phenolic panels are manufactured in factories as part of modular building systems, making them more environmentally responsible. Factories have less of a chance of accidental ground contamination. According to the European phenolic resins' association, its use is associated to energy savings due to thermal insulation, positively contributing to reduce CO_2 emissions. This resins also extend the life of products, leading to less waste.

3.5. Loads

Structures are subjected to actions through their design life, caused by climatic conditions, use and wear and accidental influences. This structures, especially in this case of a public building with a very reiterate use, are crucial to be designed in a way that it can withstand those actions with a degree of reliability and in an economic way for all their design life, of usually 50 years in the case of buildings.

The types, influences, and design recommendations of this loads are specified in EN 1991[2], based on systematic measurements or meteorological observations.

EN 1990 [1] categorises actions in function of their duration, magnitude, and probability of occurrence in permanent, variable, accidental and seismic. They can also be distinguished between direct (applied to the structure) and indirect (caused by deformation or acceleration).

3.5.1. Permanent actions.

The permanent actions involve the weight of the load bearing elements and non-structural elements like roofing, walls, facades, etc.

The dead load is represented by the self-weight, which is a singular characteristic value based on dimensions and densities. The value of the dead load is calculated by the analysis software.

In the case of this structure, the roof has three different types of material composition, causing three different weights on three different areas.

The first area, which takes up around $930m^2$, is composed of the basic structure of steel, polyurethane and rockwool, plus a layer of phenolic birch wood as a finishing, whereas the second area (around $500m^2$) is finished with plasterboard as a fake roof on the inside. For calculating, the mean value of the weight is used.

On the report provided by the architect [4], the following weights of some elements are specified (Table 2), such as:

Table 2: expected weight of elements.

Reinforced concrete	25,0 kN/m^3
Sandwich facade	0,30 kN/m^2
Roof (including insulation)	0,50 kN/m^2
Main truss	1,00 kN/m
Basketball baskets hanging from the truss	0,80 kN/m^2

3.5.2. Imposed loads.

Imposed loads on buildings are those caused by occupancy, such as the circulation of people, furniture and movable objects, vehicles and any predictable event that involves concentration of people or things, even accidental loads from machines if relevant.

They are classified as a variable free action, unless specified otherwise. They are considered as quasi-static actions, which load model might include dynamic effects, especially in cases where the resonance effect from movement of people (such as dancing or jumping) is expected.

Imposed loads are modelled as UDL (q) to account for the general effect, and concentrated load (Q) for local effect, not simultaneously. For this, Eurocodes divides categories of use that are assigned a load value. National index may add on to this, however the one we're looking at (Spanish national index [14]) does not.

Eurocode UNE-EN1991-1-1 [2] contains information regarding the imposed loads. The imposed loads to be considered for serviceability limit state verifications should be specified in accordance with the service conditions and the requirements concerning the performance of the structure. Spanish national index fixes, in chapter AN.2 the values of the nationally determined parameters (NDP) that UNE-EN 1991-1-1 leaves for determination at national level.

The prescribed value for the category of use is stated in table 3.

Table 3: categories of use (cited from EN1991-1-1 Table 6.1. 1).

Category	Specific use	Example
A	Areas for domestic and residential activities.	Rooms in residential buildings and houses; bedrooms and wards in hospitals; bedrooms in hotels and hostels kitchen and toilets.
B	Office areas.	
C	Areas where people may congregate (with the exception of areas defined under category A, B and D).	C1: Areas with tables, etc C2: Areas with fixed seats. C3: Areas without obstacles for moving people. C4: Areas with possible physical activities. C5: Areas susceptible to large crowds.
D	Shopping areas.	D1: Areas in general retail shops. D2: Areas in department stores.
NOTE 1: Depending on their anticipated uses, areas likely to be categorised as C2, C3, C4 may be categorised as C5 by decision of the client and/or National annex. NOTE 2: The National annex may provide subcategories to A, B, C1 to C5, D1 and D2.		

In this case, this floor is subjected to multiple use, for calculations the most unfavourable category of loading will be considered. For all calculations regarding horizontal loads on partition walls, the value of q_k will be applied.

Category of use C5:

$$q_k = \frac{5kN}{m^2} \quad Q_k = 4{,}5kN$$

According to Eurocode EN1991-1-1 [2], when designing a roof, the imposed load is considered as a free action applied to the most unfavourable part of the influence area of the action effects being considered. All roofs should be designed to resist 1,5kN on an area based on a square with a side of $50mm$. The values given by the Eurocode are described in Table 4.

Table 4: categorization of roofs (cited from EN1991-1-1 Table 6.9).

Categories of loaded area	Specific use
H	Roofs not accessible expect for normal maintenance and repair.
I	Roofs accessible with occupancy according to categories A to G.
K	Roofs accessible for special services, such as helicopter landing areas.

Roof only accessible for maintenance: H

$$q_k = \frac{0{,}4kN}{m^2} \quad Q_k = 1kN$$

3.5.3. Snow loads.

Snow loads classify as static and variable, fixed actions.

Table 5: Specific weight of snow as function of the altitude (cited from National annex, Table AN.1).

Altitude, H $[m]$	$\gamma\ [kN/m^3]$
$1800 \geq H \geq 1500$	3,3
$1500 \geq H \geq 1000$	2,7
$1000 \geq H \geq 800$	2,0
$H < 800$	1,5

For $H < 800,\ \gamma = 1{,}5kN/m^3, \psi_0 = 0{,}50\ , \psi_1 = 0{,}20\ , \psi_2 = 0{,}00$

The characteristic value of the snow loads on the ground is obtained through the tables and graphs in the national annex, being:

Figure 9: winter climatic zones (from National annex).

Table 6: Snow loads on horizontal surfaces (cited from National annex, Table AN.2).

Altitude [m]	Winter climatic zone according to figure 1						
	1	2	3	4	5	6	7
	$s_k, [kN/m^2]$						
0	0,3	0,4	0,2	0,2	0,2	0,2	0
200	0,5	0,5	0,2	0,2	0,3	0,2	0

According to the national annex [15], it is climate zone 6, and the characteristic value of snow load on the ground equals: $S_k = 0{,}2kN/m^2$

In cases where a more refined value of the load on the ground is needed, it can be refined by using a statistical analysis of records taken in s sheltered area near the site. However, since snow on this location is extremely unlikely, the value given by the national annex is enough to cover all possible accidental situations.

Snow loads on the roof depend on:

- Properties of the roof: shape, thermal properties, roughness of the surface.
- Other factors: heat generated under the roof, nearby buildings, surrounding terrain and the local meteorological climate.

The load is assumed to act vertically to a horizontal projection of the roof area.

$$s = \mu_i \cdot C_e \cdot C_t \cdot s_k \quad (12)$$

The exposure coefficient (C_e) is used to determine the snow load on the roof, considering the future development around the site, always equals 1 unless specified for a specific topography. In this case, the topography is normal, since there is no significant removal of snow by wind, due to the terrain and surrounding area.

The thermal coefficient (C_t) is used to consider the reduction of snow loads on roofs with a high value of thermal transmittance, such as glass roofs. Since the material of this roof is not in this category, the recommended value from Eurocode is taken, equalling 1.

Regarding local effects, the national annex only requests them in altitudes higher than $1000m$.

The roof is flat, meaning the snow loads shape coefficients equal $\mu = \mu_2 = 0{,}8$.

$$s = 0{,}8 \cdot 1 \cdot 1 \cdot 0{,}2 = \frac{0{,}16kN}{m^2} \quad (12)$$

Since snowing is so rare, accounting for the values of snow drifting and overhanging is not necessary. Overall, it can be concluded that there is no significant or relevant snow load.

3.5.4. Wind loads.

They are classified as variable fixed actions and represented by a simplified set of forces, equivalent to those produced by the extreme effects of turbulent wind.

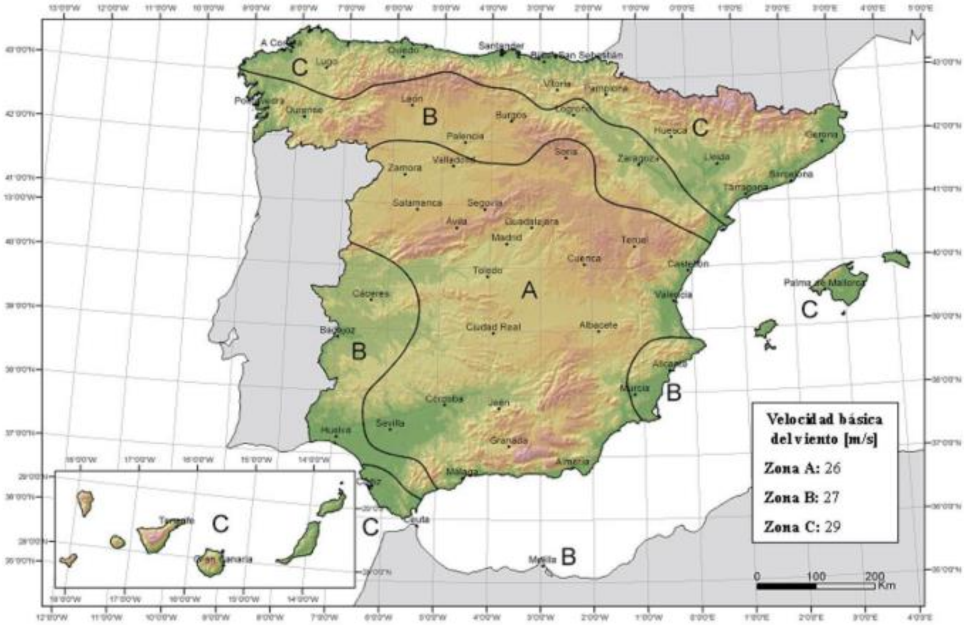
Wind actions change with time and act as pressures on the external surfaces of the structures, and, due to the porosity of the external surface, can also act indirectly on the internal. The resulting pressure forces act on areas of the surface that are normal to the surface of the structure. When large areas are severely affected, friction forces tangent to the surface might be significant.

Effect on the structure depends on its size, shape, and dynamic properties. For many steel structures, wind constitutes the main horizontal loads, however for not flexible structures wind can be considered of static nature.

In accordance with EN 1990 [1], wind actions are determined for each design situation identified. Other actions such as snow, ice or traffic that will modify its effects due to the wind should be considered, as well as the changes of the structure during stages of execution.

From the national annex it is known that the terrain type is II – suburban terrain, located in zone A for wind speed, being $26m/s$ according to figure 2.

Figure 10: Map of fundamental value of basic wind velocity (from National annex).



Directional and season factor both equal one, which implies that the fundamental value of basic wind velocity equals:

$$V_{b,0} = V_b = 26\, m/s$$

The basic value is a characteristic value that has an annual probably of exceedance of 0,02, equivalent to a mean return period of 50 years. The fundamental value on the other hand, is the characteristic 10 minutes mean wind velocity at 10 meters above ground level in an open country field without any obstacles.

The mean wind velocity at a height z above the terrain depends on the roughness and orography factors, equalling:

$$V_m(z) = C_r(z) \cdot C_o(Z) \cdot V_b \qquad (13)$$

The value for the velocity pressure can be calculated. This value is used to calculate the response of the structure at the reference height in the undisturbed wind field. It depends on wind climate, terrain roughness and orography, and the reference height.

$$q_b = \frac{1}{2} \cdot \rho \cdot {V_b}^2 = \frac{1}{2} \cdot 1{,}25 \frac{kg}{m^3} \cdot \left(26 \frac{m}{s}\right)^2 = \frac{0{,}42kN}{m^2} \qquad (14)$$

The peak velocity pressure at a height z above the terrain (including short- and long-term velocity fluctuation) involves:

$$q_p(z) = [1 + 7 \cdot I_v(z)] \cdot \frac{1}{2} \cdot \rho \cdot {V_m}^2(z) = C_e(z) \cdot q_b \qquad (15)$$

However, the National annex gives C_e a general value of 2,4.

$$q_p(z) = 2{,}4 \cdot 0{,}42 = \frac{1{,}014kN}{m^2} \qquad (15)$$

Both the wind velocity and the velocity pressure are composed of a mean and a fluctuating component. The peak velocity pressure equals the mean velocity pressure plus the short-term pressure fluctuations.

The value ρ describes the air density, dependant on the altitude, temperature, and barometric pressure expected during windstorms. In the national annex is recommended to use the value $1{,}25kg/m^3$.

Roughness factor:

It accounts for the variation of the mean wind velocity, caused by the height above ground level and the ground's roughness. For the calculations, since it is terrain category II, the constant values are: $k = 0{,}19 \quad L = 0{,}05m \quad z = 2m$. For flat terrain where the orography factor equals one, the exposure factor is illustrated as a function of height above terrain and of terrain category.

$$C_r = 0{,}19 \cdot \ln\left(\frac{10{,}70}{0{,}01}\right) = 1{,}325 \qquad (16)$$

Orography factor:

In terrains where orography such as hills or cliffs increases wind velocity by more than a 5 percent, the effects must be considered, but the effects can be neglected when the average slope is less than 3 degrees. In this case, the value can be assumed as 1.

Turbulence intensity:

Defined as the standard deviation of the turbulence divided by the mean wind velocity.

$$I_v(z) = \frac{1}{1\left(\ln\left(\frac{10{,}70}{0{,}01}\right)\right)} = 0{,}143 \qquad (17)$$

A. Wind pressure on surfaces.

The net pressure on any surface is the difference between the pressures on opposite surfaces, considering their sign. Forces towards the surface (pressure) are positive and forces away from the surface (suction) are negative.

1. Wind pressure on the walls.

EN 1991-1-4 [16] states regarding vertical walls in rectangular plan buildings, that any building whose height is less than b is one part, giving a uniform shape of profile of velocity pressure.

On the external surfaces, the wind pressure is defined as:

$$w_e = q_p(z_e) \cdot c_{pe} \qquad (18)$$

The external pressure coefficient (c_{pe}) depends on the size of the loaded area, which is the area of the structure that produces the wind action in the section to be calculated. For $1m^2 < A < 10m^2$:

$$c_{pe} = c_{pe,1} - (c_{pe,1} - c_{pe,10}) \log_{10} A \qquad (19)$$

Two of the external walls have an area of $A_{ew1} = 46{,}33 \cdot 10{,}70 = 495{,}731 m^2$

The other two have an area of $A_{ew2} = 29{,}70 \cdot 10{,}70 = 317{,}79 m^2$

On internal surfaces, the wind pressure is defined as:

$$w_i = q_p(z) \cdot c_{pi} \qquad (20)$$

Being c_{pi} a function of the ratio of the height and the depth of the building and the opening ratio for each wind direction.

The wind forces for the structure are determined by calculating forces by using force coefficient or surface pressures. It can be calculated directly of by vectorial summation. For this calculation, the expression used is:

$$F_w = c_s c_d \cdot c_f \cdot q_p(z_e) \cdot A_{ref} \qquad (21)$$

The structural factor $c_s c_d$ in this case is taken as 1 since the height of the building is less than 15 meters.

In this case, the building is longer than it is taller, for it we apply $e = 2 \cdot h = 21{,}4\ m.$ We also know that the height of the building and its width have a ratio of $\frac{h}{d} = \frac{10{,}70}{27{,}60} = 0{,}775.$ Since all the areas are bigger than 10, the value of the external pressure coefficient is calculated with linear interpolation.

Table 7: Areas and values of c_{pe} on the walls, according to Eurocode.

	Zone				
	A	B	C	D	E
Area $[m^2]$	45,8	183,18	66,34	473,69	473,69
c_{pe}	-1,2	-0,8	-0,5	0,8	-0,3
$w_e [kN/m^2]$	-1,217	-0,081	-0,507	0,081	-0,304

2. Wind pressure on the roof.

Flat roofs are defined as those with a slope α such as $-5° < \alpha < 5°$.

When the roof has parapets, the height is: $z_e = h + h_p$.

Being that $\frac{h_p}{h} = \frac{2,05}{8,66} = 0,237$, the estimated values are:

Table 8: Areas and values of c_{pe} on the roof, according to Eurocode.

	Zone			
	F	G	H	I
Area $[m^2]$	7,49	22,9	378,95	473,69
c_{pe}	-1,8	-0,9	-0,7	-0,2
$w_e\ [kN/m^2]$	-1,825	0,917	0,710	0,203

3.5.5. Termic loads.

Thermal actions are classified as variable and indirect actions, since they do not act on the surface, but induce deformations, forces and moments that depend on the magnitude of the variation and the constraint conditions of the structures.

They are characteristic values, with an annual probability of being exceeded of 0,02. Loadbearing elements need to be checked to ensure that the movement caused by temperature changes will not cause overstressing. This structure is exposed to daily and seasonal temperature changes; therefore, the thermic loads need to be considered.

These thermal actions are caused by the changes in shade air temperature, radiation, and other atmospheric characteristics. How intense these actions are depending on local climatic conditions, orientation, mass, finishings and ventilation regimes and insulation. The stress they cause depend on the geometry and boundary conditions of the element, and on the properties of the materials. All of this is prescribed in EN 1991-1-5 [17].

The temperature distribution is composed of four components: uniform temperature (ΔT_u), linearly varying temperature difference about the z-z axis (ΔT_{MY}), linearly varying temperature difference about the y-y axis (ΔT_{Mz}) and nonlinear temperature difference component (ΔT_E). For calculations, the maximum and minimum annual temperature is needed; said value can be taken from the tables and graphs in the National annex.

Figure 11: Map of maximum annual temperature of air (from National annex).

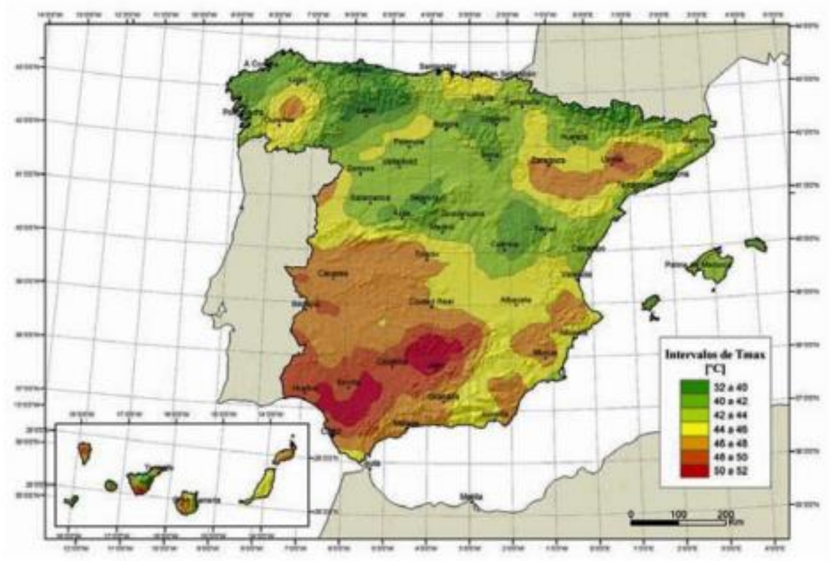


Table 9: Minimum annual temperature of air (cited from National annex).

Altitude [m]	Winter climatic zone according to figure 1						
	1	2	3	4	5	6	7
	Temperature [Cº]						
0	-7	-11	-11	-6	-5	-6	6
200	-10	-13	-12	-8	-8	-8	5

Based on the guidelines provided on the national annex [17], it is known:

$$T_{max} = yellow\ zone = 44\ to\ 46℃$$

$$T_{min} = zone\ 6 = -6℃$$

Being the value of maximum and minimum shade air temperature.

$$T_{out}(2m) = 10℃\ in\ summer\ and\ 5℃\ in\ winter$$

Being the temperature of the outer environment.

The values for calculations are $\Delta T_{con} + 15℃\ and\ \Delta T_{exp} + 15℃$.

According to the geotechnical and topography report, values are:

$$T_{max} = 40℃, T_{min} = -2℃, T_{av} = 13'3℃, \Delta T = 26'7℃, L(\Delta T) = 105m$$

The correction factors for air conditioning, heat and the union of pillars and foundations are:

$$f_{ac} = 1'15, f_h = 1'00, f_{up} = 0'85$$

$$L = L(\Delta T) \cdot f_{ac} \cdot f_h \cdot f_{up} = 105 \cdot 1'15 \cdot 1'00 \cdot 0'85 = 102'64m \qquad (22)$$

From this we know that only one expansion joint is needed to account for the thermic deformation of concrete.

3.5.6. Action of earthquakes.

Involves loads that occur during earthquakes, due to the important ground acceleration. According to EN 1998-1 [18], they must be defined in structures located in seismic regions, which is not the case here. They are inertial forces which magnitude depends on parameters like ductility, soil type and ground acceleration.

The basic seismic acceleration in this location is $a_b = 0'11g$, it is granular soil with medium compact ability, and has a behaviour coefficient of $\mu = 2$ since it has low ductility.

Seismic forces are determined from a response spectrum that gives the acceleration as a function of the vibration period; this spectral acceleration can be multiplied by the mass to know the value of the seismic forces or divided by the peak ground acceleration to obtain the elastic response spectrum. Depending on the period and the stiffness, the spectral acceleration can be larger than the peak ground acceleration, only being smaller for structures that are very flexible; also, spectral acceleration becomes smaller when the structure has an inelastic behaviour.

In high seismicity regions where the peak acceleration is closed to the gravity acceleration, ductile design is anticipated, making the structure have a nonlinear behaviour. This, however, cannot be done for all kinds of structures since it might lead to structural damage.

The seismic base shear force, is calculated according to Eurocode 8 with the following equation:

$$F_b = S_d(T_1) \cdot m \cdot \lambda \qquad (23)$$

Being T_1 the fundamental period of vibration based on the lateral elastic displacement of the top of the building.

However, according to National annex and in this case, with such a low value for basic acceleration and located in a low seismic activity area, the impact of this forces is not considered in the calculations.

3.5.7. Accidental actions.

They are constituted by fire and other unexpected situations that can arise. A bigger emphasis has been placed on the proper study of this actions, since in the most recent history there have been plenty of huge accidents whose impact could have been reduced by a better application of existing knowledge on this field. Values of accidental actions like internal explosions or collisions are covered in EN 1991-1-2 [19], and EN 1991-1-7 [20] exposes the proper design normative for structures exposed to fire.

Applying a structural fire design procedure is fundamental, especially in public use structures, where an accident could be terribly fatal. A structural fire design analysis is composed by a selection of design fire scenarios, determination of design fires, calculation of temperature evolution and of the mechanical behaviour of the exposed structure.

3.5. Combination of actions.

In conclusion, the values we consider for out calculations are:

Value of the imposed loads on the roof: $q_k = 0{,}4 kN/m^2$

Value of the imposed loads on the floor: $q_k = 5\ kN/m^2$

Value of the snow load on the roof: $s = 0{,}16 kN/m^2$

Value of the wind load: for this, we use the highest absolute value out of all the areas analysed: $w_e = -1{,}825 kN/m^2$

All the equations are in function of the value of the permanent, out of which, the dead load will be provided by the software for every case studied in the dimensioning.

Ultimate limit state $[\boldsymbol{kN/m^2}]$:

Load combinations according to Eurocode 0 are expressed in equations 5, 6, and 7.

Actions on the roof:

1. $E_d = G_k \cdot \gamma_G + q_k \cdot \gamma_q + s \cdot \gamma_q \cdot \psi_{01}$
$$E_d = G_k \cdot 1{,}35 + 0{,}4 \cdot 1{,}5 + 0{,}16 \cdot 1{,}5 \cdot 0{,}6 = G_k \cdot 1{,}35 + 0{,}744$$

2. $E_d = G_k \cdot \gamma_G + s \cdot \gamma_q + q_k \cdot \gamma_q \cdot \psi_{01}$
$$E_d = G_k \cdot 1{,}35 + 0{,}16 \cdot 1{,}5 + 0{,}4 \cdot 1{,}5 \cdot 0{,}6 = G_k \cdot 1{,}35 + 0{,}600$$

3. $E_d = G_k \cdot 1{,}0 + w_e \cdot \gamma_q$
$$E_d = G_k \cdot 1{,}0 - 1{,}217 \cdot 1{,}5 = G_k - 1{,}826$$

4. $E_d = G_k \cdot \gamma_G + q_k \cdot \gamma_q + s \cdot \gamma_q \cdot \psi_{01} + w_e \cdot \gamma_q \cdot \psi_{02}$
$$E_d = G_k \cdot 1{,}35 + 0{,}4 \cdot 1{,}5 + 0{,}16 \cdot 1{,}5 \cdot 0{,}6 - 1{,}217 \cdot 1{,}5 \cdot 0{,}5 = G_k \cdot 1{,}35 + 0{,}169$$

Actions on the floor:

$$E_d = G_k \cdot \gamma_G + q_k \cdot \gamma_q$$
$$E_d = G_k \cdot 1{,}35 + 5 \cdot 1{,}5 = G_k \cdot 1{,}35 + 7{,}500$$

Serviceability limit state $[kN/m^2]$

Load combinations according to Eurocode 0 are expressed in equations 9, 10 and 11.

Actions on the roof:

Combination causing vertical deflection:

1. $E_d = G_k + q_k + s \cdot \psi_{01}$

$$E_d = G_k + 0{,}4 + 0{,}16 \cdot 0{,}6 = G_k + 0{,}496$$

2. $E_d = G_k + s + q_k \cdot \psi_{01}$

$$E_d = G_k + 0{,}16 + 0{,}4 \cdot 0{,}6 = G_k + 0{,}400$$

Combination causing horizontal deflection:

3. $E_d = G_k + w_e$

$$E_d = G_k - 1{,}217 = G_k - 1{,}217$$

Actions on the floor:

$$E_d = G_k + q_k = G_k + 5$$

4. Statical analysis

4.5. About SAP2000.

The analysis is conducted using SAP2000. SAP2000 is a structural analysis and design software developed by Computers and Structures, Inc. (CSI), widely used by civil and structural engineers for analysing and designing various types of structures, including buildings, bridges, dams, towers, and other infrastructure projects.

It performs linear and nonlinear static and dynamic analysis of structures, handling a wide range of loading conditions. It includes integrated design modules compliant with international building codes and design standards, such as AISC, ACI, Eurocode, and others, which automate the process of simulating.

SAP2000 provides a set of modelling tools for creating finite element models of structures. It offers advanced visualization capabilities for displaying analysis results, including deformed shapes, stress contours, displacement plots, and mode shapes. SAP2000 integrates seamlessly with other engineering software packages, such as ETABS (also developed by CSI) for advanced building analysis and design, and SAFE for foundation design and detailing.

4.6. Main benefits of SAP2000.

SAP2000 stands out among similar software in the structural engineering field due to several key benefits. Mainly, it is highly versatile and can handle a wide range of structural analysis and design tasks, including linear and nonlinear analysis, static and dynamic analysis, and various loading conditions.

Also, SAP2000 offers an intuitive and user-friendly interface, which makes it accessible to people with little experience, minimizing the learning curve. It includes advanced analysis capabilities, such as finite element analysis (FEA), time-history analysis, pushover analysis, and performance-based design.

4.7. Analysis.

The analysis of this structure is conducted on the different frames that comfort it, taking advantage of the symmetry of it. All the joints in the structure are welded, therefore all six degrees of freedom are constrained. All pillars are made of a double U profile, whereas all other elements are an I profile.

The next pages contain the relevant plans from the architect’s report [4], being:

Figure 12: General 3D view of the structure.

Figure 13: Extract of the connection of nodes. Further specified in annex 1.

Figure 14: Top plane of the metallic structure. In it, all studied frames are marked, as follows in table 10.

Table 10: colours assigned to frames.

Frames	Colour
XZ01	Yellow
XZ02	Orange
XZ05	Red
YZ01	Blue
YZ02	Green
YZ09	Purple

Figure 15: Front plane of frame XZ01.

Figure 16: Front plane of frame XZ02. This frame is identical to frames XZ03 and XZ04.

Figure 17: Front plane of frame XZ05.

Figure 18: Front plane of frame YZ01.

Figure 19: Front plane of frame YZ02. This frame is identical to frames YZ03, YZ04, YZ05, YZ06, YZ07 and YZ08.

Figure 20: Front plane of frame YZ09.

Figure 12: General 3D view of the structure.

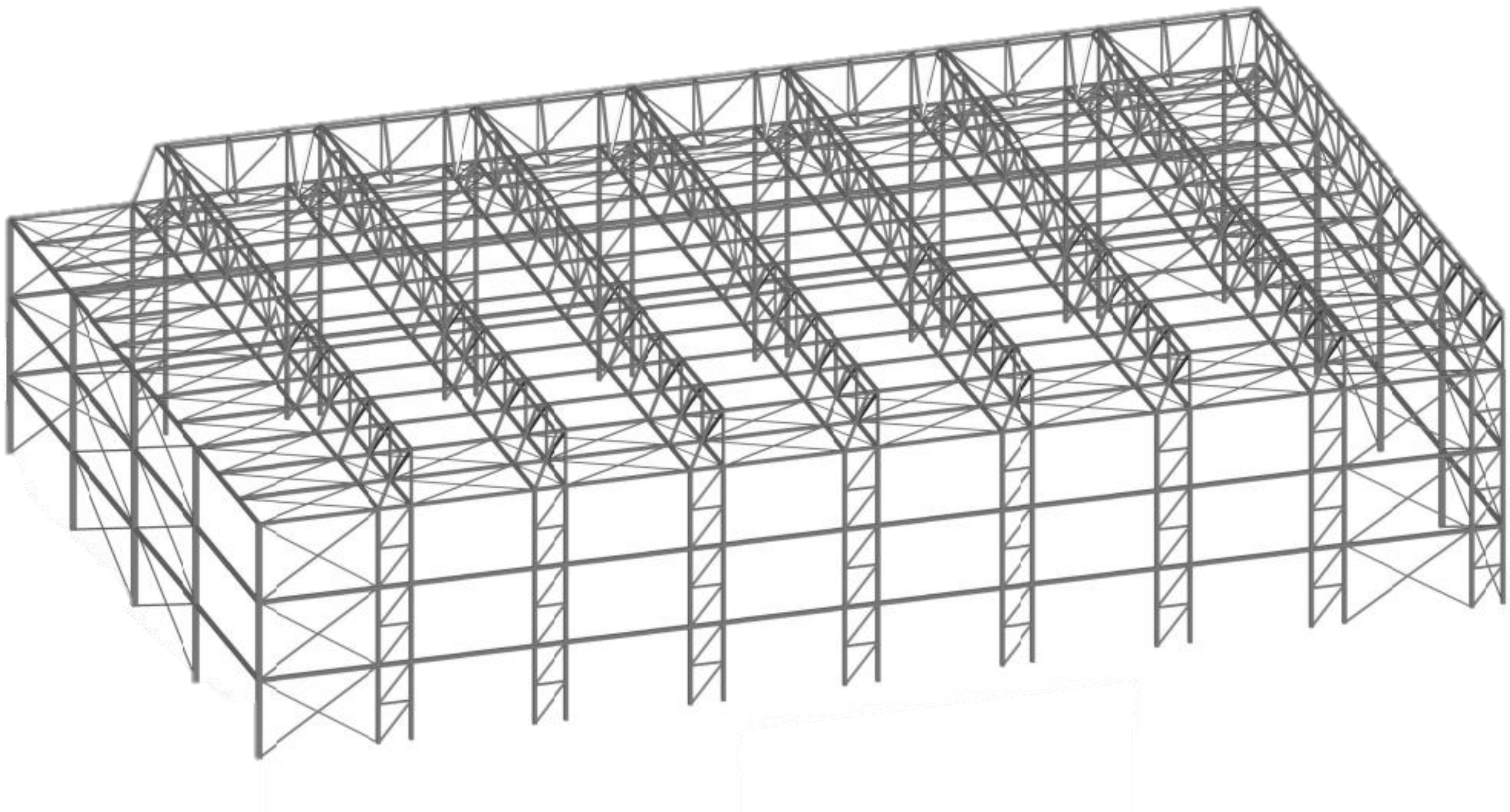


Figure 13: Extract of the connection of nodes.

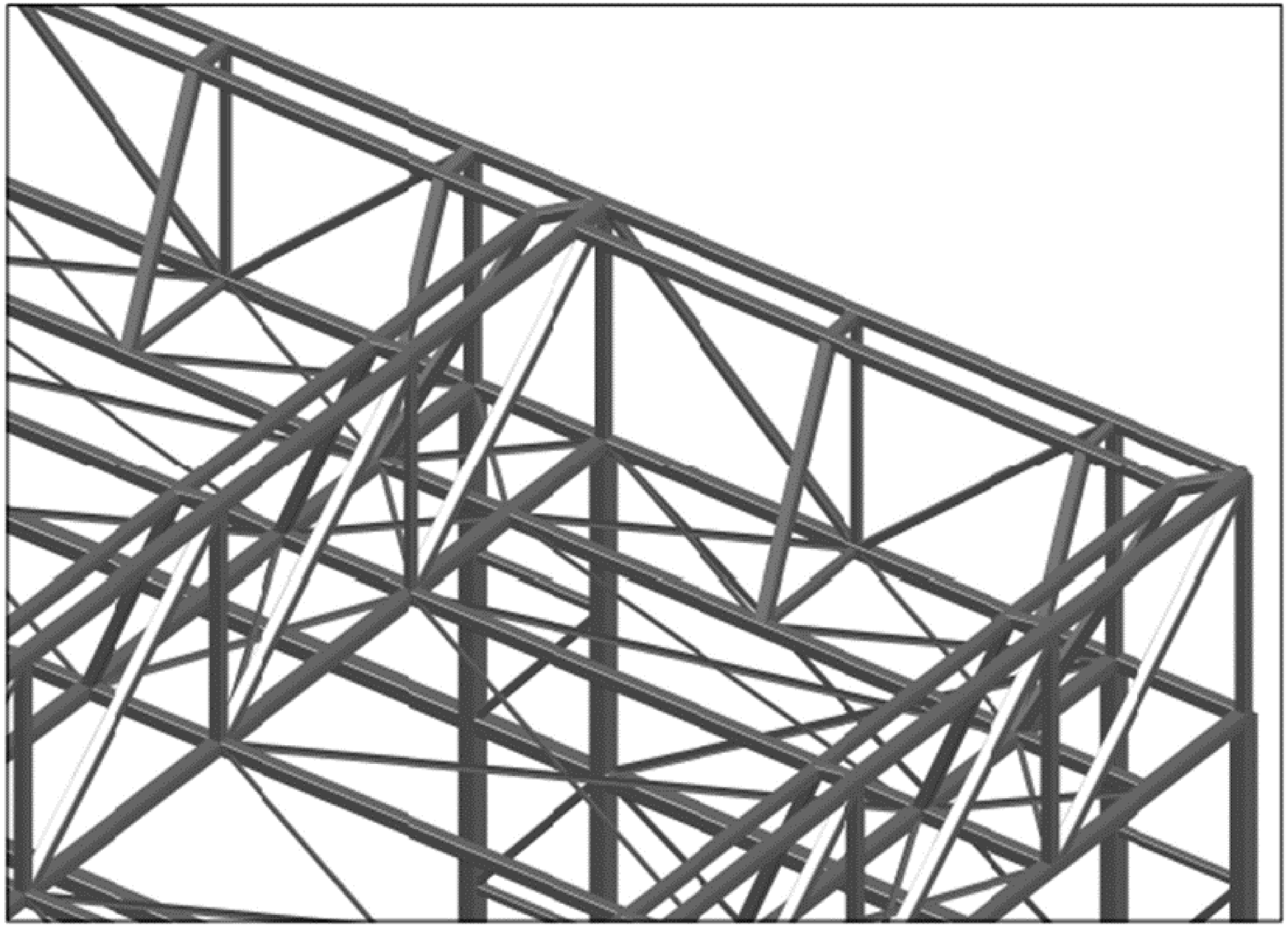


Figure 14: Top plane of the metallic structure with marked frames.

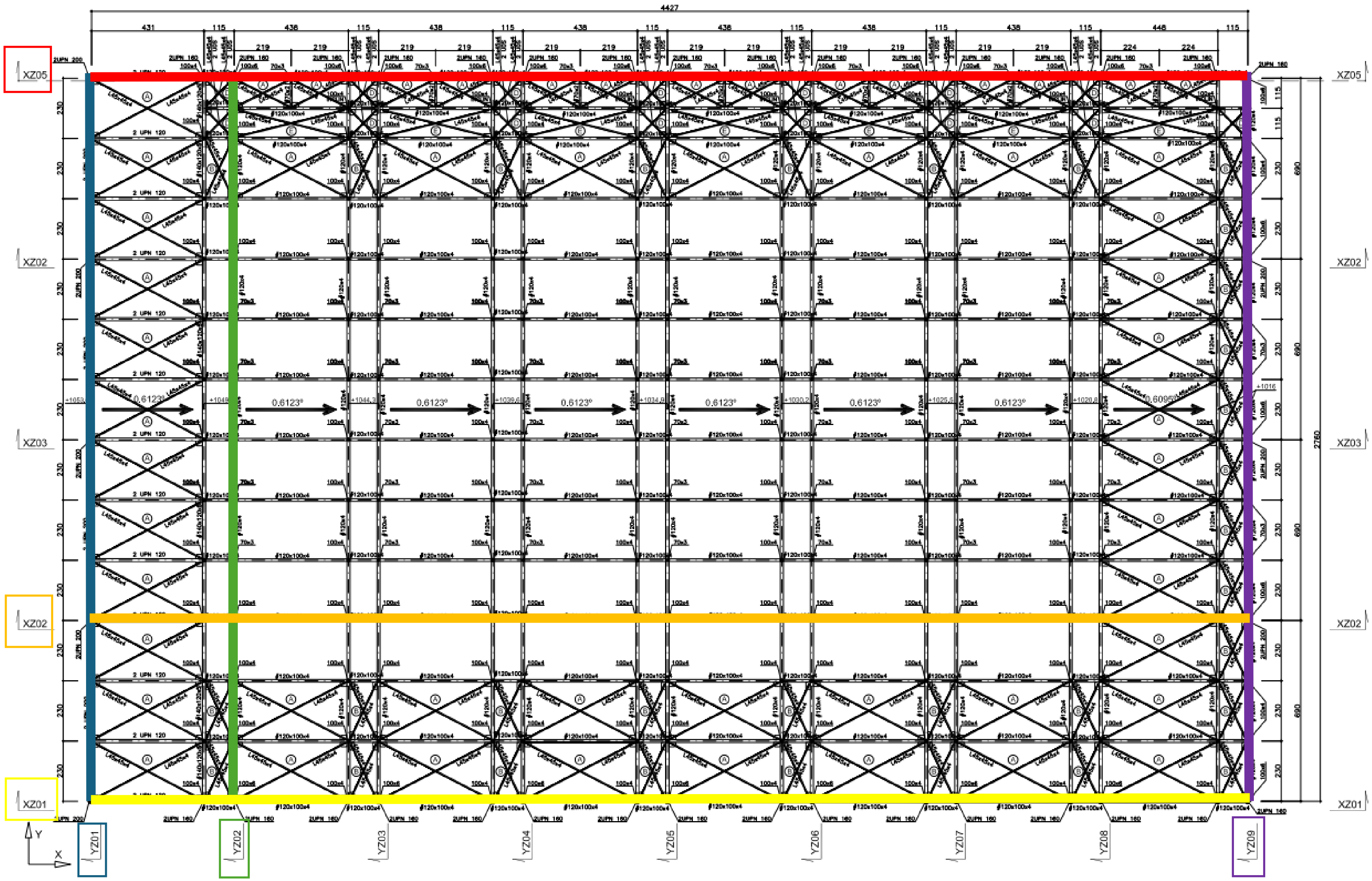


Figure 15: Front plane of frame XZ01.

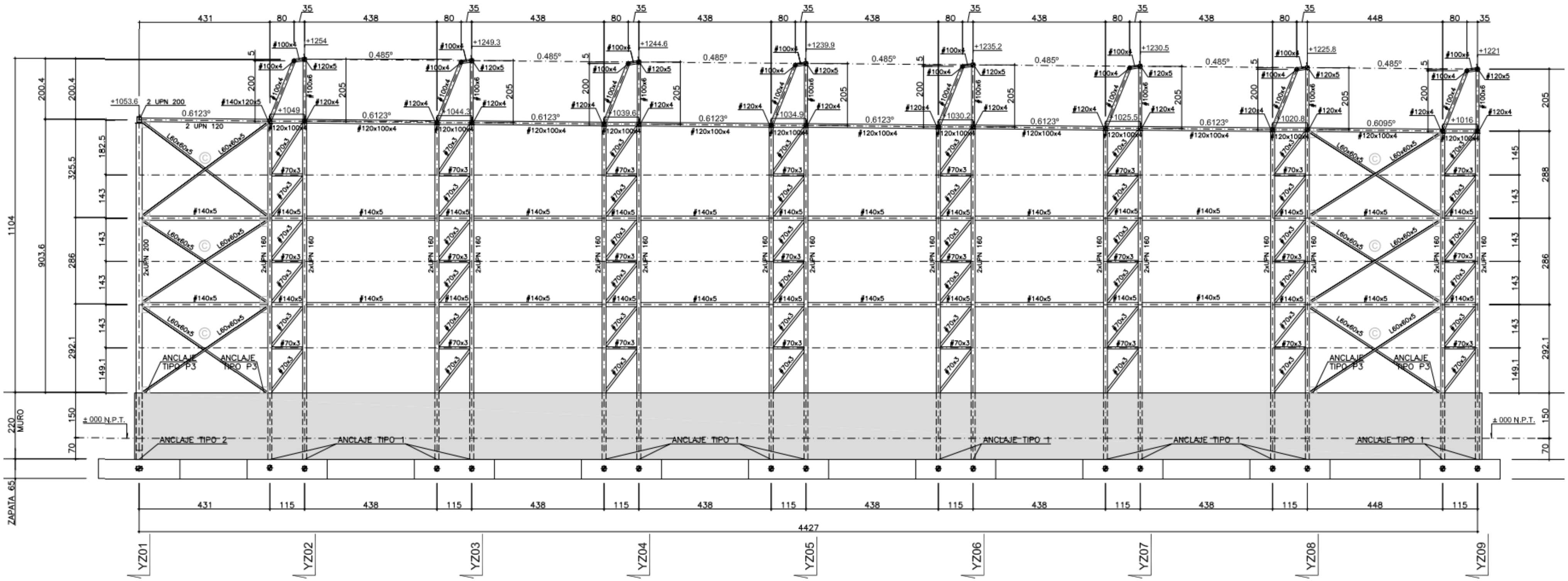


Figure 16: Front plane of frame XZ02.

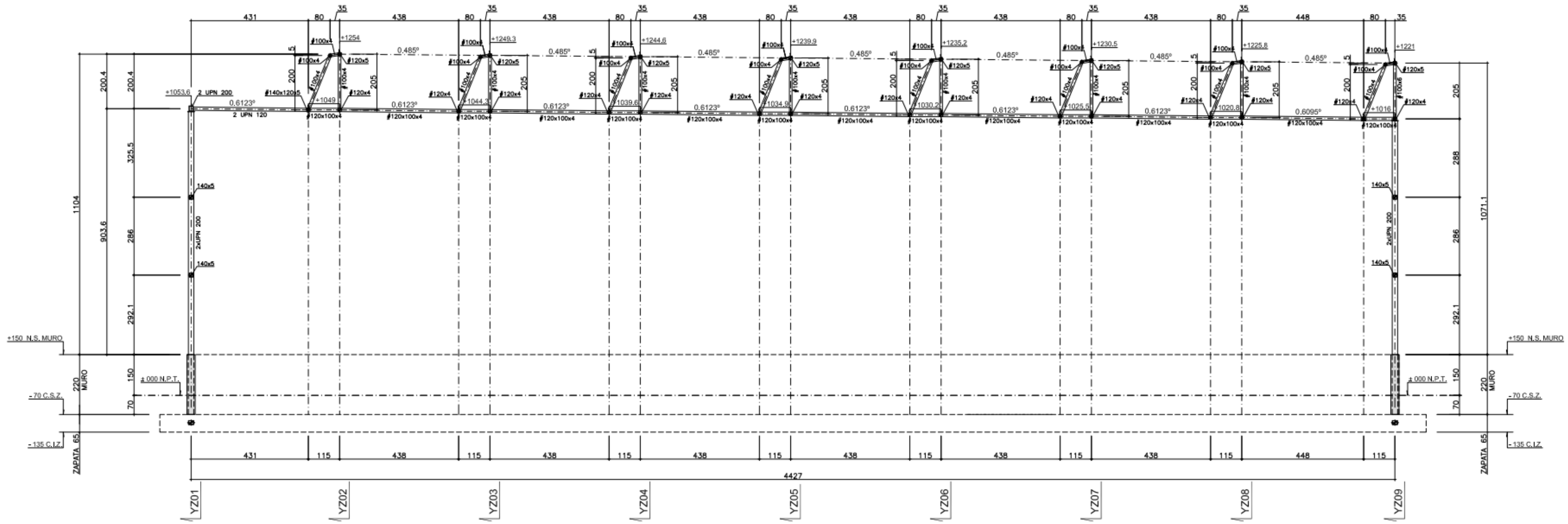


Figure 17: Front plane of frame XZ05.

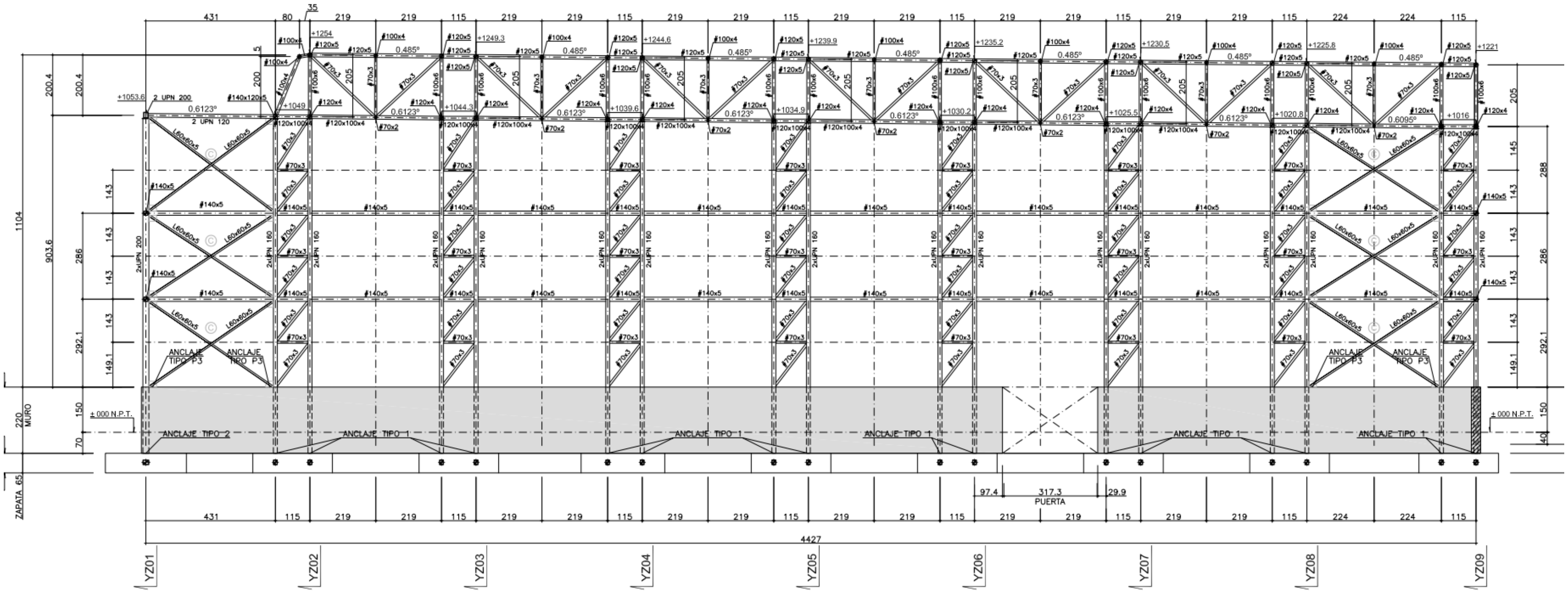


Figure 18: Front plane of frame YZ01.

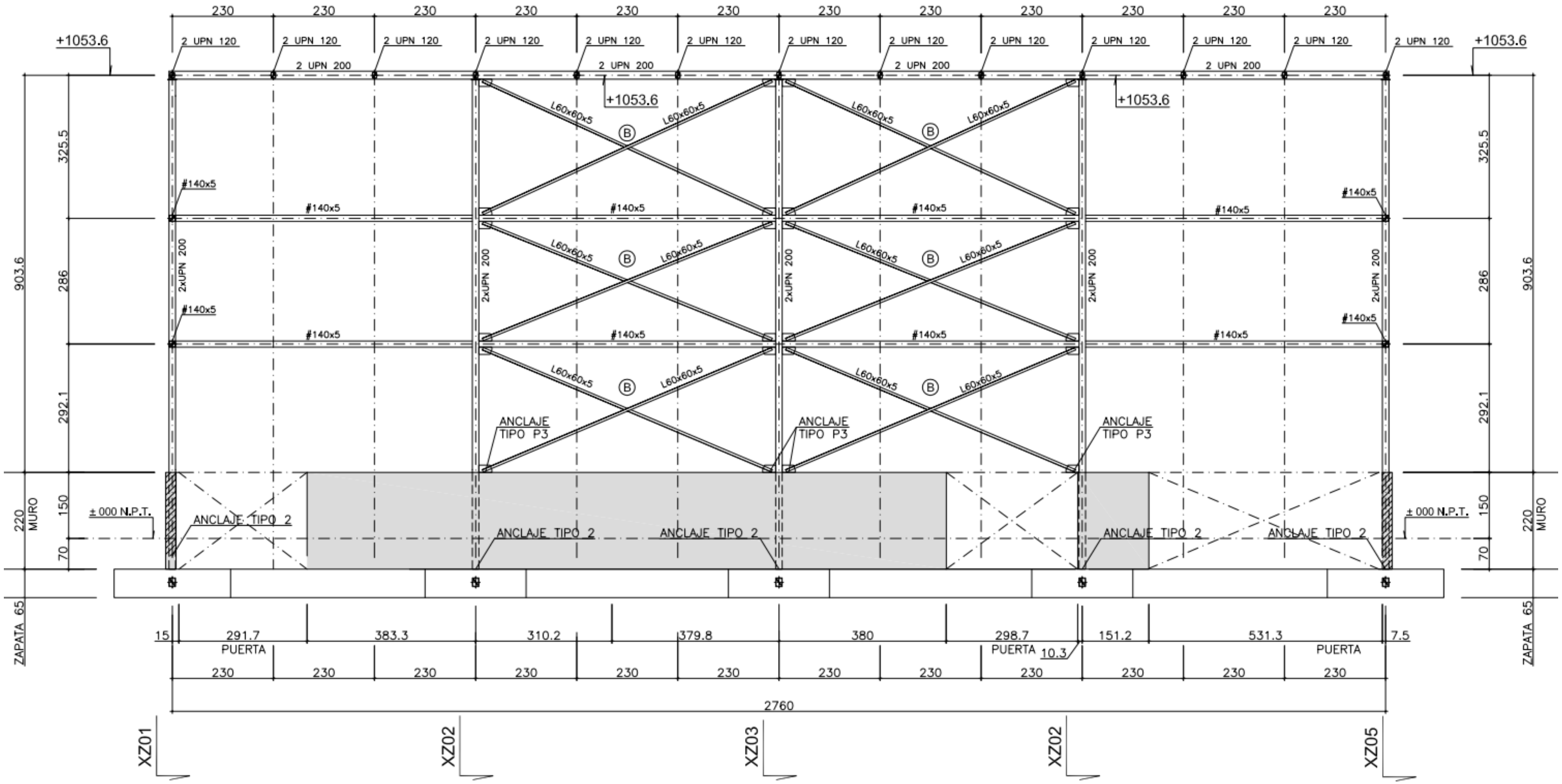


Figure 19: Front plane of frame YZ02.

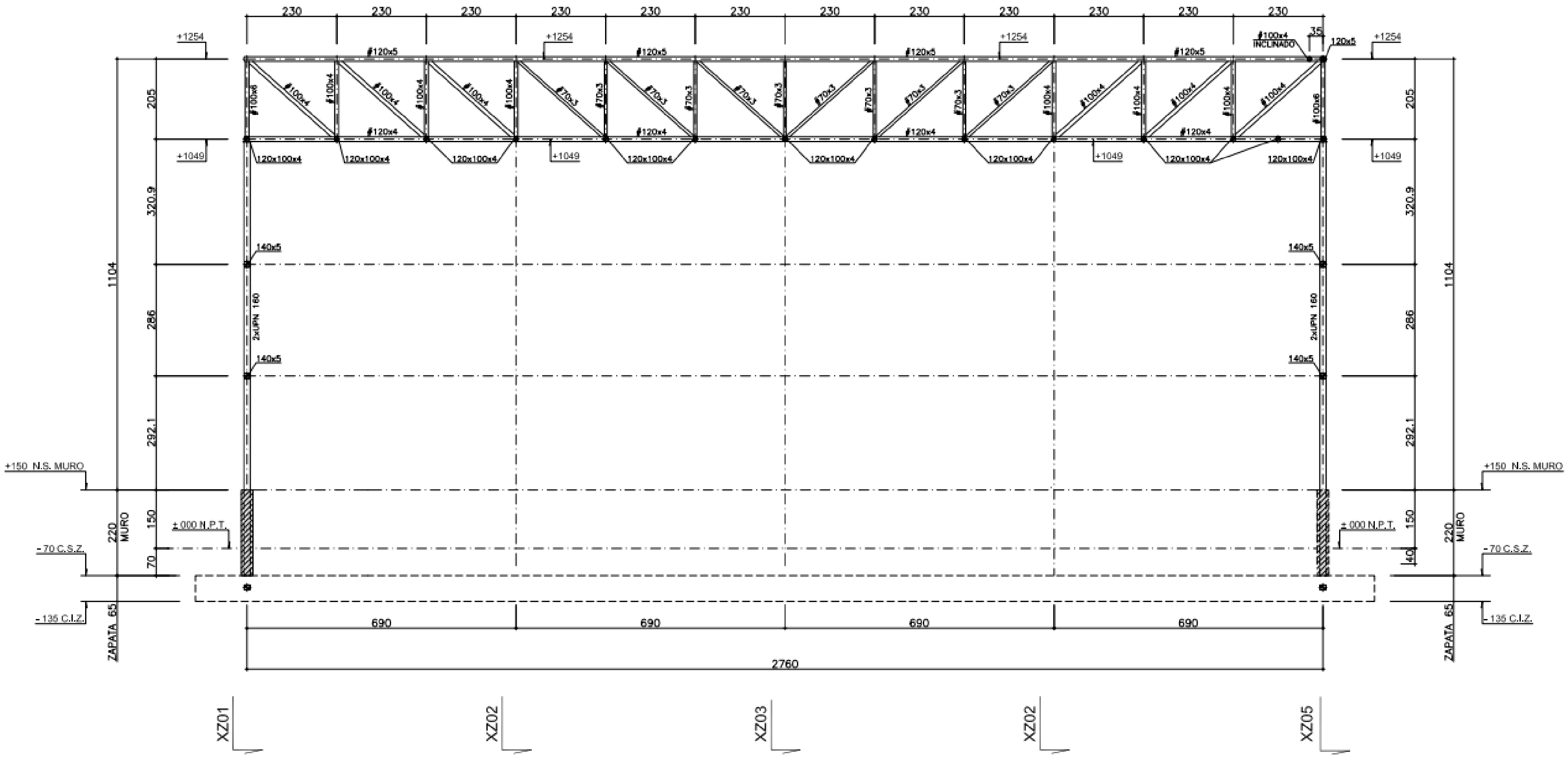
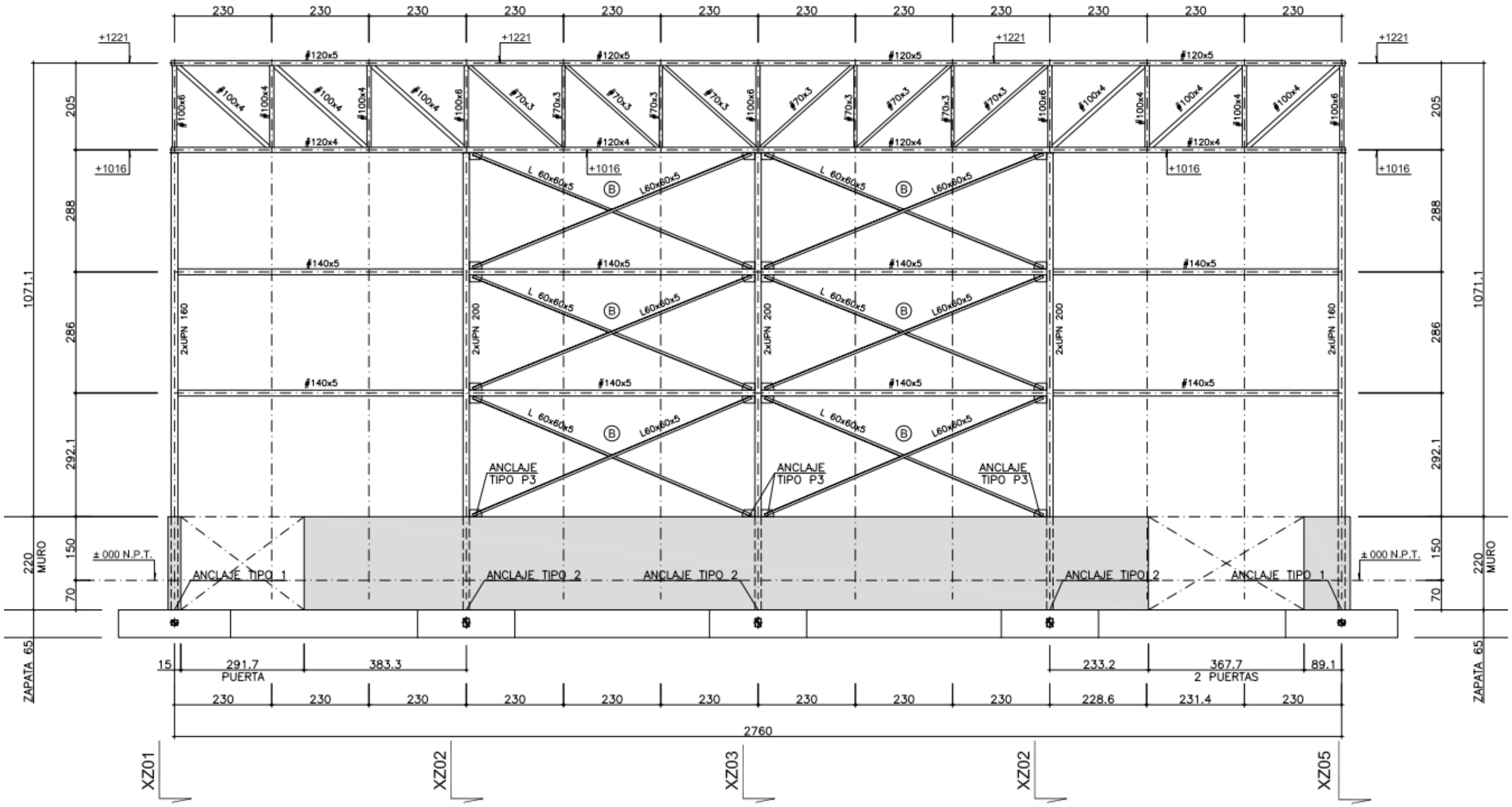


Figure 20: Front plane of frame YZ09.



In the study of all the frames, attention is placed on three conditions:

a. Maximum Von Mises stress cannot be bigger than fyd.
b. Maximum vertical displacement cannot be bigger than $\frac{L}{300}$.
c. Maximum horizontal displacement cannot be bigger than $\frac{H}{500}$.

The material studied is S235, giving a fyd of $f_{yd} = \frac{f_{yk}}{y_M} = \frac{235}{1,15} = 204N/mm^2$. The maximum value obtained for Von Mises cannot be higher than 204MPa.

Figure 21: Undeformed shape of XZ01.

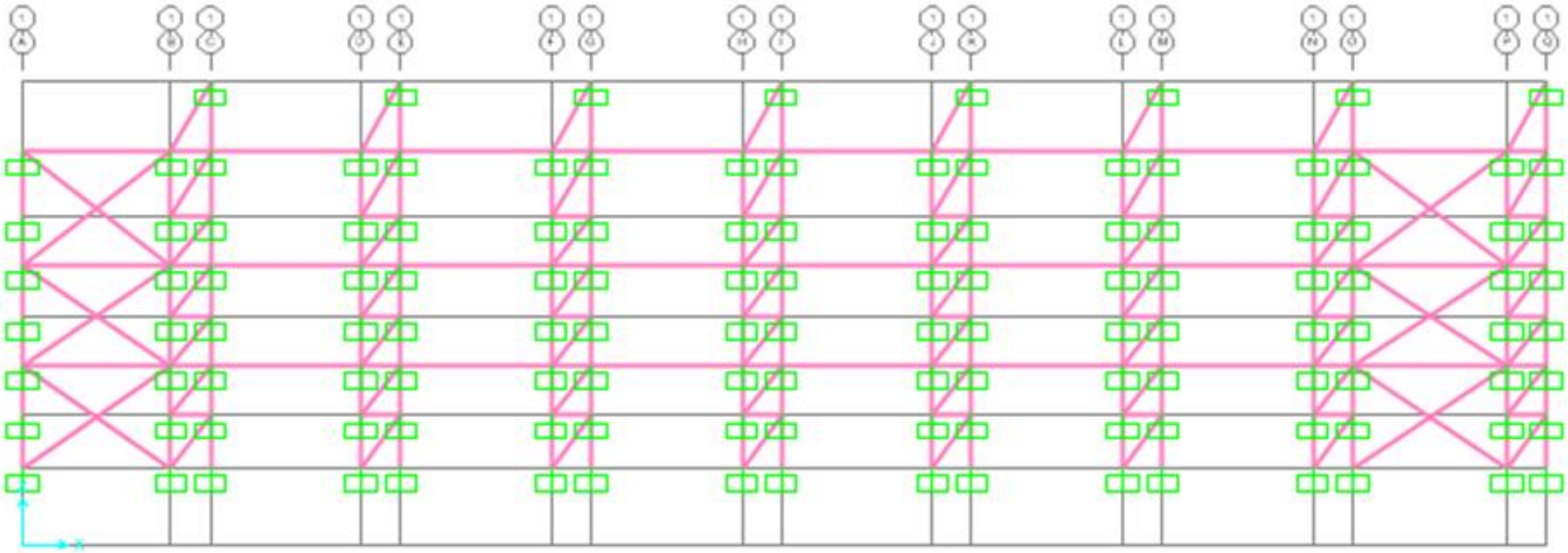


Figure 22: Deformed shape of XZ01 under the combination ULS 1.

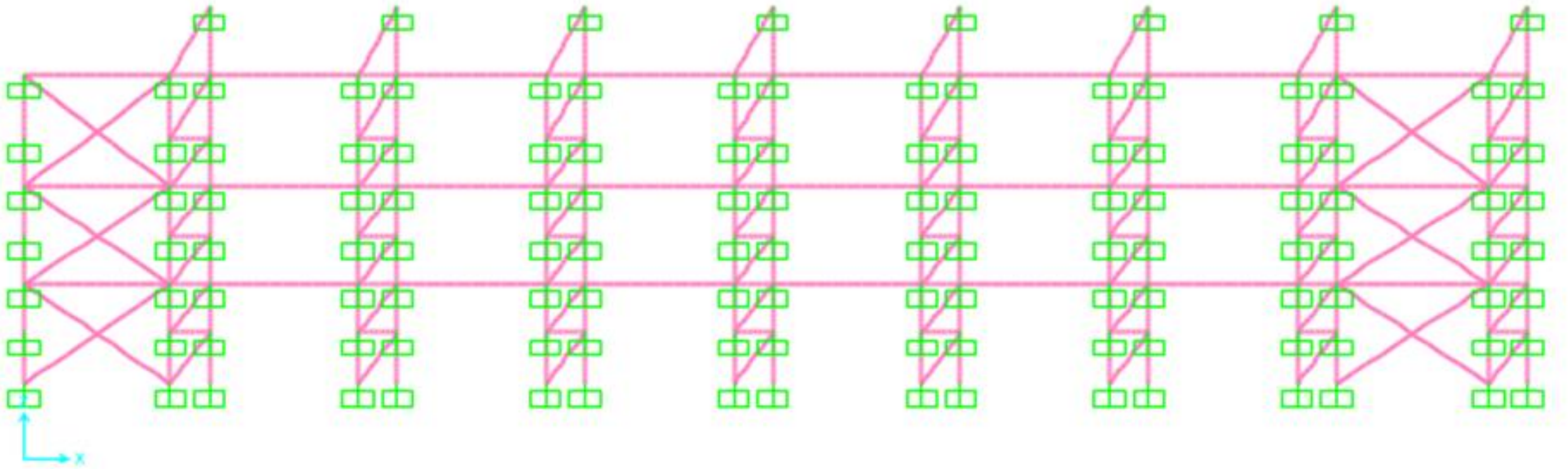


Figure 23: Diagram of SVM maximum stresses for combination ULS 1 for XZ01.

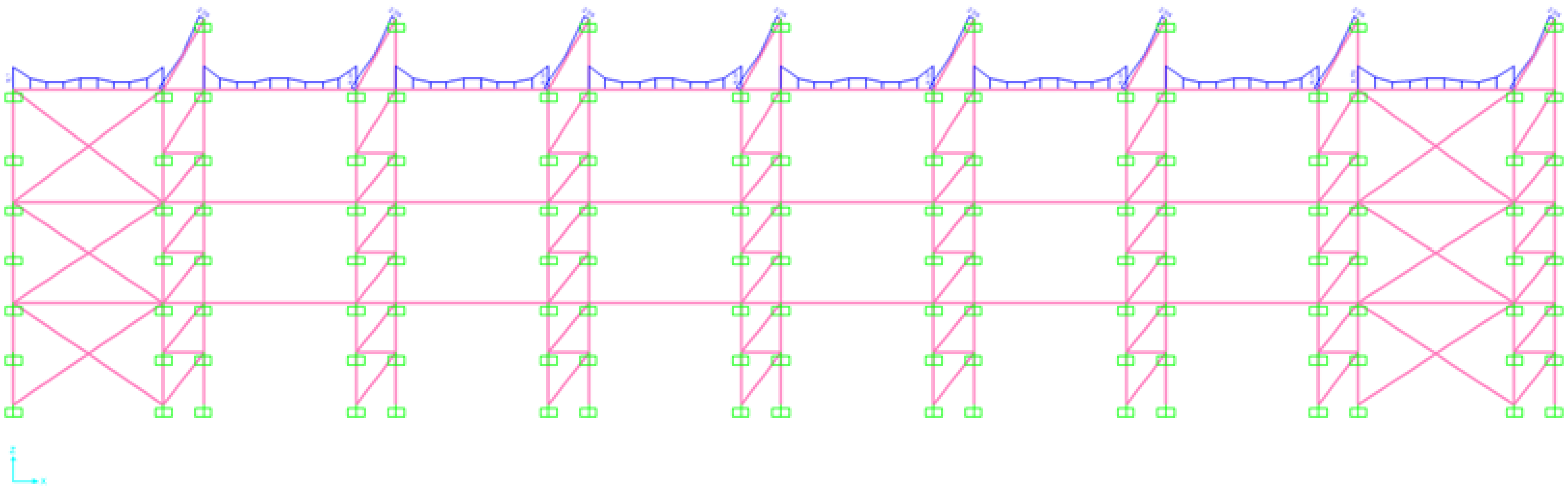
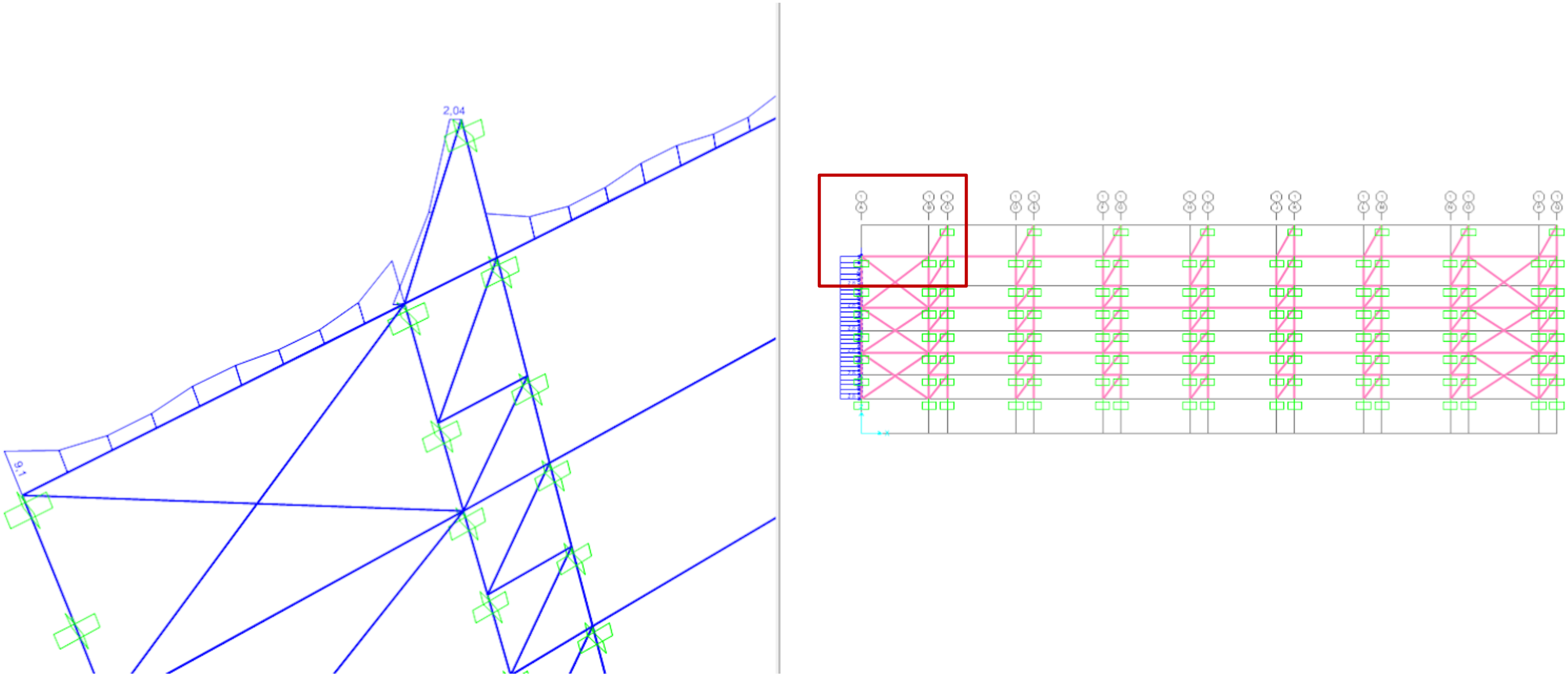


Figure 24: Extract of the most critical point of the SVM diagram for XZ01.



FRAME XZ01:

The maximum value for Von Mises is given by the first combination of Ultimate limit states, not involving the wind. The maximum value is $9{,}11 MPa < 204 MPa$.

For the displacement, the joints were studied, given the following results:

Table 11: Joint displacements in frame XZ01 under load combination ULS 1.

TABLE: Joint Displacements								
Joint	**OutputCase**	**CaseType**	**U1**	**U2**	**U3**	**R1**	**R2**	**R3**
Text	Text	Text	mm	mm	mm	Radians	Radians	Radians
2	ULS1	Combination	0	0	0	0	0	0
2	ULS2	Combination	0	0	0	0	0	0
2	ULS4	Combination	0	0	0	0	0	0
2	SLS1	Combination	0	0	0	0	0	0
2	SLS3	Combination	0	0	0	0	0	0
3	ULS1	Combination	0	0	0	0	0	0
3	ULS2	Combination	0	0	0	0	0	0
3	ULS4	Combination	0	0	0	0	0	0
3	SLS1	Combination	0	0	0	0	0	0
3	SLS3	Combination	0	0	0	0	0	0

Giving a value of zero in all displacements.

Figure 25: Undeformed shape of XZ02.

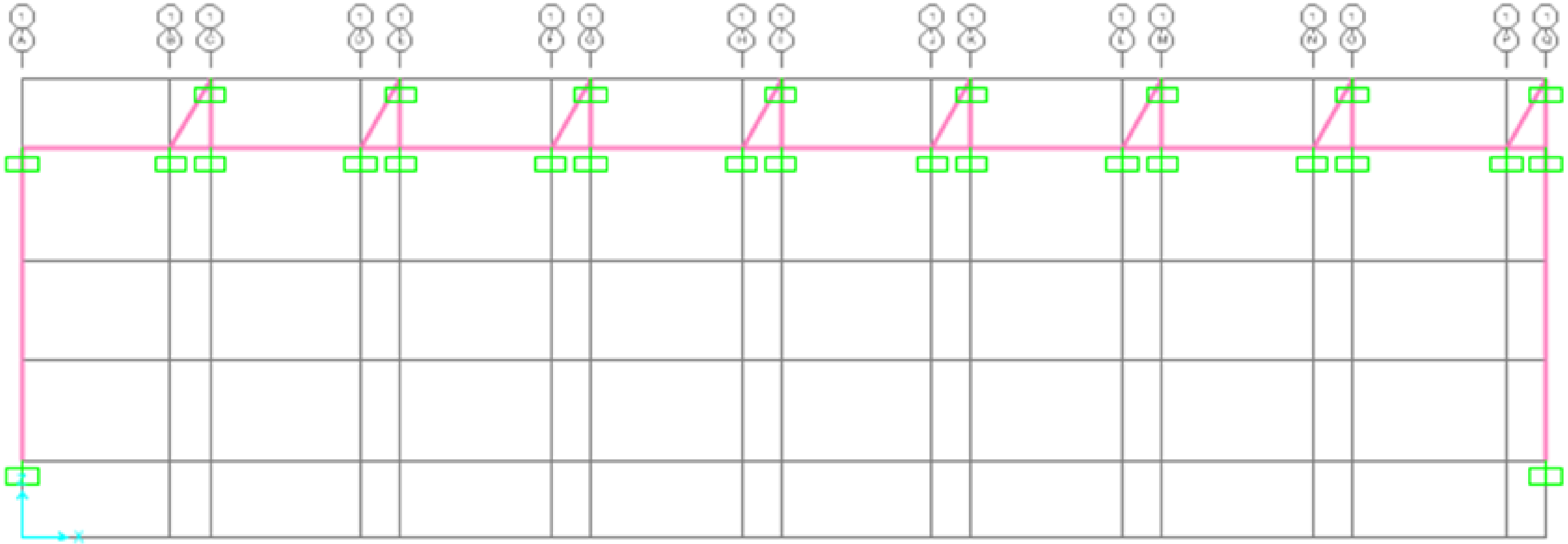
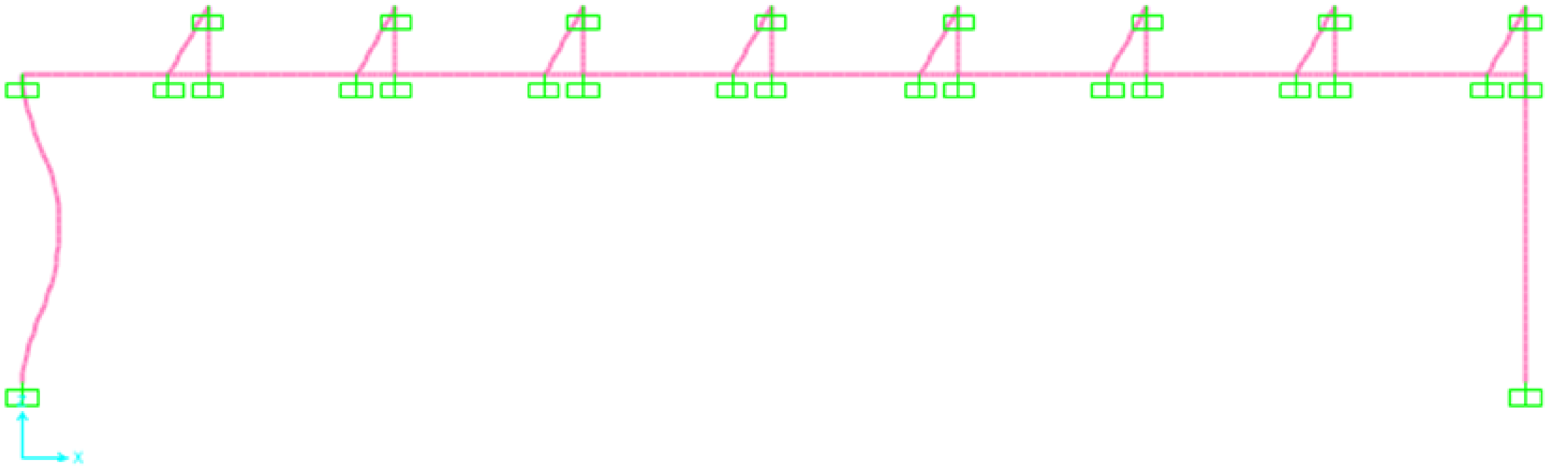


Figure 26: Deformed shape of XZ02 under combination ULS 3.



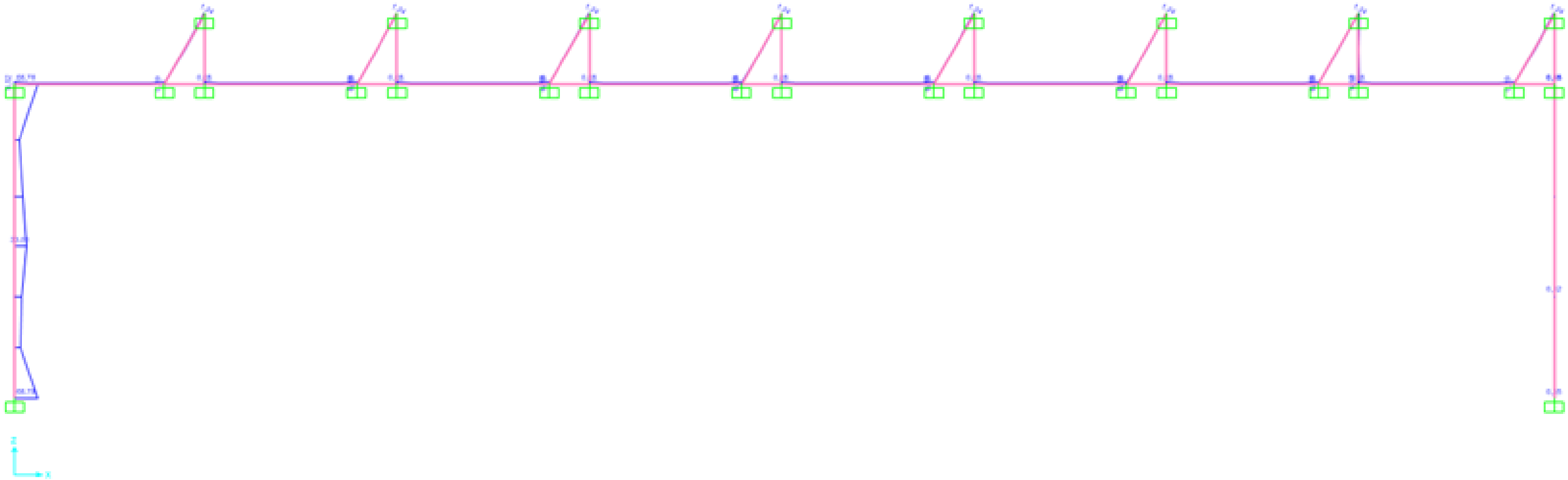
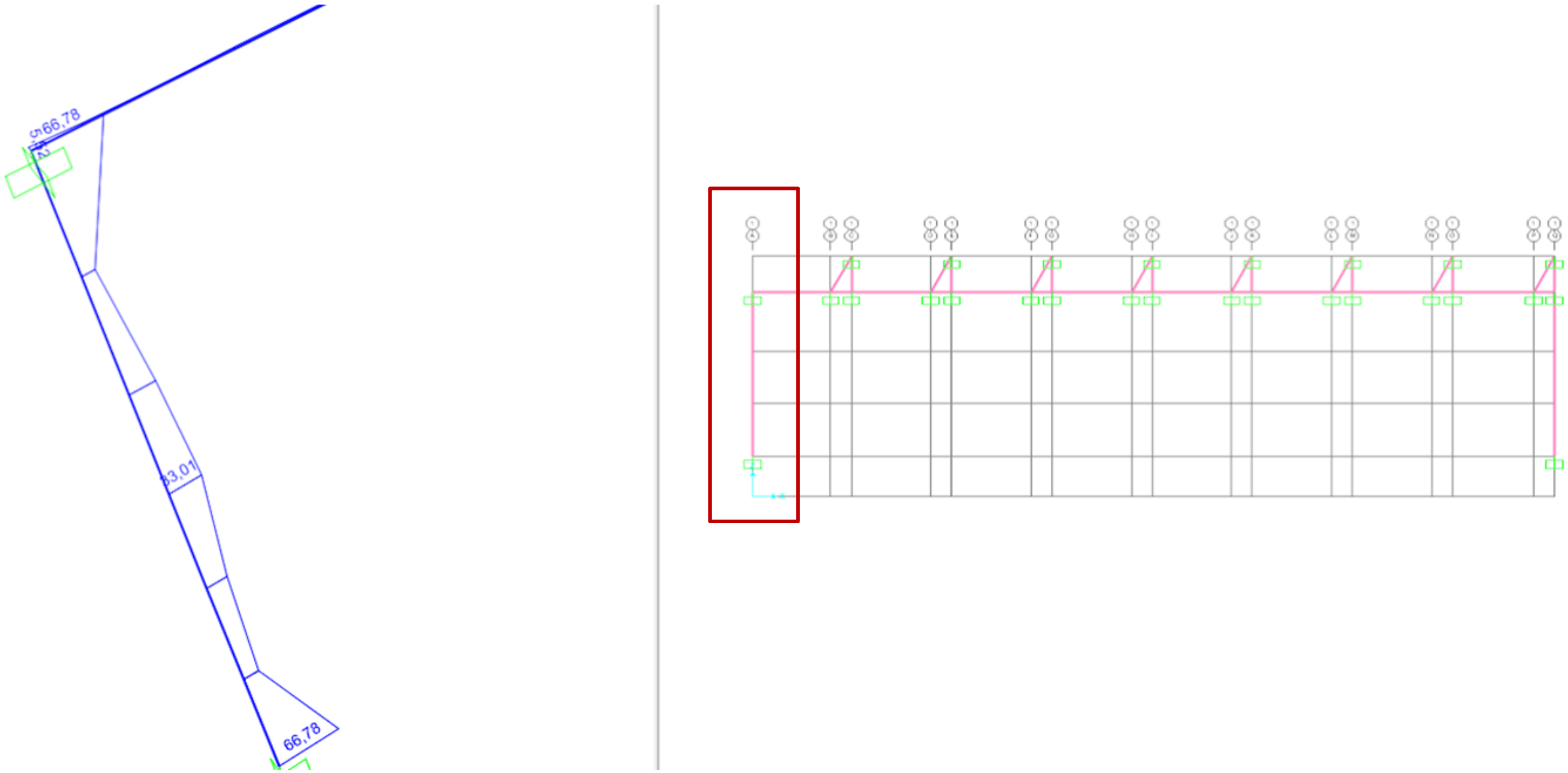


Figure 27: Diagram of SVM maximum stresses for combination ULS 3 for XZ02.

Figure 28: Extract of the most critical point of the SVM diagram for XZ02.



FRAME XZ02

In this frame the maximum value for Von Mises comes from the third combination of Ultimate limit states, which involves the wind.

Overall, the highest value is $66{,}78 MPa < 204 MPa$.

The displacement of joints gave the following results:

Table 12: Joint displacements in frame XZ02 under load combination ULS 3.

TABLE: Joint Displacements								
Joint	OutputCase	CaseType	U1	U2	U3	R1	R2	R3
4	ULS4	Combination	2,371157	0	-0,004656	0	-0,000603	0
4	ULS1	Combination	0	0	-0,004656	0	0	0
100	ULS1	Combination	0	0	-0,004656	0	0	0
100	ULS4	Combination	0	0	-0,004656	0	0	0
3	ULS4	Combination	2,142618	0	-0,004419	0	0,000723	0
3	ULS1	Combination	0	0	-0,004419	0	0	0
99	ULS1	Combination	0	0	-0,004419	0	0	0
99	ULS4	Combination	0	0	-0,004419	0	0	0
4	ULS3	Combination	4,742314	0	-0,003449	0	-0,001205	0
4	SLS3	Combination	3,161543	0	-0,003449	0	-0,000804	0
4	DEAD	LinStatic	0	0	-0,003449	0	0	0
4	SLS1	Combination	0	0	-0,003449	0	0	0
100	DEAD	LinStatic	0	0	-0,003449	0	0	0
100	ULS3	Combination	0	0	-0,003449	0	0	0
100	SLS1	Combination	0	0	-0,003449	0	0	0
100	SLS3	Combination	0	0	-0,003449	0	0	0
3	ULS3	Combination	4,285235	0	-0,003274	0	0,001447	0
3	SLS3	Combination	2,856824	0	-0,003274	0	0,000964	0
3	DEAD	LinStatic	0	0	-0,003274	0	0	0
3	SLS1	Combination	0	0	-0,003274	0	0	0
99	DEAD	LinStatic	0	0	-0,003274	0	0	0
99	ULS3	Combination	0	0	-0,003274	0	0	0
99	SLS1	Combination	0	0	-0,003274	0	0	0
99	SLS3	Combination	0	0	-0,003274	0	0	0
2	DEAD	LinStatic	0	0	0	0	0	0
2	ULS1	Combination	0	0	0	0	0	0

The highest value for a horizontal deflection is $4{,}7423 mm$ in joint 4 in the third combination of ULS. All combinations that involve vertical deflection give a value of zero.

For the case of XZ02, H is $10{,}70 m$, therefore $4{,}7423 < 21{,}4 mm$.

Figure 29: Undeformed shape of XZ05.

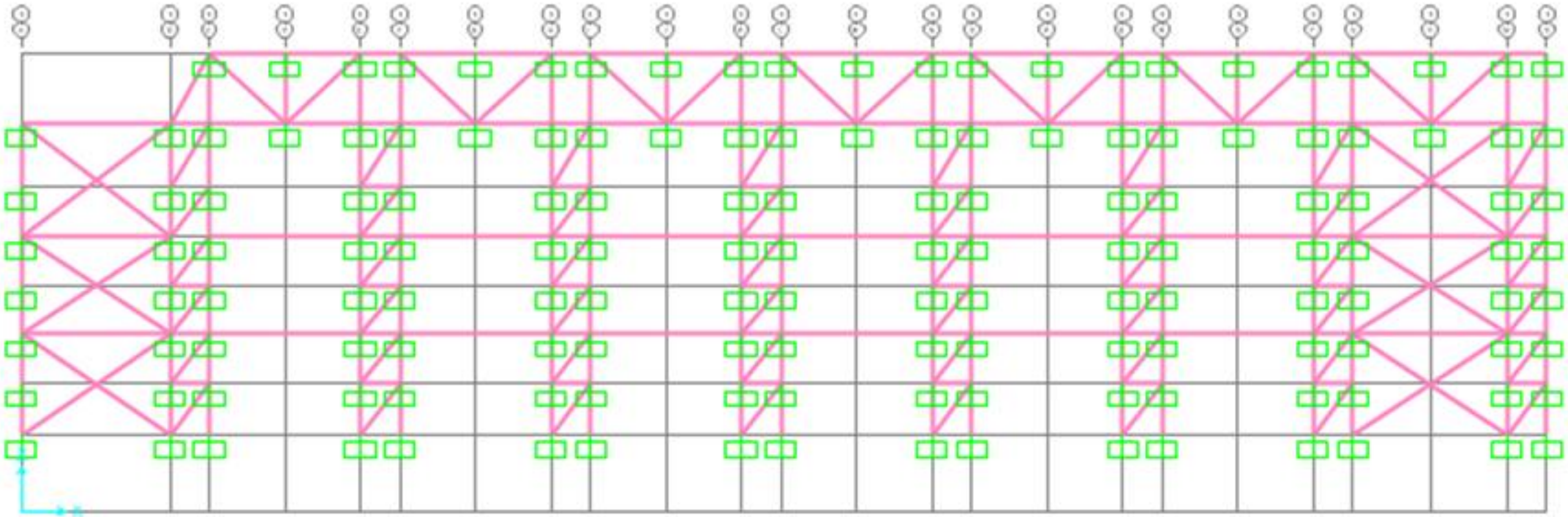


Figure 30: Deformed shape of XZ05 with load combination ULS 4.

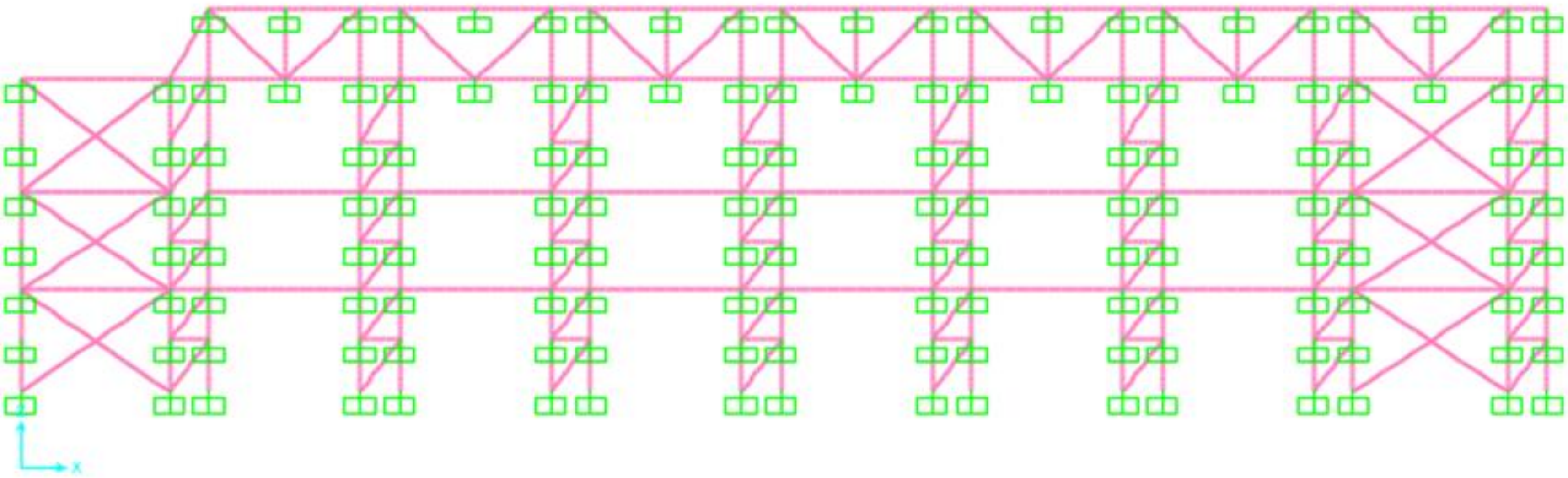


Figure 31: Diagram of SVM maximum stresses for combination ULS 4 for XZ05.

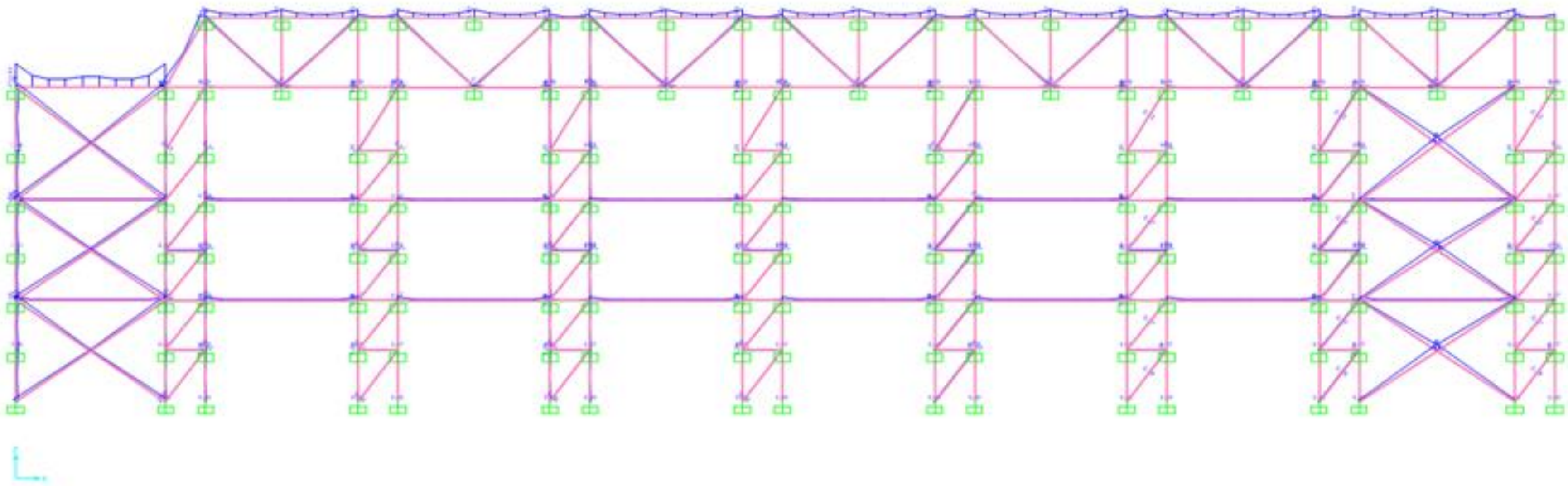
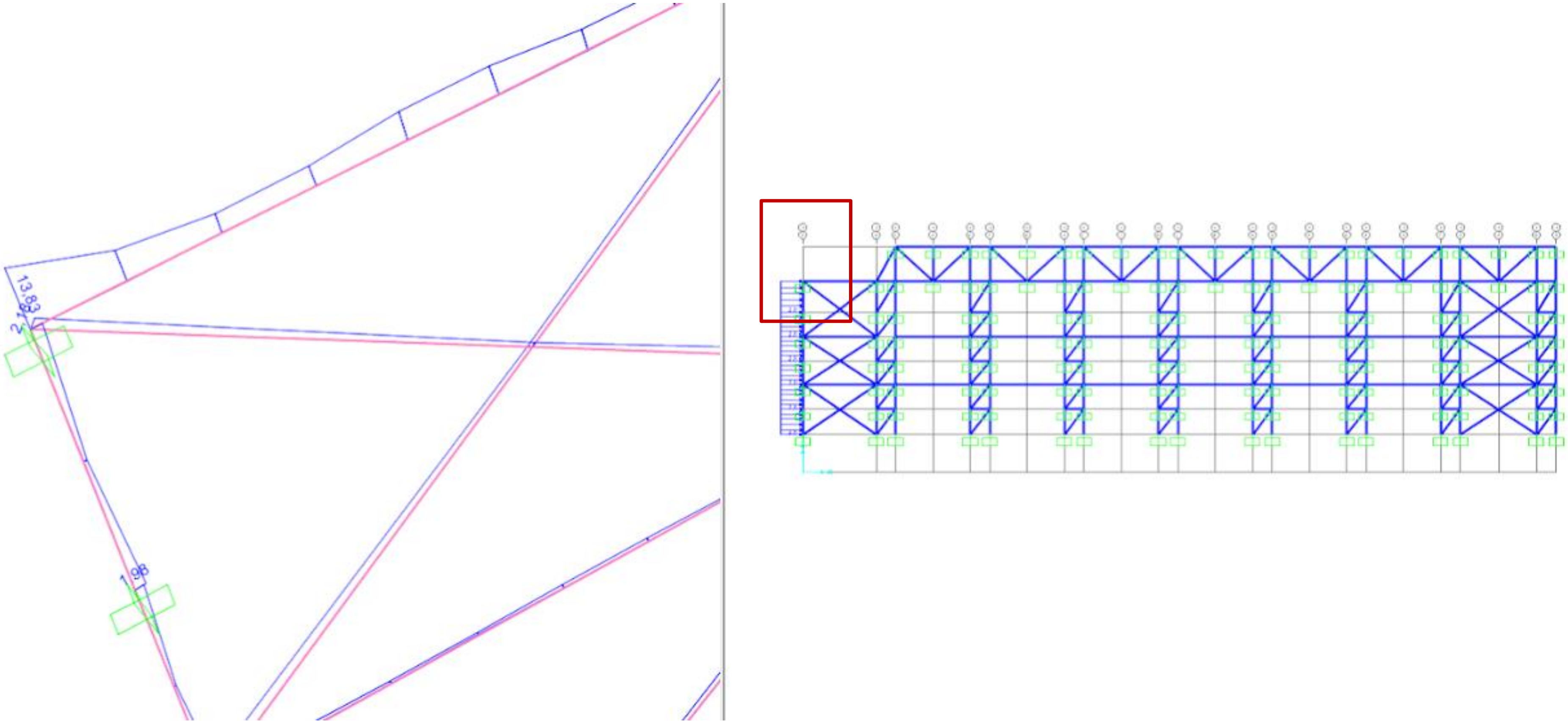


Figure 32: Extract of the most critical point of the SVM diagram for XZ05.



FRAME XZ05

For this frame the highest value of Von Mises is given by the fourth combination of ULS. A value of $13{,}83MPa < 204MPa$.

When it comes to joint displacement, it is a similar case to XZ01, where all values are zero.

Table 13: Joint displacements in frame XZ05 under load combination ULS 4.

TABLE: Joint Displacements								
Joint	**OutputCase**	**CaseType**	**U1**	**U2**	**U3**	**R1**	**R2**	**R3**
Text	Text	Text	mm	mm	mm	Radians	Radians	Radians
2	ULS1	Combination	0	0	0	0	0	0
2	ULS2	Combination	0	0	0	0	0	0
2	ULS4	Combination	0	0	0	0	0	0
2	SLS1	Combination	0	0	0	0	0	0
2	SLS3	Combination	0	0	0	0	0	0
3	ULS1	Combination	0	0	0	0	0	0
3	ULS2	Combination	0	0	0	0	0	0
3	ULS4	Combination	0	0	0	0	0	0
3	SLS1	Combination	0	0	0	0	0	0
3	SLS3	Combination	0	0	0	0	0	0

Figure 33: Undeformed shape of YZ01.

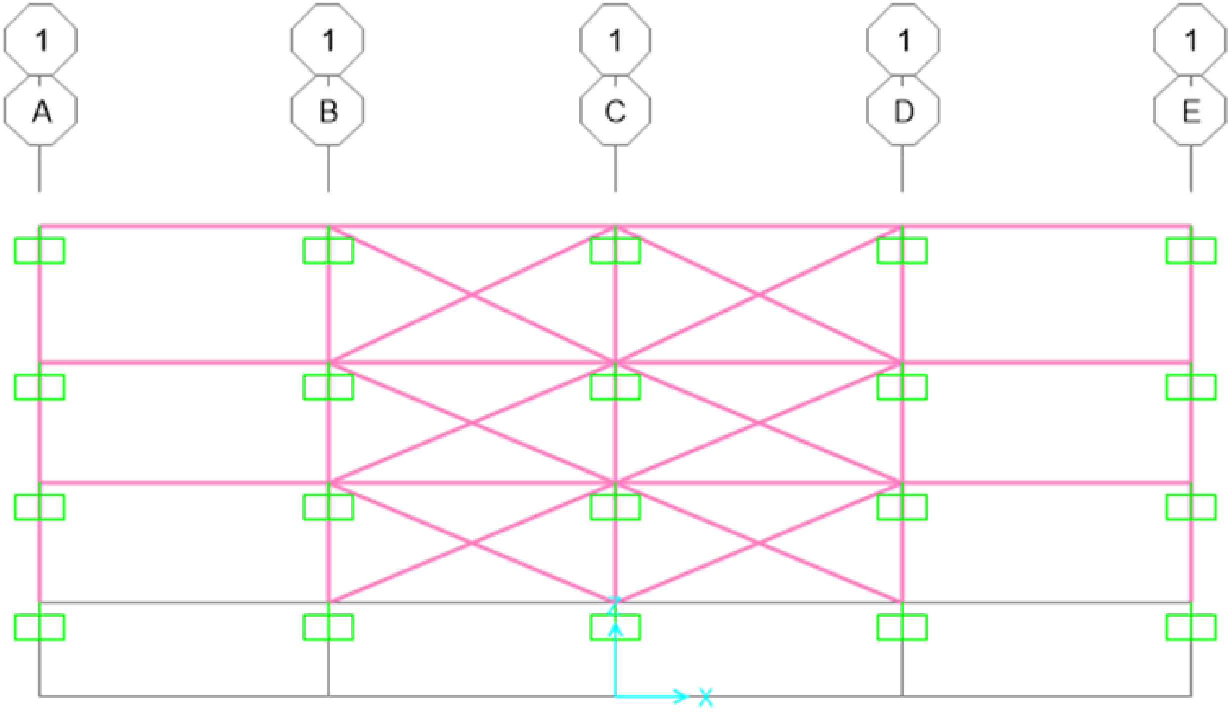


Figure 34: Deformed shape of YZ01 under load combination ULS 3.

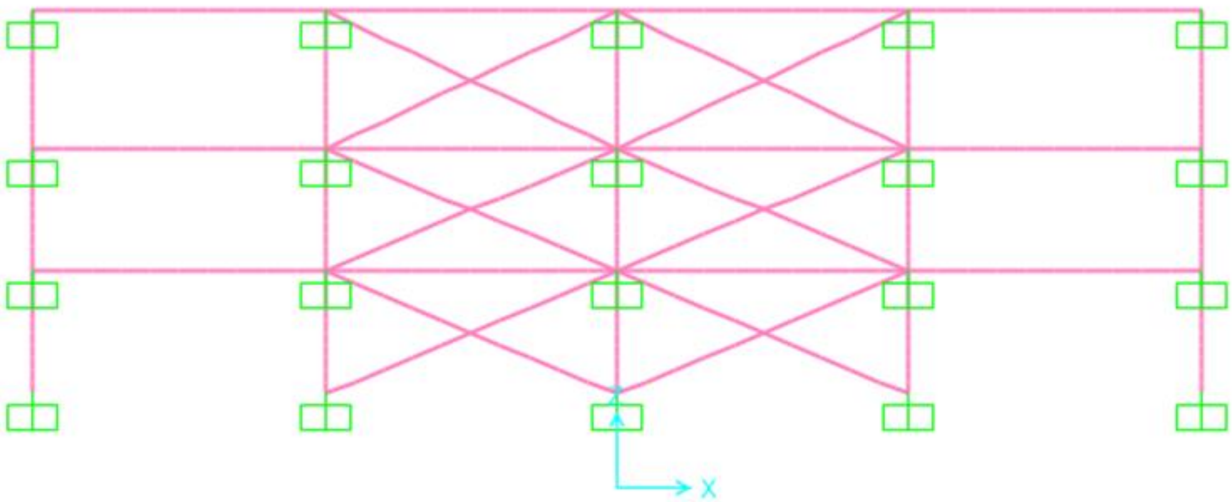


Figure 35: Diagram of SVM maximum stresses for combination ULS 3 for YZ01.

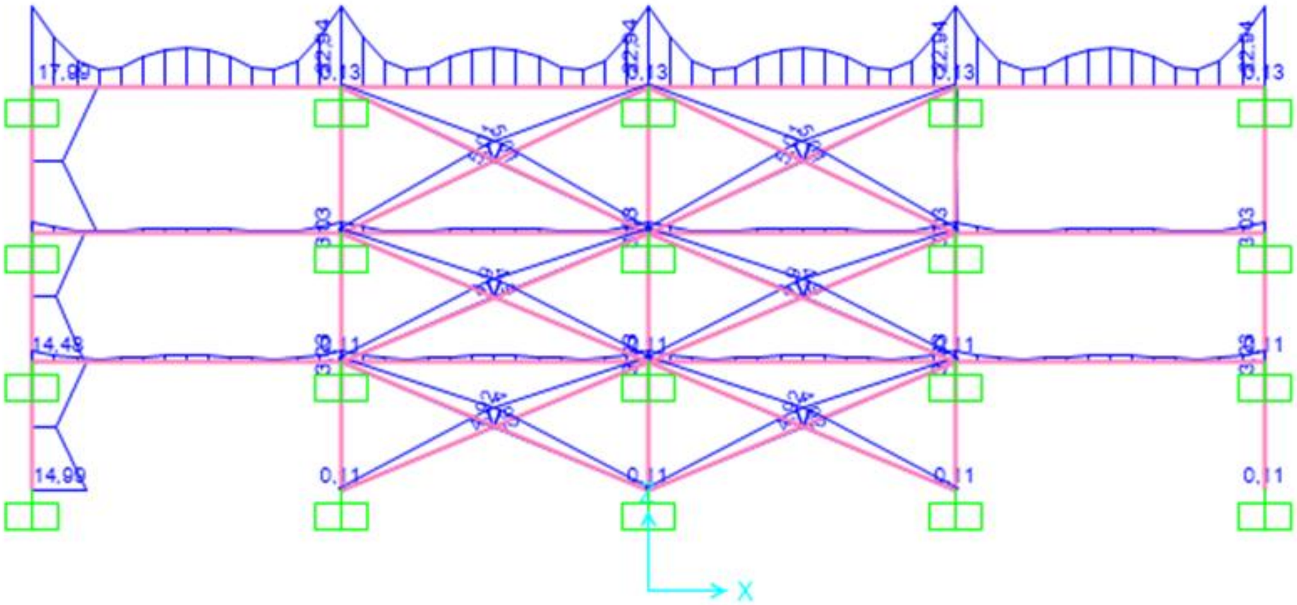
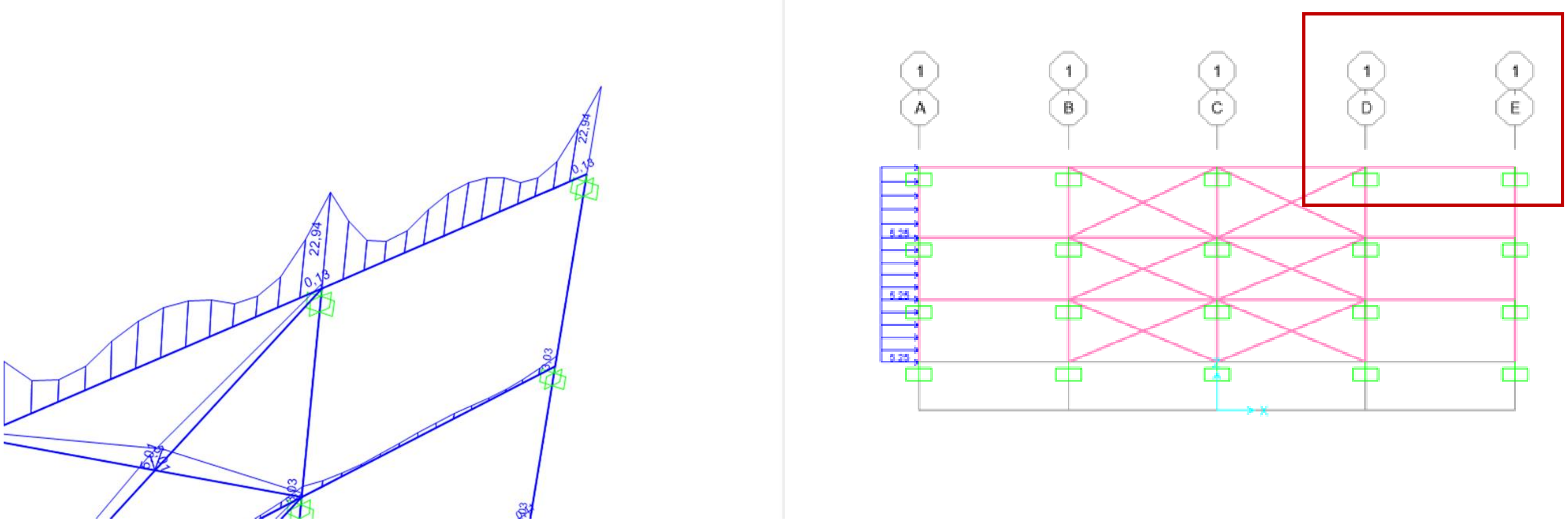


Figure 36: Extract of the most critical point of the SVM diagram for YZ01.



FRAME YZ01

The highest value for Von Mises is $22{,}94 MPa < 204 MPa$, given by combination three of the Ultimate limit states.

The joint displacements all give a value of zero.

Table 14: Joint displacements in frame YZ01 under load combination ULS 3.

TABLE: Joint Displacements								
Joint	**OutputCase**	**CaseType**	**U1**	**U2**	**U3**	**R1**	**R2**	**R3**
Text	Text	Text	mm	mm	mm	Radians	Radians	Radians
2	ULS1	Combination	0	0	0	0	0	0
2	ULS3	Combination	0	0	0	0	0	0
2	ULS4	Combination	0	0	0	0	0	0
2	SLS1	Combination	0	0	0	0	0	0
2	SLS3	Combination	0	0	0	0	0	0
3	ULS1	Combination	0	0	0	0	0	0
3	ULS3	Combination	0	0	0	0	0	0
3	ULS4	Combination	0	0	0	0	0	0
3	SLS1	Combination	0	0	0	0	0	0
3	SLS3	Combination	0	0	0	0	0	0

Figure 37: Undeformed shape of YZ02.

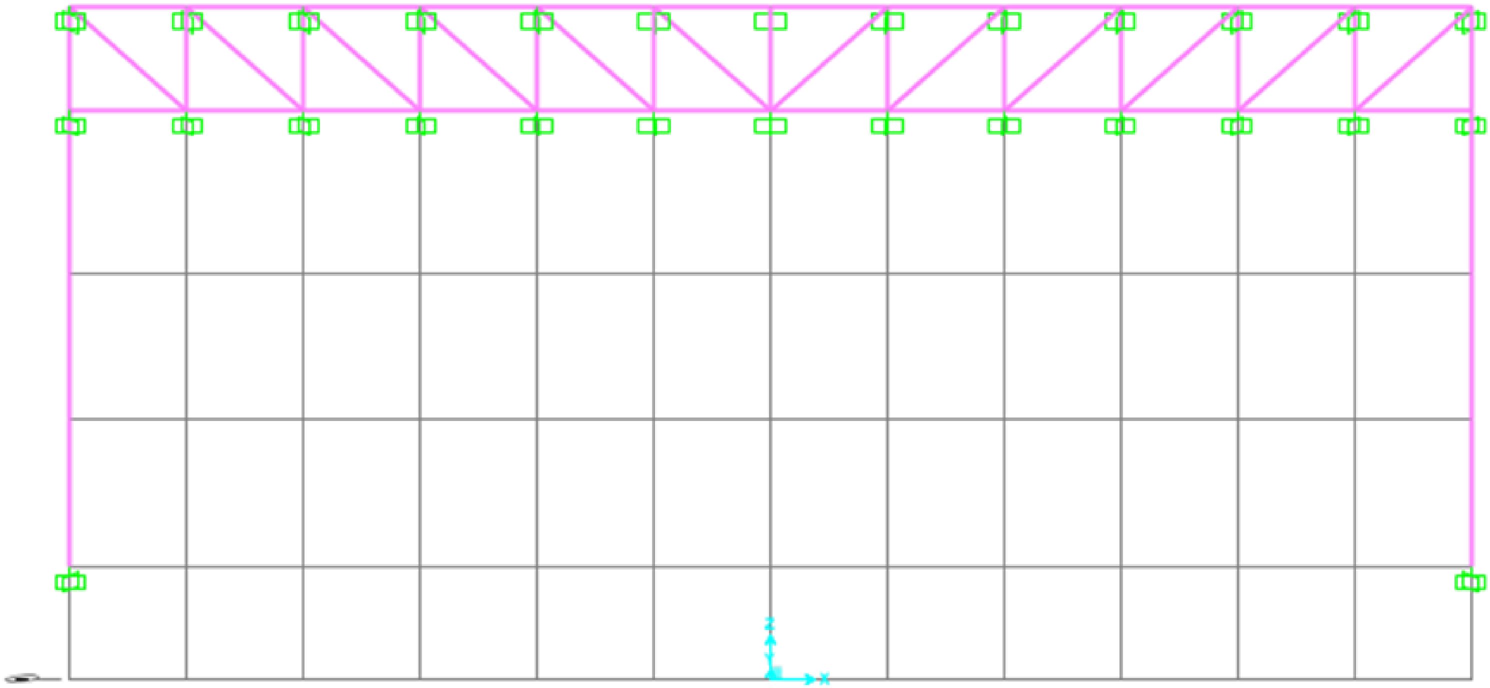


Figure 38: Deformed shape of YZ02 under load combination ULS 4.

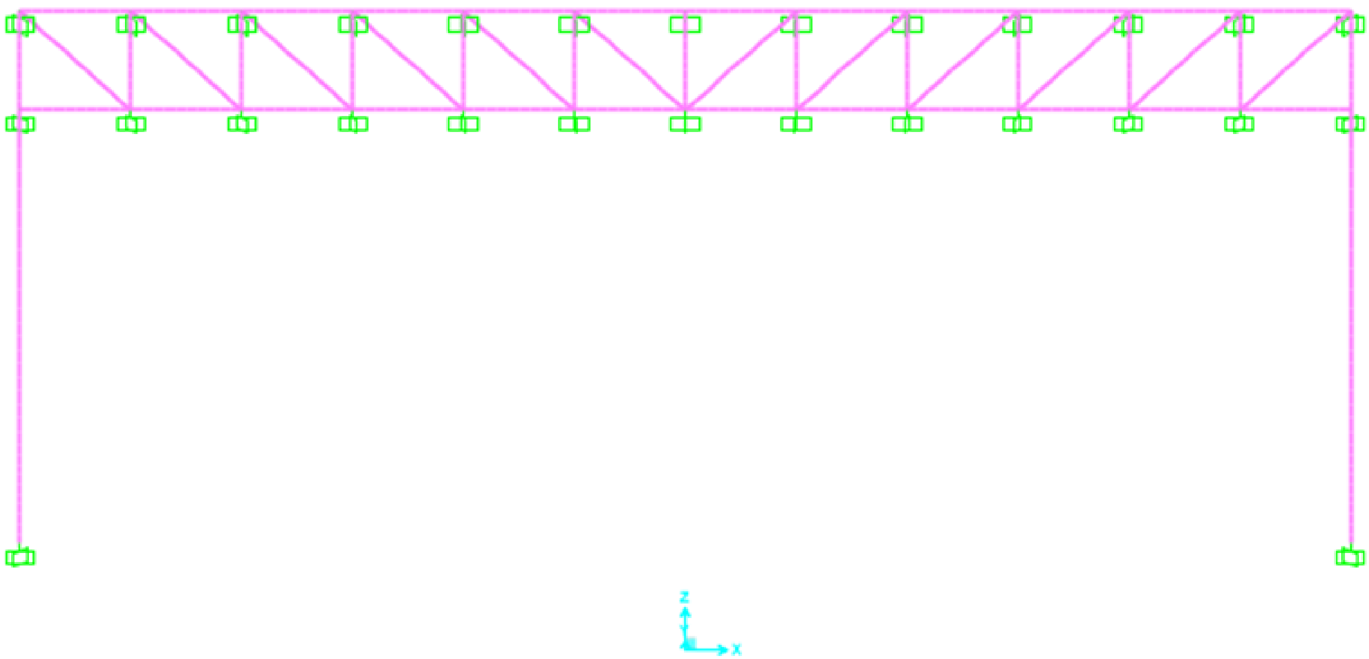


Figure 39: Diagram of SVM maximum stresses for combination ULS 4 for YZ02.

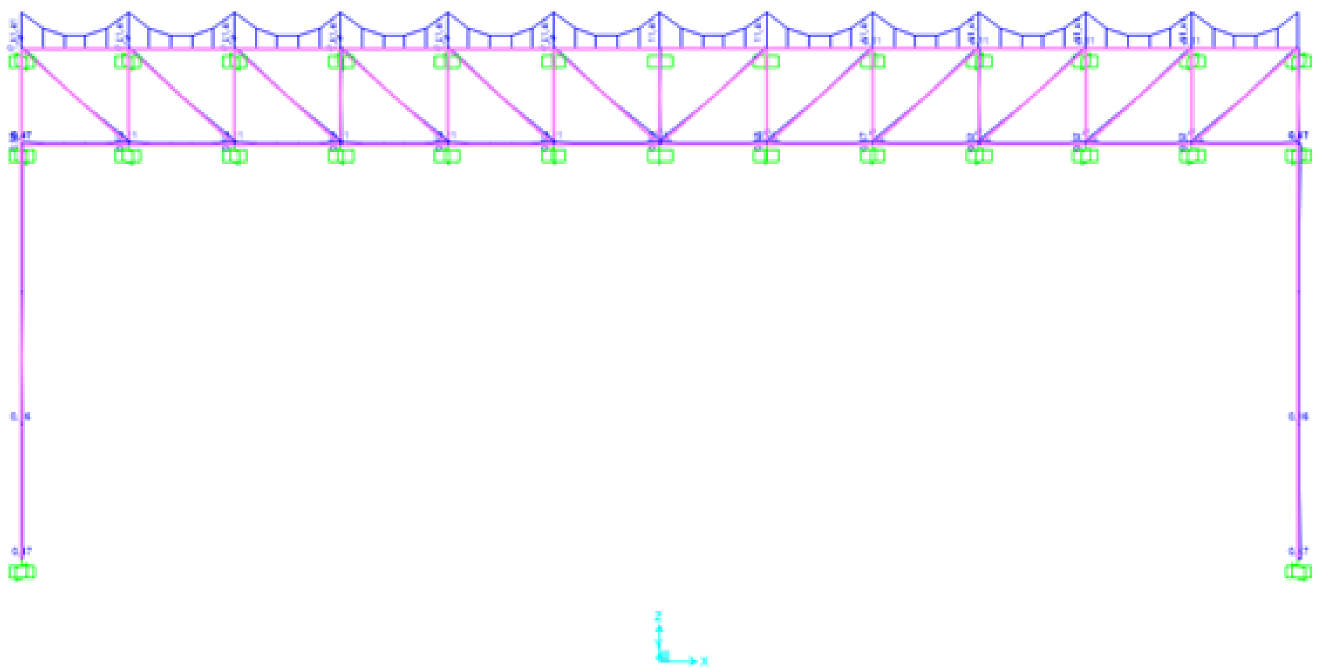
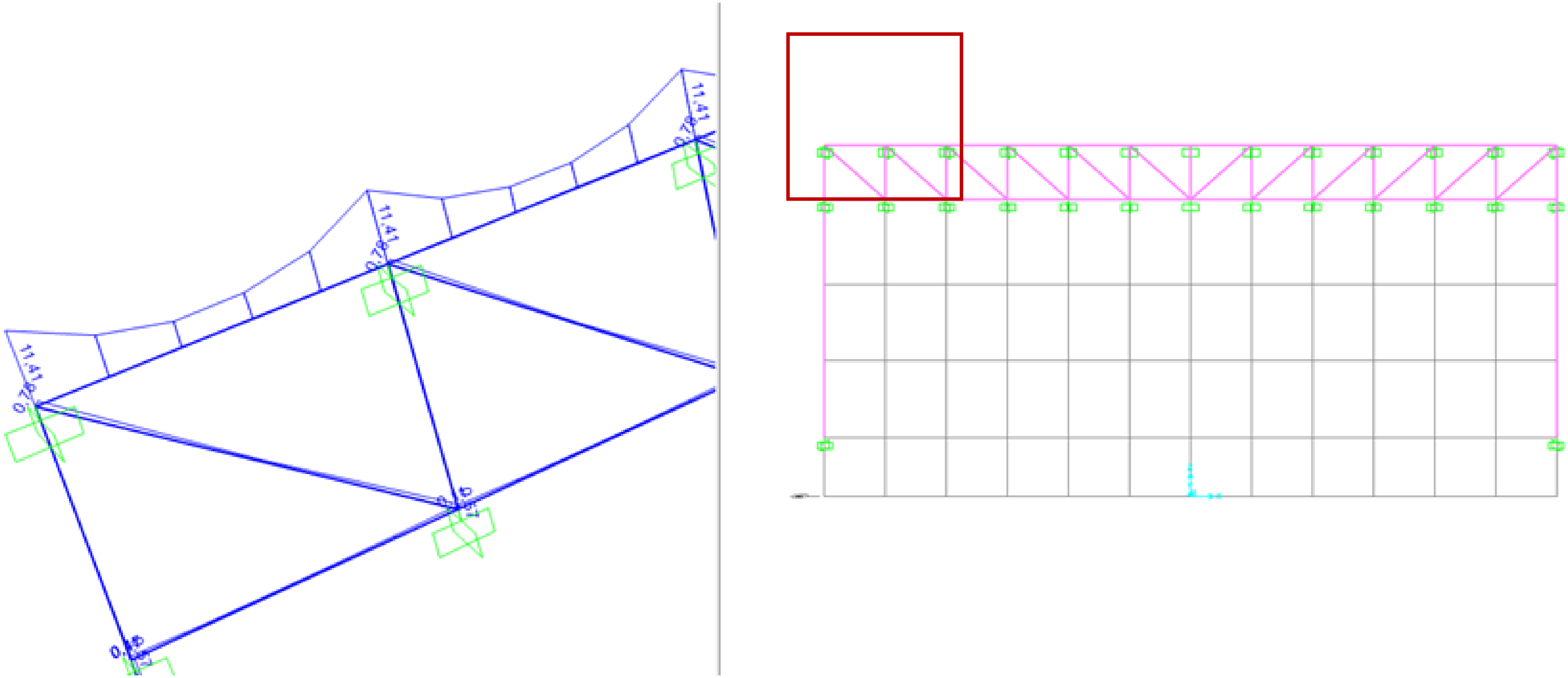


Figure 40: Extract of the most critical point of the SVM diagram for YZ02.



FRAME ZY02

The maximum value for von mises is given in the fourth combination of ULS and it's equal to $11{,}41MPa < 204MPa$.

The displacement of joints holds similitude to the one of XZ02, being:

Table 15: Joint displacements in frame YZ02 under load combination ULS 4.

TABLE: Joint Displacements								
Joint	OutputCase	CaseType	U1	U2	U3	R1	R2	R3
4	ULS1	Combination	0	0	-0,00459	0	0	0
4	ULS4	Combination	0	0	-0,00459	0	0	0
76	ULS1	Combination	0	0	-0,00459	0	0	0
76	ULS4	Combination	0	0	-0,00459	0	0	0
3	ULS1	Combination	0	0	-0,004386	0	0	0
3	ULS4	Combination	0	0	-0,004386	0	0	0
75	ULS1	Combination	0	0	-0,004386	0	0	0
75	ULS4	Combination	0	0	-0,004386	0	0	0
4	ULS3	Combination	0	0	-0,0034	0	0	0
4	SLS1	Combination	0	0	-0,0034	0	0	0
4	SLS3	Combination	0	0	-0,0034	0	0	0
76	ULS3	Combination	0	0	-0,0034	0	0	0
76	SLS1	Combination	0	0	-0,0034	0	0	0
76	SLS3	Combination	0	0	-0,0034	0	0	0
3	ULS3	Combination	0	0	-0,003249	0	0	0
3	SLS1	Combination	0	0	-0,003249	0	0	0
3	SLS3	Combination	0	0	-0,003249	0	0	0
75	ULS3	Combination	0	0	-0,003249	0	0	0
75	SLS1	Combination	0	0	-0,003249	0	0	0
75	SLS3	Combination	0	0	-0,003249	0	0	0
2	ULS1	Combination	0	0	0	0	0	0
2	ULS3	Combination	0	0	0	0	0	0

The maximum horizontal displacement being 0,00459, virtually zero.

Figure 41: Undeformed shape of YZ09.

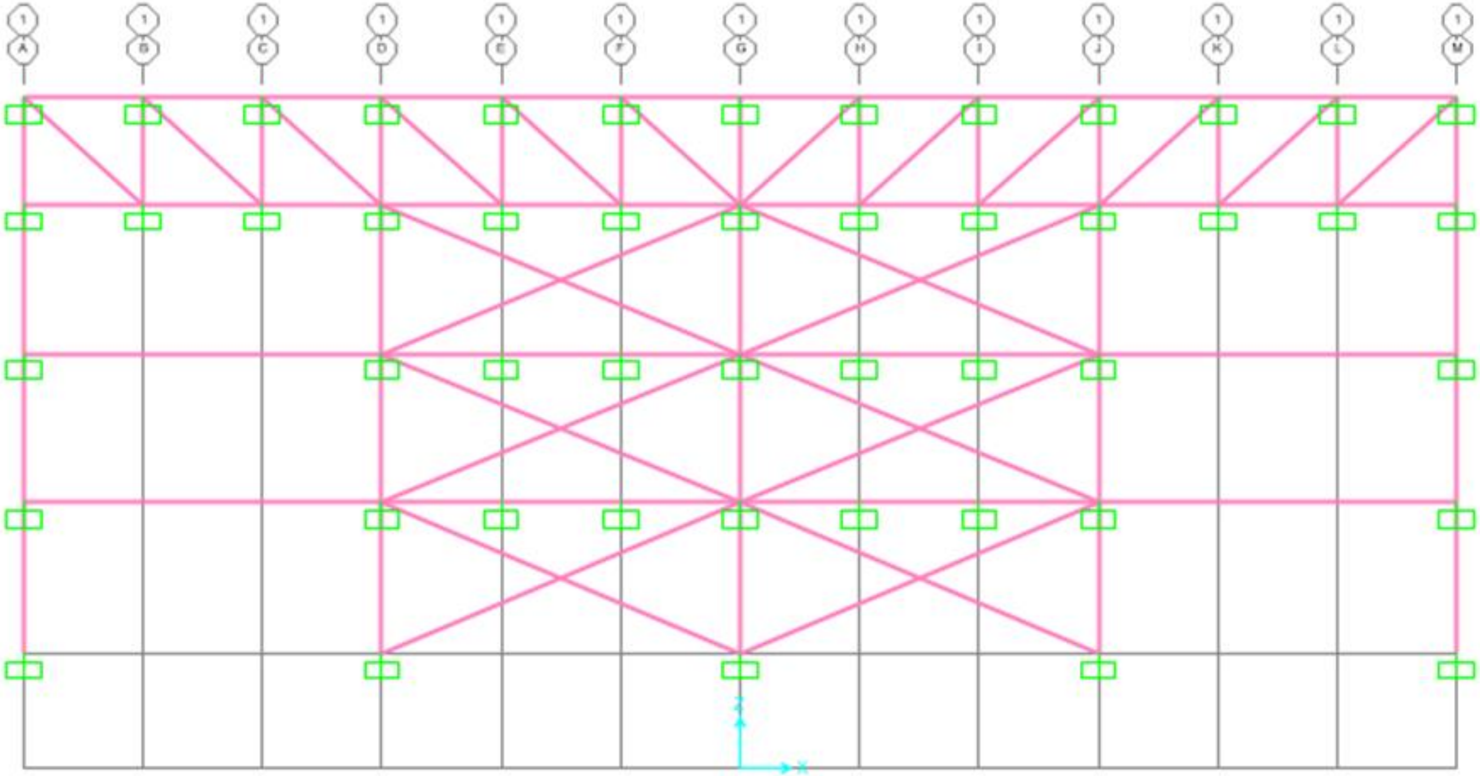


Figure 42: Deformed shape of YZ09 under load combination ULS 4.

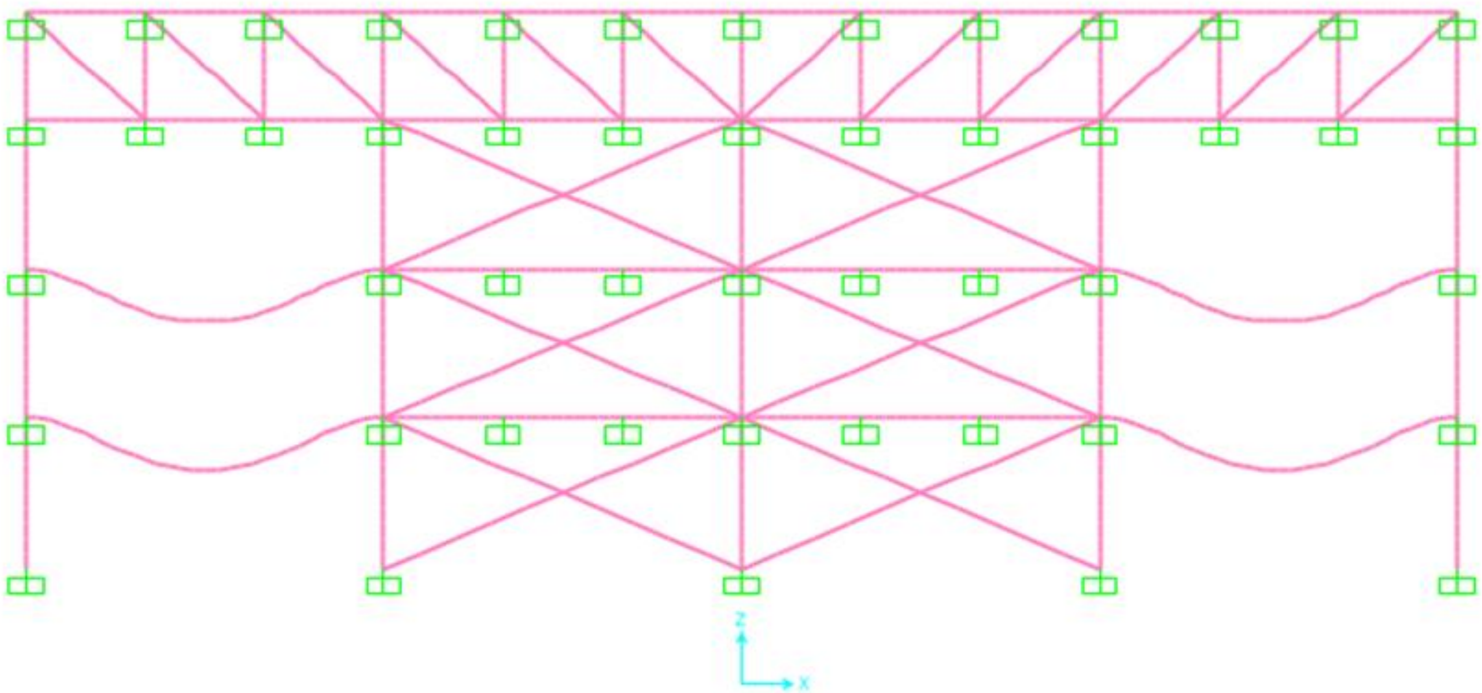


Figure 43: Diagram of SVM maximum stresses for combination ULS 4 for YZ09.

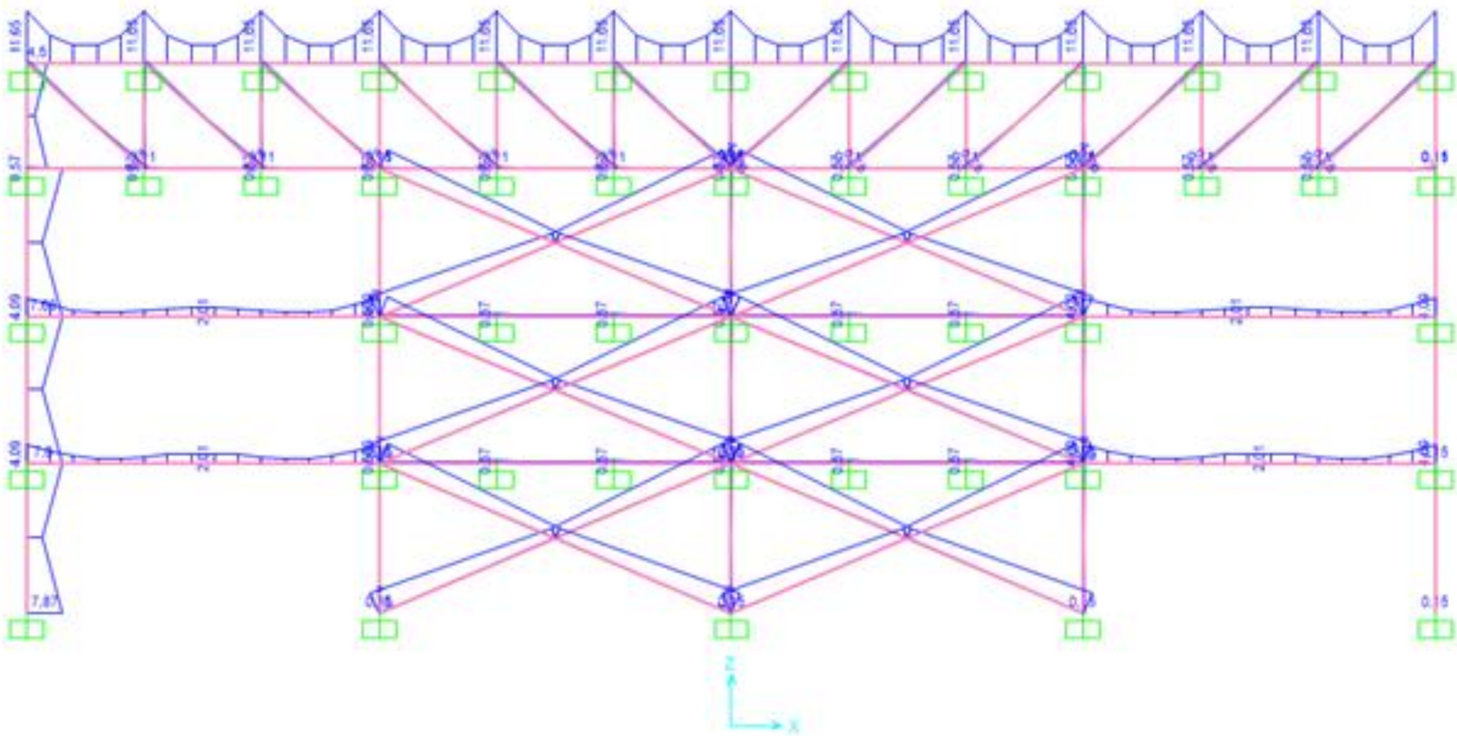
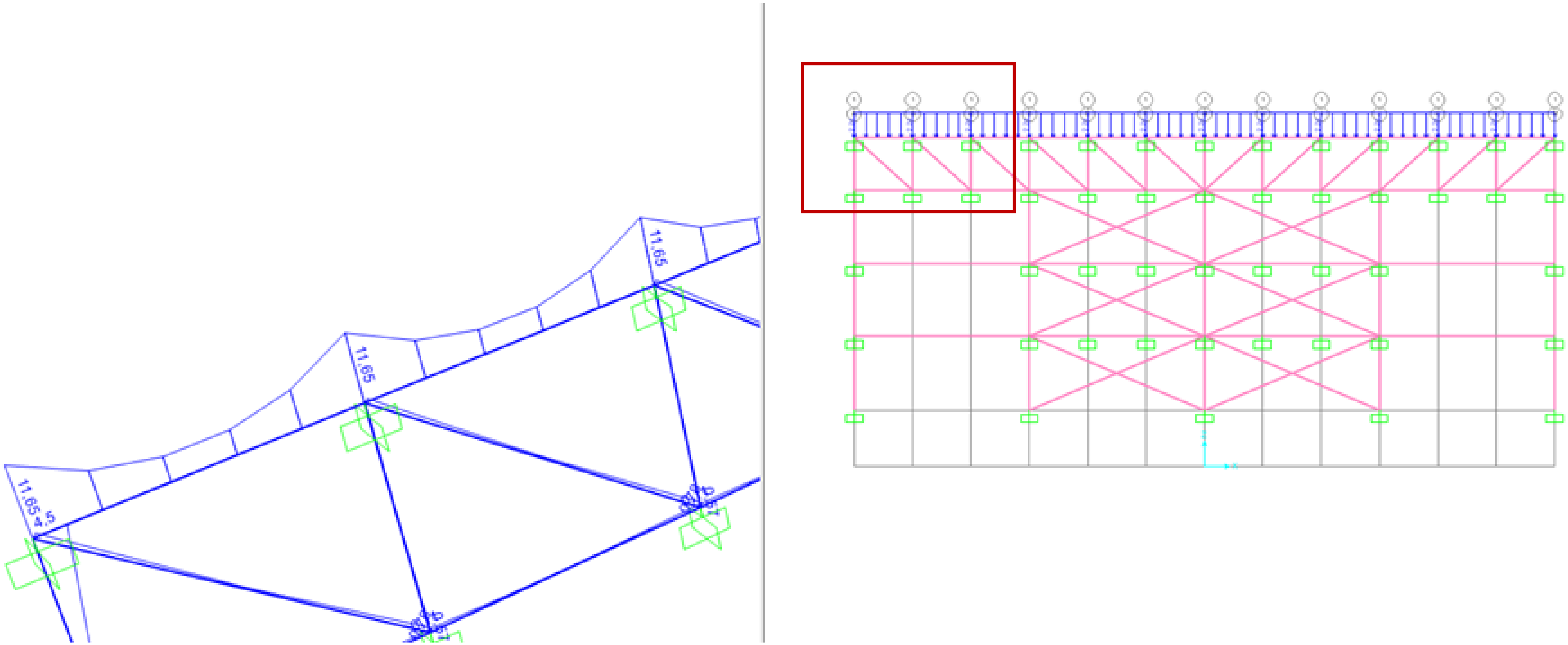


Figure 44: Extract of the most critical point of the SVM diagram for YZ09.



FRAME YZ09

This last frame has a maximum value of Von Mises of $11{,}65MPa < 204MPa$, given by combination number four of the ultimate limit states.

The displacement of joints is as follows:

Table 16: Joint displacements in frame YZ09 under load combination ULS 4.

TABLE: Joint Displacements								
Joint	OutputCase	CaseType	U1	U2	U3	R1	R2	R3
9	ULS1	Combination	0	0	-0,164742	0	0,000065	0
9	ULS4	Combination	0	0	-0,164742	0	0,000065	0
10	ULS1	Combination	0	0	-0,164742	0	0,000065	0
10	ULS4	Combination	0	0	-0,164742	0	0,000065	0
15	ULS1	Combination	0	0	-0,164742	0	-0,000065	0
15	ULS4	Combination	0	0	-0,164742	0	-0,000065	0
16	ULS1	Combination	0	0	-0,164742	0	-0,000065	0
16	ULS4	Combination	0	0	-0,164742	0	-0,000065	0
63	ULS1	Combination	0	0	-0,164742	0	0,000065	0
63	ULS4	Combination	0	0	-0,164742	0	0,000065	0
64	ULS1	Combination	0	0	-0,164742	0	0,000065	0
64	ULS4	Combination	0	0	-0,164742	0	0,000065	0
69	ULS1	Combination	0	0	-0,164742	0	-0,000065	0
69	ULS4	Combination	0	0	-0,164742	0	-0,000065	0
70	ULS1	Combination	0	0	-0,164742	0	-0,000065	0
70	ULS4	Combination	0	0	-0,164742	0	-0,000065	0
9	ULS3	Combination	0	0	-0,122031	0	0,000048	0
9	SLS1	Combination	0	0	-0,122031	0	0,000048	0
9	SLS3	Combination	0	0	-0,122031	0	0,000048	0
10	ULS3	Combination	0	0	-0,122031	0	0,000048	0
10	SLS1	Combination	0	0	-0,122031	0	0,000048	0
10	SLS3	Combination	0	0	-0,122031	0	0,000048	0
15	ULS3	Combination	0	0	-0,122031	0	-0,000048	0
15	SLS1	Combination	0	0	-0,122031	0	-0,000048	0
15	SLS3	Combination	0	0	-0,122031	0	-0,000048	0
16	ULS3	Combination	0	0	-0,122031	0	-0,000048	0
16	SLS1	Combination	0	0	-0,122031	0	-0,000048	0
16	SLS3	Combination	0	0	-0,122031	0	-0,000048	0
63	ULS3	Combination	0	0	-0,122031	0	0,000048	0
63	SLS1	Combination	0	0	-0,122031	0	0,000048	0
63	SLS3	Combination	0	0	-0,122031	0	0,000048	0

For this frame, the values of H and L are $H = 10{,}70m$ and $L = 27{,}60m$. This means that the maximum values for horizontal and vertical deflection are $21{,}40mm$ and $92mm$.

Given that the maximum values of displacement are $0{,}1647mm$ for both vertical and horizontal, all criteria apply to this frame.

5. Analysis of results and dimensioning.

5.5. Analysis of results.

In these results, we can see how the frames that are composed by more cables, beams and pillars suffered little to no stresses, whereas the frames with the least elements were the most affected, specially XZ02.

In the event of changing the material to a steel with a lower value of fyk, the study could be focused only on that one frame since it has been proved it is the one that experiences the most distress under this loading combinations.

The middle frames, with the least pillars, suffering more stresses was expected, although the structure altogether behaved successfully. This could be due to the use of doble U profiles, much more stable, or because of all joints having such a restricted freedom of movement makes the structure more cohesive, therefore moving together and transmitting stresses more efficiently. The better transmission of stress makes the whole structure suffer less deformation.

The combination that caused the biggest stresses according to Vin Mises was ULS3, composed of the permanent loads with a coefficient of 1,35 and the wind with a coefficient of 1,5. The structure is very strong to vertically applied loads, however, the value of the horizontal load gives the structure the most critical condition.

This is, again, very logical when the structure is glanced at. The system that it is composed of is better at transmitting forces vertically, especially in frames like XZ02 where there are no additional beams. This sort of load transmission pattern is beneficial considering we are working with concrete attached to the bottom of the steel frames, since concrete works great to compression, and it is better if the loads that get transmitted there are compressing the concrete.

5.6. Dimensioning.

The profile used for the pillars is double UPN, with a flange thickness of 9,654mm and a web thickness of 6,35mm. For the remaining elements, it is a simpler IPE profile. The steel used is S235.

Ways of making this more cost efficient, since the values of deformation could be closed to the maximum tolerated without compromising safety or integrity of the structure, would be using a steel with a lower fyk or reducing the profiles to smaller sizes.

A second analysis was conducted, only analysing XZ02, since It suffers the most stresses. For this second one, all elements were given the same profile, instead of separating between UPN and IPE.

All elements having the same profile makes not only the calculations easier, but also the construction of the structure. The profiles selected for this were HEB profiles.

In the next pages all the figures and tables referenced are attached, being:

Figure 45: Diagram of maximum SVM stresses under load combination ULS 3, for profile HEB100 in frame XZ02

Figure 46: Diagram of maximum SVM stresses under load combination SLS 3, for profile HEB100 in frame XZ02.

Figure 47: Diagram of maximum SVM stresses under load combination ULS 3, for profile HEB120 in frame XZ02.

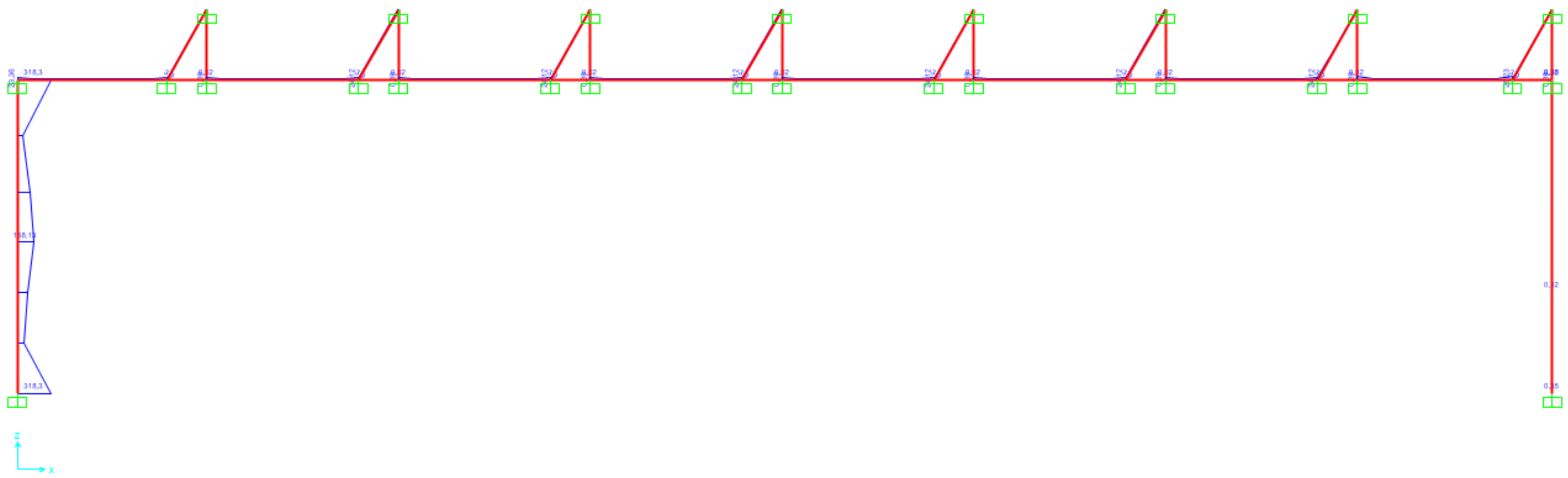
Figure 48: Diagram of maximum SVM stresses under load combination ULS 3, for profile HEB140 in frame XZ02.

Figure 49: Extract of the most critical point of the SVM diagram for XZ02 with profile HEB140.

Table 17: Joint displacement in frame XZ02 for profile HEB120.

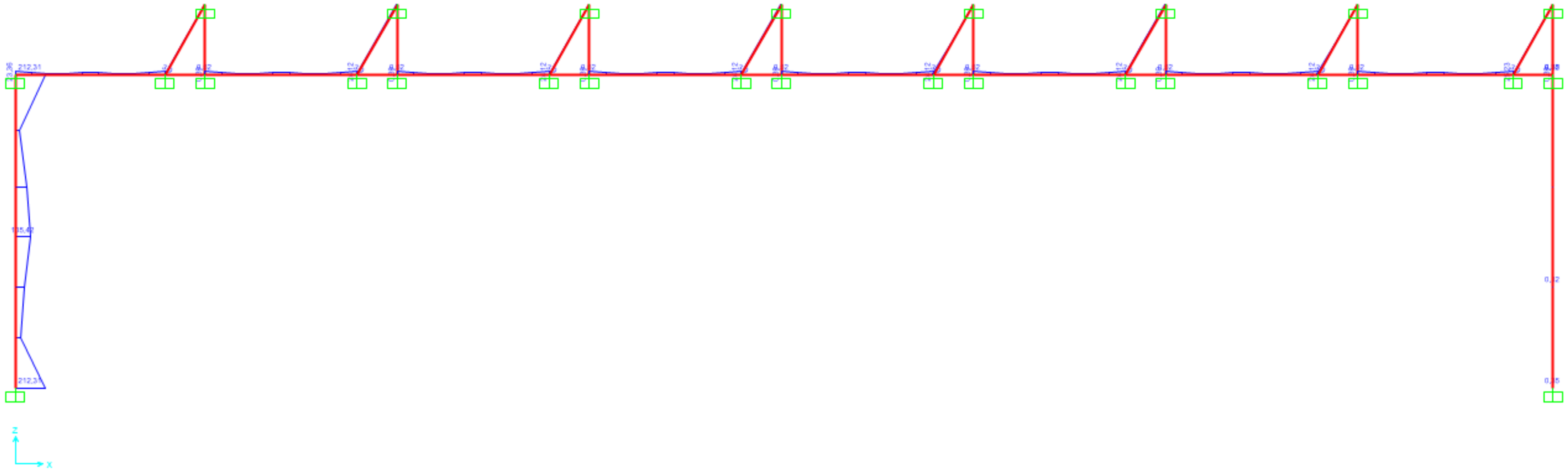
Table 18: Joint displacement in frame XZ02 for profile HEB140.

Figure 45: Diagram of maximum SVM stresses under load combination ULS 3, for profile HEB100 in frame XZ02.



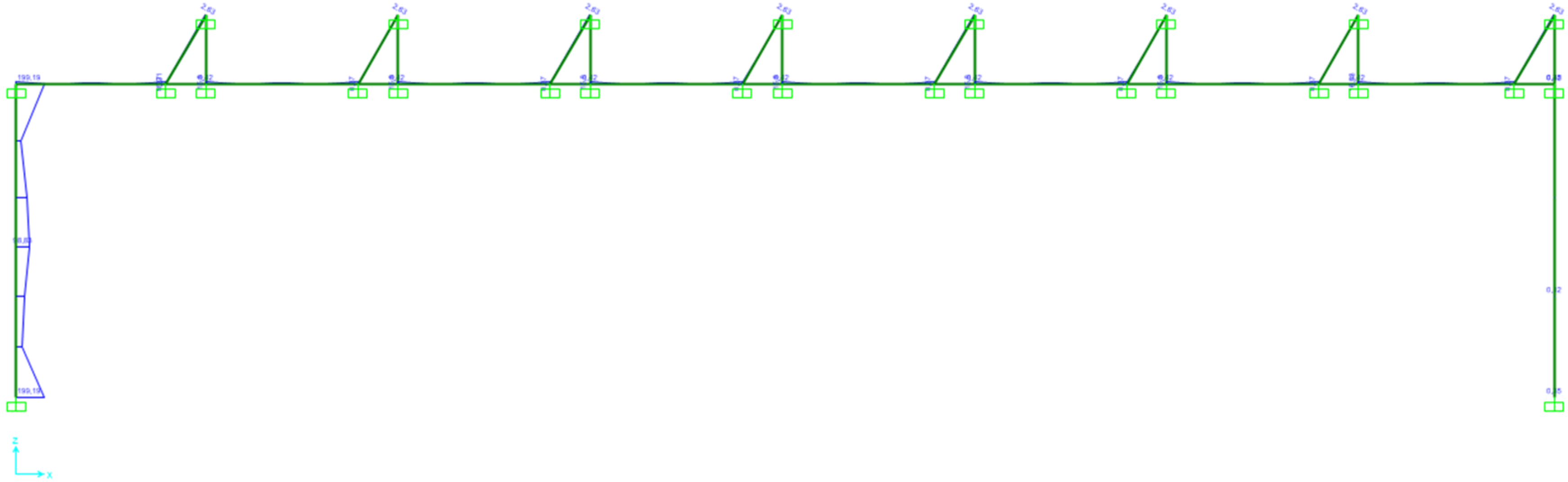
Maximum value: 318,3 MPa.

Figure 46: Diagram of maximum SVM stresses under load combination SLS 3, for profile HEB100 in frame XZ02.



Maximum value: 212,31MPa.

Figure 47: Diagram of maximum SVM stresses under load combination ULS 3, for profile HEB120 in frame XZ02.

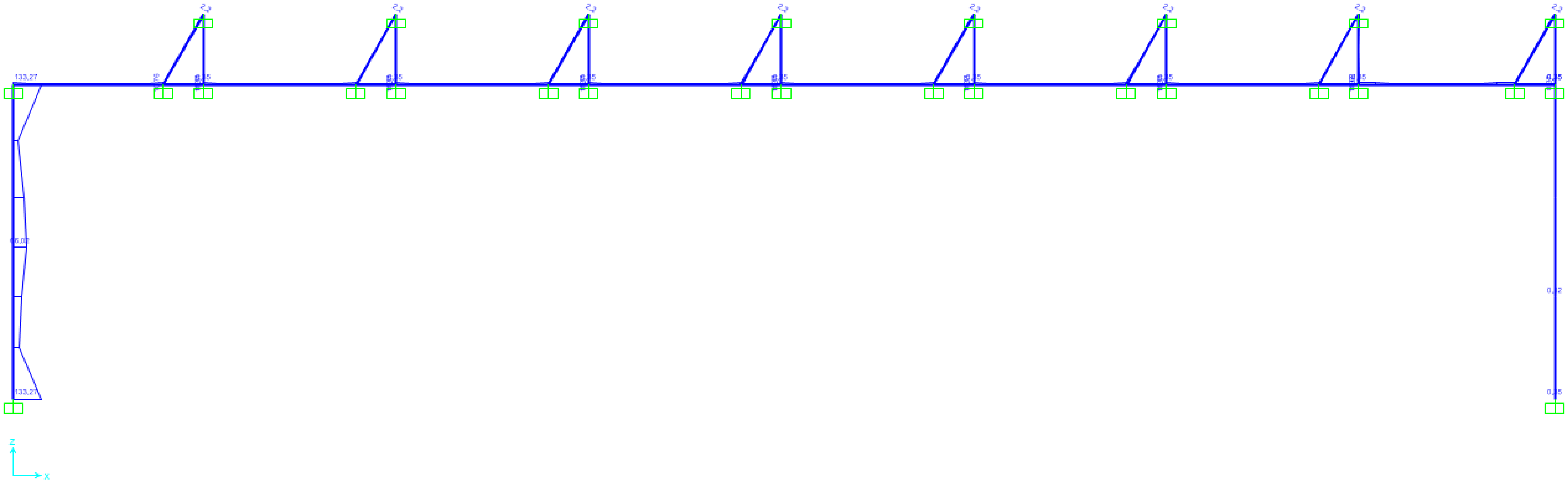


Maximum value: 199,19 MPa.

Table 17: Joint displacement in frame XZ02 for profile HEB120.

TABLE: Joint Displacements								
Joint	OutputCase	CaseType	U1	U2	U3	R1	R2	R3
Text	Text	Text	mm	mm	mm	Radians	Radians	Radians
2	DEAD	LinStatic	34,778986	0	0	0	0	0
2	ULS1	Combination	31,367774	0	0	0	0	0
2	ULS3	Combination	23,185991	0	0	0	0	0
2	ULS4	Combination	20,91185	0	0	0	0	0
2	SLS1	Combination	17,389493	0	0	0	0	0
2	SLS3	Combination	15,683887	0	0	0	0	0
3	DEAD	LinStatic	0	0	-0,003274	0	0	0
3	ULS1	Combination	0	0	-0,004419	0	0	0
3	ULS3	Combination	0	0	-0,003274	0	0,011005	0
3	ULS4	Combination	0	0	-0,004419	0	0,005503	0
3	SLS1	Combination	0	0	-0,003274	0	0	0
3	SLS3	Combination	0	0	-0,003274	0	0,007337	0
4	DEAD	LinStatic	0	0	-0,003449	0	0	0
4	ULS1	Combination	0	0	-0,004656	0	0	0
4	ULS3	Combination	0	0	-0,003449	0	-0,009169	0
4	ULS4	Combination	0	0	-0,004656	0	-0,004585	0
4	SLS1	Combination	0	0	-0,003449	0	0	0
4	SLS3	Combination	0	0	-0,003449	0	-0,006113	0
5	DEAD	LinStatic	0	0	0	0	0	0
5	ULS1	Combination	0	0	0	0	0	0

Figure 48: Diagram of maximum SVM stresses under load combination ULS 3, for profile HEB140 in frame XZ02.



Maximum value: 133,27 MPa.

Figure 49: Extract of the most critical point of the SVM diagram for XZ02 with profile HEB140.

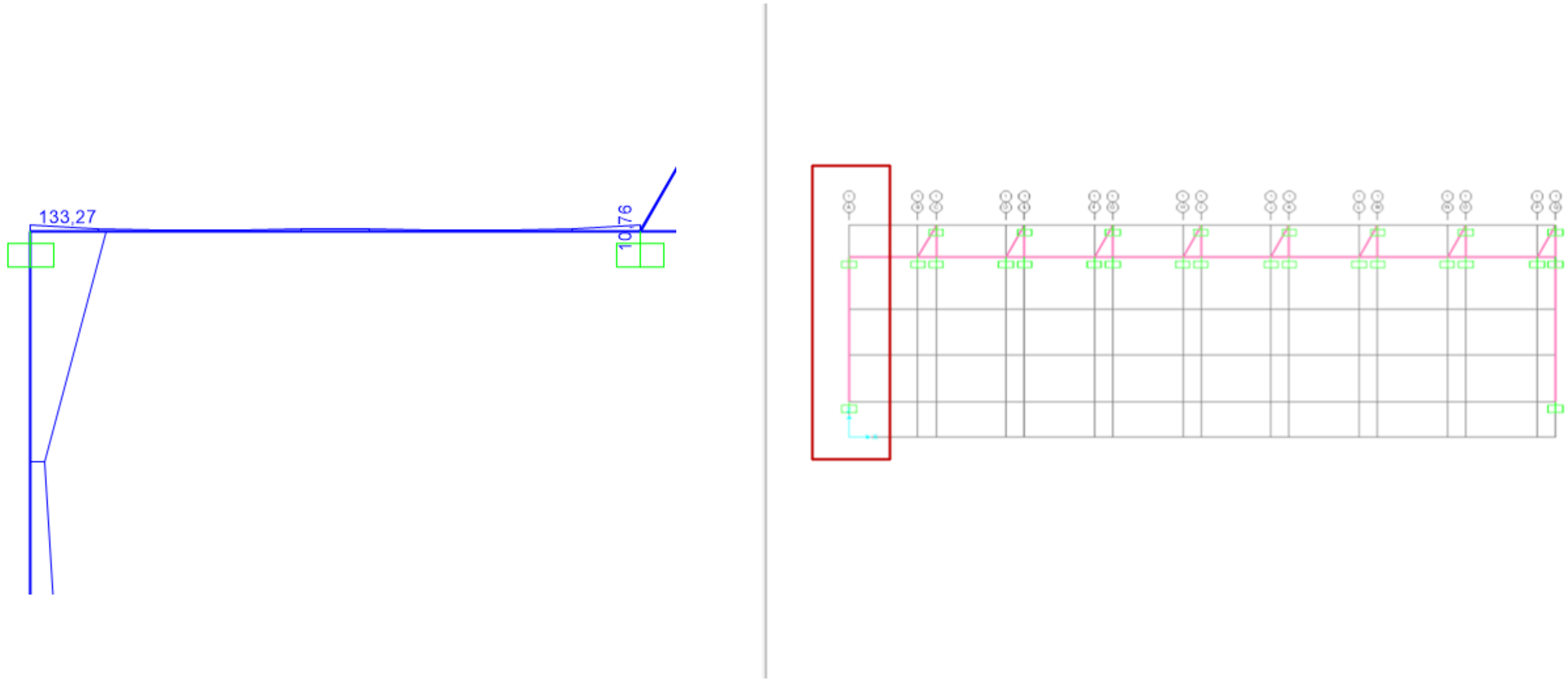


Table 18: Joint displacement in frame XZ02 for profile HEB140.

TABLE: Joint Displacements								
Joint	**OutputCase**	**CaseType**	**U1**	**U2**	**U3**	**R1**	**R2**	**R3**
Text	Text	Text	mm	mm	mm	Radians	Radians	Radians
2	DEAD	LinStatic	20,053313	0	0	0	0	0
2	ULS1	Combination	18,093062	0	0	0	0	0
2	ULS3	Combination	13,368875	0	0	0	0	0
2	ULS4	Combination	12,062041	0	0	0	0	0
2	SLS1	Combination	10,026656	0	0	0	0	0
2	SLS3	Combination	9,046531	0	0	0	0	0
3	DEAD	LinStatic	0	0	-0,003274	0	0	0
3	ULS1	Combination	0	0	-0,004419	0	0	0
3	ULS3	Combination	0	0	-0,003274	0	0,006301	0
3	ULS4	Combination	0	0	-0,004419	0	0,003151	0
3	SLS1	Combination	0	0	-0,003274	0	0	0
3	SLS3	Combination	0	0	-0,003274	0	0,004201	0
4	DEAD	LinStatic	0	0	-0,003449	0	0	0
4	ULS1	Combination	0	0	-0,004656	0	0	0
4	ULS3	Combination	0	0	-0,003449	0	-0,00525	0
4	ULS4	Combination	0	0	-0,004656	0	-0,002625	0
4	SLS1	Combination	0	0	-0,003449	0	0	0
4	SLS3	Combination	0	0	-0,003449	0	-0,0035	0
5	DEAD	LinStatic	0	0	0	0	0	0
5	ULS1	Combination	0	0	0	0	0	0
5	ULS3	Combination	0	0	0	0	0	0
5	ULS4	Combination	0	0	0	0	0	0
5	SLS1	Combination	0	0	0	0	0	0
5	SLS3	Combination	0	0	0	0	0	0

The first profile assigned was HEB100, which gives a maximum value of Von Mises of 318,3MPa, which is $318{,}3 > 204MPa$, making the profile non suitable.

The second profile assigned was HEB120, giving a maximum value of Von Mises of 199,19MPa, which is less than the limit, but when the joint displacement was studied, the maximum displacement is 34,778mm, way above the limit of 21,4mm.

The third and last profile assigned was HEB140 with a maximum value of VM of 133,27MPa, which is tolerable, and a maximum joint displacement of 20,053mm, less than 21,4mm.

After this second analysis, it can be concluded that the most cost-effective and efficient profile to be used would be an HEB140 on the whole structure.

6. Conclusions.

This paper is, as established in the beginning, based on the architect's report. After all the calculations and analysis, it can be concluded that, although this building is extremely resistant and does fulfil the need it was built to cover, the selection of materials and construction methods, from a sustainable point of view, could have been better.

However, given the importance placed on the budget (after all it is a public building), the selection is as well thought out as it can be without blowing the budget. Which could conclude that establishing more environmentally sustainable methods in this industry is still a complicated issue and something reserved for higher budget builds.

It is also relevant how the design of the structure is very efficient, hardly giving in to any compressive force, due to the placement and type of the load bearing elements.

The key concepts to highlight, basic for any dimensioning, are:

Firstly, the study and analysis of loads, based on Eurocodes and national annex, with the application of the proper load combinations, respecting the requirements and criteria of Ultimate limit states and Serviceability limit states.

The design of the structure should not only be aesthetically pleasing, but its main goal is to be functional and cost effective. From a perspective of structural mechanics, the objective is to design a sturdy, economic structure that satisfies all the needs it was build for, for at least 50 years.

Also, the selection of materials is crucial to ensure safety and good performance. For this, having a clear understanding of the physical and mechanical behaviour of materials as well as how their properties can change with the manufacturing and construction process (with processes like welding) is useful and necessary. This selection depends on design on the structure, needs, budget, location, etc…

Lastly, the incorporation of sustainable practices and materials is something that cannot be ignored. The work to be done here is not only to make sure the building does not collapse, break, etc… but also ensure the comfort, and that starts with taking better care of nature and being more responsible about the use of the resources it gives us.

ANNEX – Further plans.

In the following pages, the plans of the structure specifying the nodes connections are displayed, being:

Figure 50: 3D view of the structure with the names of sections A to F.

Figure 51: View of section A.

Figure 52: View of section B.

Figure 53: View of section C.

Figure 54: View of section D.

Figure 55: View of section E.

Figure 56: View of section F.

Figure 50: 3D view of the structure with the names of sections A to F.

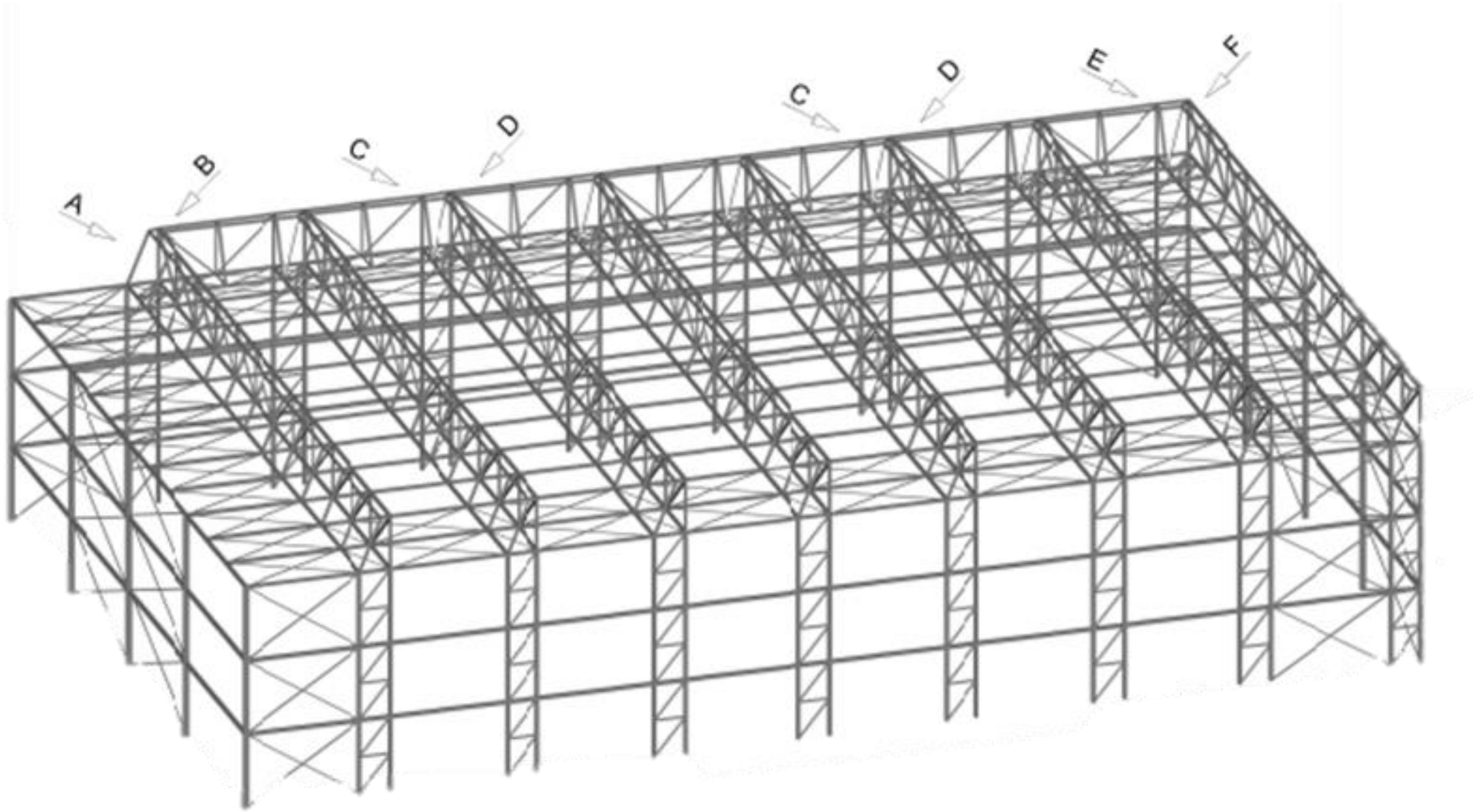


Figure 51: View of section A.

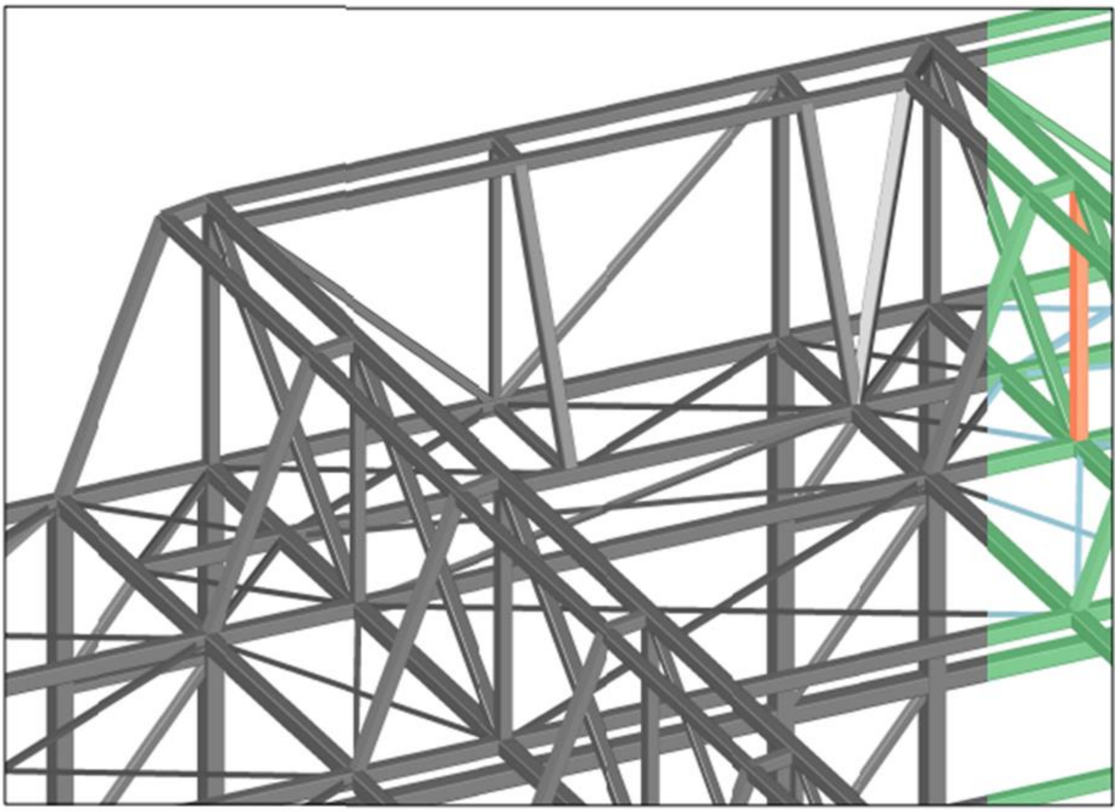


Figure 52: View of section B.

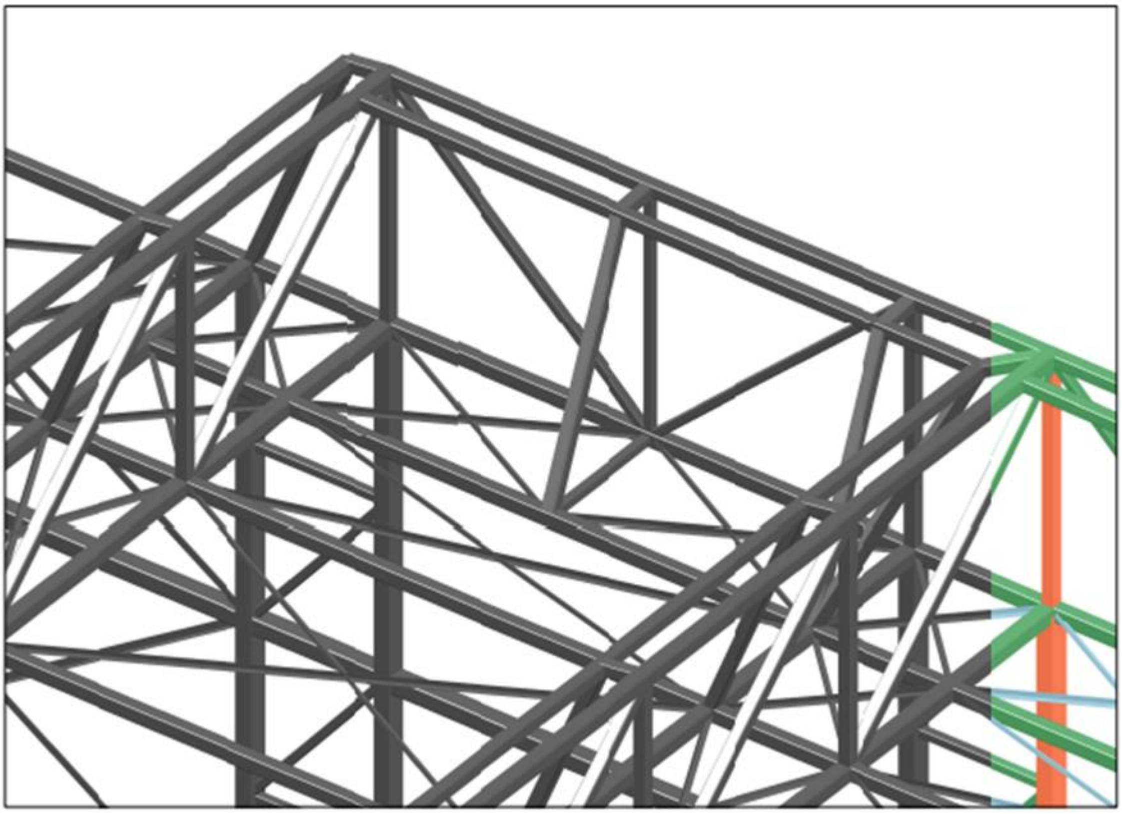


Figure 53: View of section C.

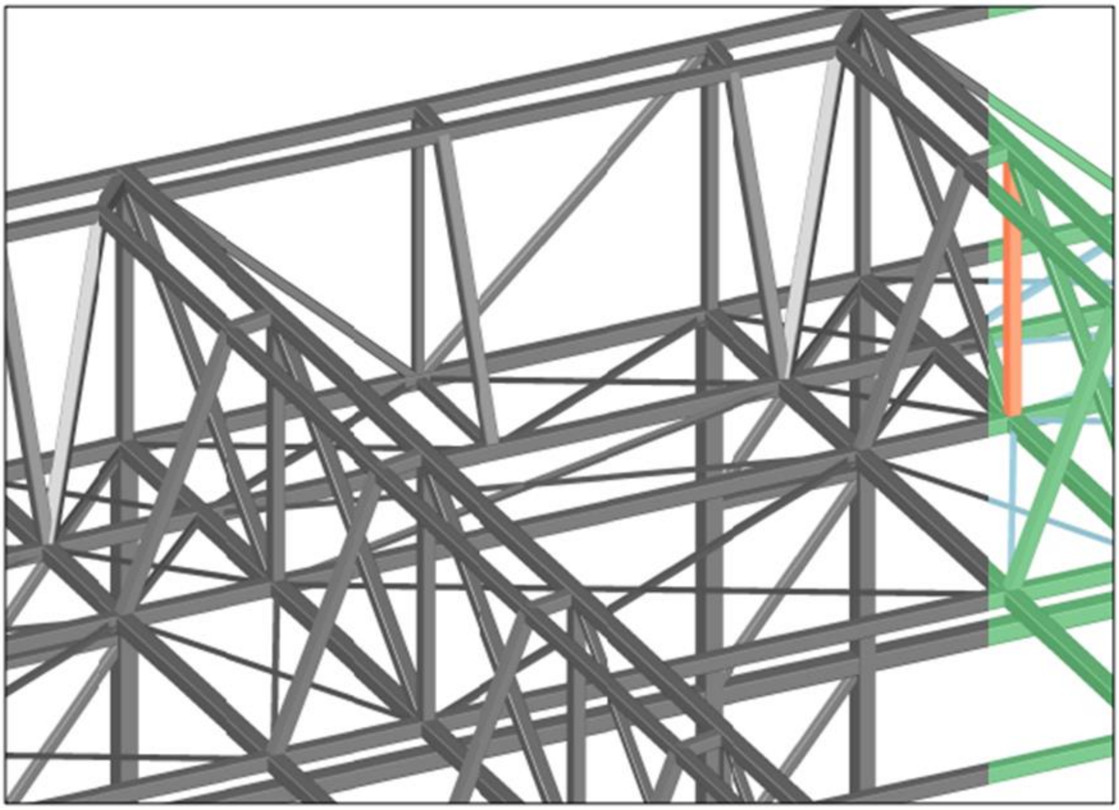


Figure 54: View of section D.

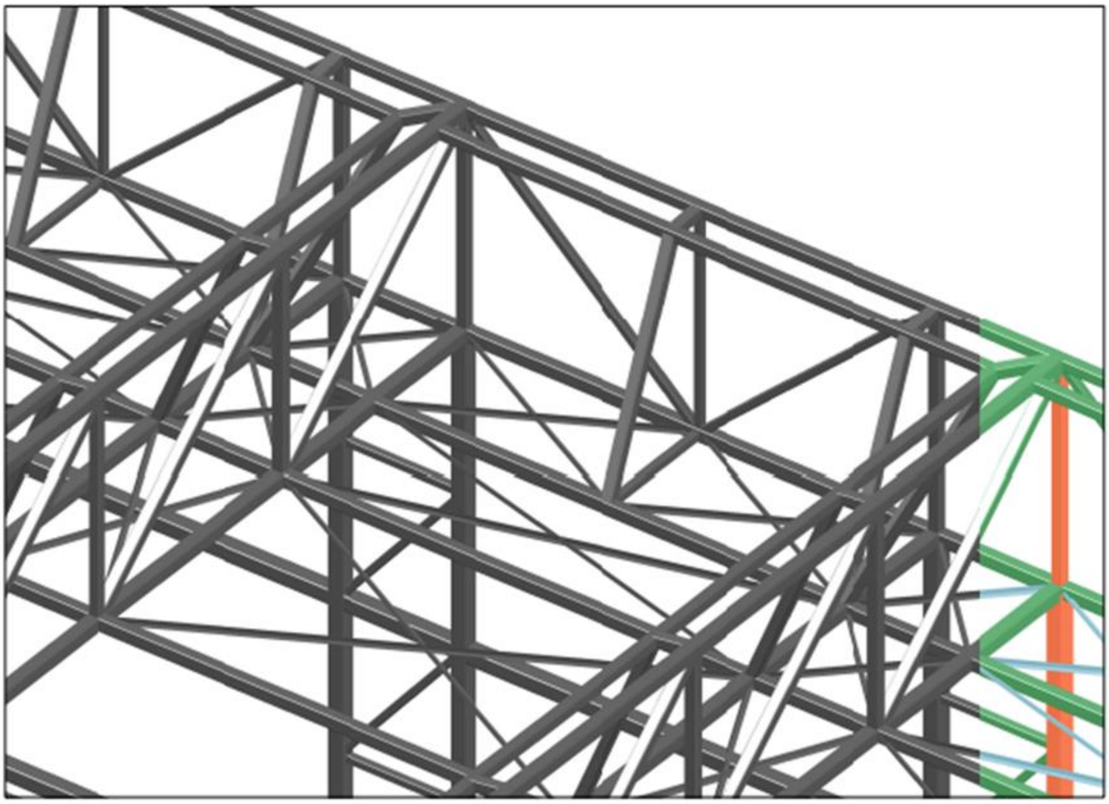


Figure 55: View of section E.

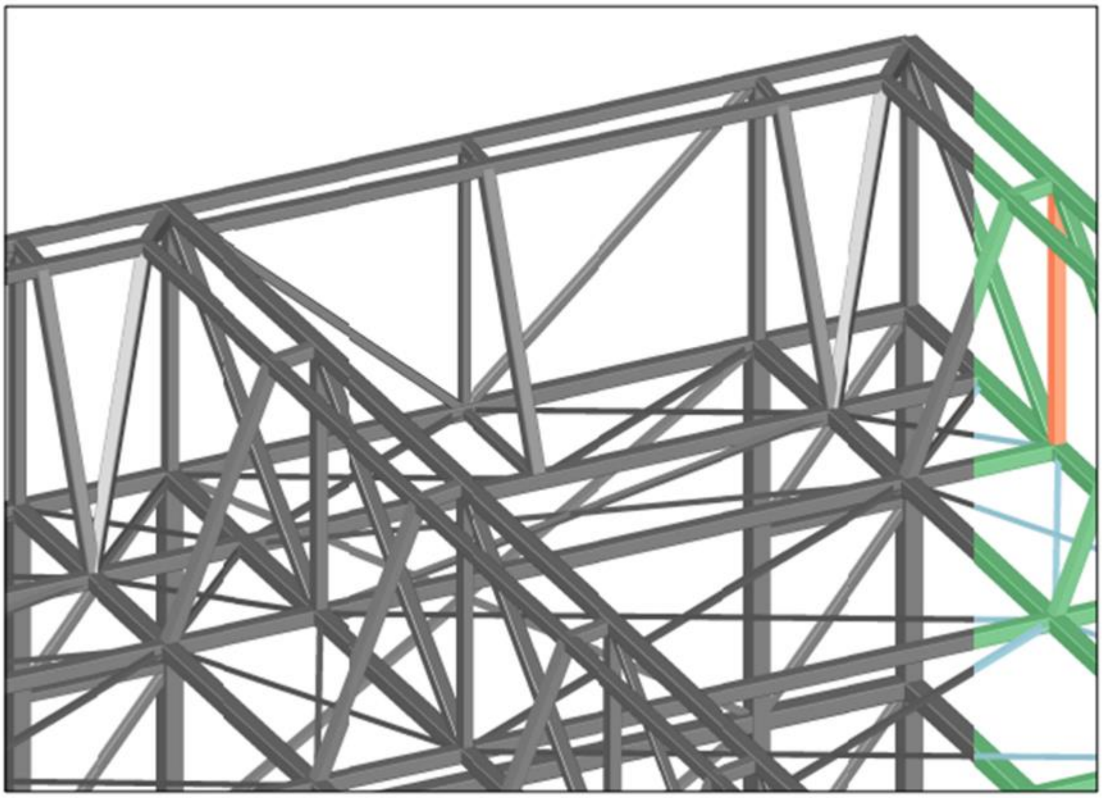
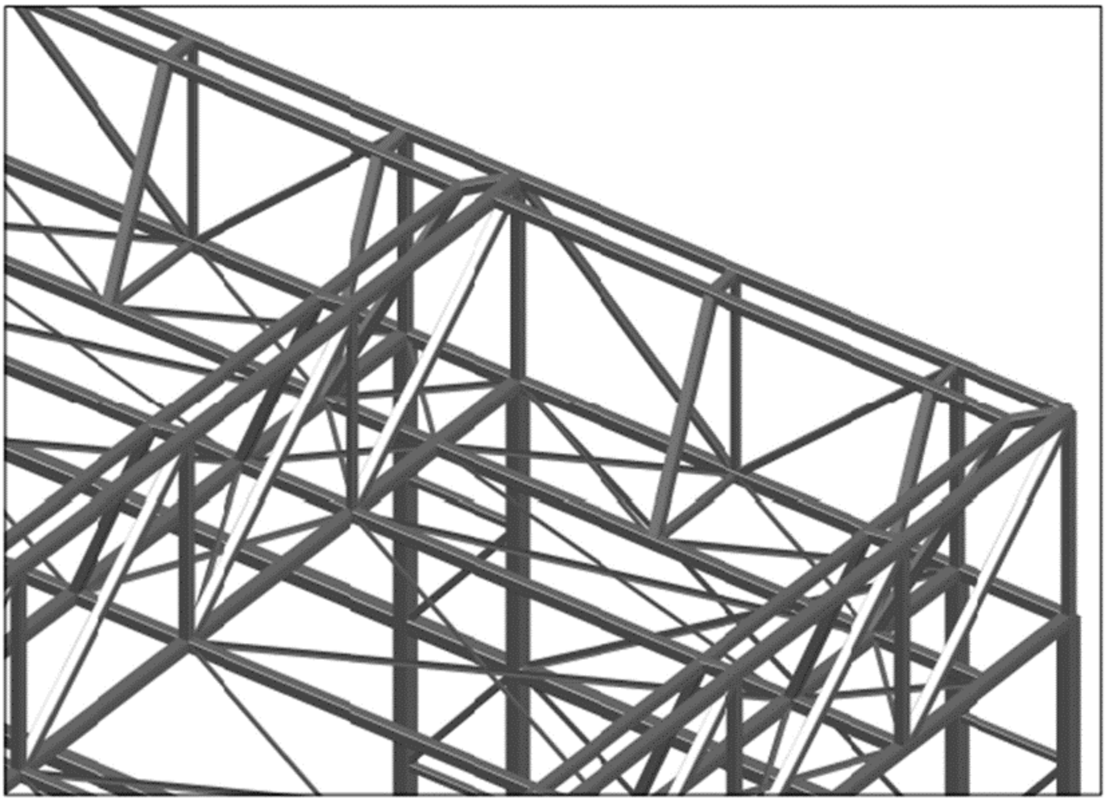


Figure 56: View of section F.



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