Study of design and calculation of the structure of a house in the municipality of Sant Cugat del Valles.
Study of design and calculation of the structure of a house in the municipality of Sant Cugat del Vallès

Acknowledgements:

Family

Close friends

Ricard Neira, Xavier Alonso and Amador Sillero – Transformados Metálicos SA
# Study of design and calculation of the structure of a house in the municipality of Sant Cugat del Vallès

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1. Introduction

1.1 Objectives

The objective of this project is to calculate and design the metallic structure of a house in Sant Cugat del Vallès.

1.2 Scope of the project

The scope of the project will be delimited by the ending of the design of the metallic structure, as well as its calculation and its assembly on the specific place. No previous land studies nor studies of the subsequent facilities will be contemplated.

1.3 Specifications

- As it is a metallic structure, the European UNE EN-1090 regulations for metallic structures must be followed.

- The structure must be able to resist the cutting, axial and bending efforts that will be generated by the charges applied to it, hence assuring the security of the people living within it before any given circumstance.

- There are three different regulations regarding the implementation of metallic structures: EN1993, CTE DB SE-A and EAE.

- The structure must be viable both in its construction and assembling.
2. Project justification

This project is motivated by my current work. As a project manager, I can dedicate myself to analyze the structures that we carry out.

Now with this project I can put more emphasis on my work and not only perform a brief analysis of the structure, but also focus on its calculation, hence being able to select profiles in detail through a calculation software and doing the design of the structure with a designer partner as well.

2.1 Transmetal

TRANSMETAL (Transformados Metálicos, SA) was born in 1995. It is focused on the manufacturing of the structures, as well as on metallic profiles formed in cold. It is mostly addressed to the building industry. Nowadays, the company consists of 3 departments: metallic structures, facade and roof.

At present, the facilities are situated in Lliçà de Vall (Barcelona), and it has CE certification available for the construction of metallic structures by Bureau Veritas (2014), which can be required since the 1st of July, 2014 for all the metallic constructions. We also have a Quality Management system, according to ISO 9001:2008 regulation, by Aliter Qualitas.
It is also important to mention that the company obtained the CONSTRUMAT award to edification for the technologic innovation, thanks to the construction of the Industrial Unit of Simon SA in Olot (Girona).

TRANSMETAL is associated to ASCEM (Association for the construction of metallic structures) and ACE (Association of constucture consultants). In addition, it is a company adherent to Colegio de Ingenieros Industriales de Cataluña. This close collaboration with these organizations, allows us to stay informed of any aspect that affects our work and it is an idea exchange forum, which makes it much more valuable.
3. State of the art

3.1 Metallic structures

The world of the metallic structure is complex and modern. Not only do we not know the involved number of difficulties, but we also do not know the number of advantages that this method entails either.

3.1.1 Initiation

If we take a look back in history, we will realize that humans have always built their homes in places that have proven to shelter them from heat, cold and floods.

The main resources for these constructions consisted on the main materials of the given time, as profiles laminated in steel had not been yet discovered nor they would come to think that anything like this would ever happen.

Masonry is part of the most primitive resources in the history of human constructions. It is known that around 7500 AD constructions involving the use of bricks already existed in Mesopotamia.

Illustration 5 - Brick manufacturing in ancient Egypt [2]

The first people to use it for construction purposes were the pre-ceramic Neolithic farmers, a time where all constructions were made of wood and stone.

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The vast majority of current buildings follow the same pattern, but thanks to technology those same buildings can be created again but being more slender and resistant.

Following this introduction, I will make a list of advantages and disadvantages regarding the construction with metallic structures in comparison to the more traditional method involving concrete.

### 3.1.2 Advantages

- Homogeneity of materials.
- Quick assembling.
- The possibility of forming previously assembled assemblings in a workshop, reducing the amount of assembling on site.
- Lower cost of foundations due to the manufacture of lighter structures.
- The possibility of manufacturing a structure that can be dismantled, reused or recycled assumes a significant saving of the investment by allowing it to be disassembled.
- Materials are of great resistance. It is not necessary to reach the large dimensions needed with concrete to obtain an adequate strength to resist the requested efforts.
- The use of ductile materials facilitates the detection of an error or an overload prior to the possible failure of the system.

### 3.1.3 Disadvantages

- Its economic cost. Not only the structure but also the subsequent operations suppose an increase in the final price.
- Ignifugation is almost mandatory as in case of fire a collapse can be caused. Being a material that absorbs temperature, its resistance will be reduced. In some cases, it can become a mandatory task.
- Corrosion. Steel does not have a great resistance to corrosion and, in humid and/or marine environments, special care must be taken.
- Buckling. All slender elements suffer from it. Although the calculation already contemplates it, a maximum of arrow per piece length must be set.
3.2 Joints

On the one hand, joints are the main feature that act as a differentiation in a metallic structure with the concrete ones, since one of its main characteristics is the absence of variants regarding joints.

On the other hand, unions are one of the most conflicting parts in the world of metallic structures, making it necessary to carefully study all of them in the projects.

There is more than one type of union, and a structural set may have more than one type.

Over the years there has been a historical evolution, with different trends when it comes to solving the joints, marked in an important way since steel was first used in 1856 in metallic structures.

Due to the risk we assume when making unions we try to generate the minimum of them, in addition to being able to represent up to 40% of the cost of a structure.

As I have said before, it is not necessary to use the same type of union in the same set, but if we want to reduce costs, we should try to typify the unions in different places. Some unions appear more important than others within the structure, depending on the level of importance of the piece to be fixed, such as in the case of a beam-pillar joint, where we must take into consideration more than one important factor in order to choose the correct type of union.
All this amount of costs that can be generated by unions should be taken into consideration in the initial proposal since a union is not accounted for by the material contributed, which comes to be minimum in consideration with the structure. However, what will be regarded with more detail is more the number of hours that the construction will involve and what ranks of operators will be needed to perform that specific union.

The realization of the union in the workshop will always be facilitated as much as possible. We will go to the work if the worker has a good access to the deposition of materials by facilitating the correct position or allowing mobility for the placement of bolts, thus reducing the cost of the total calculation of manufacture of assembling hours.

On the following part I will offer several of the options for a union that have appeared over the years.

3.2.1 Rivets

The first joining method was the rivet, which has good characteristics when it comes to solving the compression and shear stresses. At the moment it is a technique found in disuse, although we have good examples of this type of union, in characteristic buildings such as The France Station (Barcelona), The Eiffel Tower (France) or even parts of our ESEIAAT Centre.

There are different types of materials for these elements, being steel the most characteristic, but also being possible to manufacture in aluminium, copper, etc.

The current union which shares the most similarities would be the screw but without a thread, differing greatly by the method to carry out the union.
3.2.1.1 Process

The spike of the rivet is introduced into the previously made hole. Once the head of the rivet is in contact with the base of the piece to be joined with, the opposite end is heated up to high temperatures between 950°C and 1050°C.

Now in that temperature range the steel can be easily molded. The molding of the end is started also by doing the same way as the main head once the desired shape is achieved. It should be appreciated by the colour of the material that the temperature is not higher at around 700°C, so it will keep the form done.

3.2.1.2 Present

Currently, this operation has been exempt from use to fix metallic components due to the inconveniences that it presents, such as: poor use of the material requesting the components to traction, poor distribution of tensions in the area of action and especially for the poor security in the joints.

It may happen that a rivet is not completely fixed to the components and then impossible to calculate accurately in the joints.

3.2.2 Screws

After riveting, a new method with an improvement appeared, now these unions could be dismantled. We are talking about the union with screws.

With this improvement, we can proceed with greater speed to the execution of the unions.

In this type of joints, a base is necessary to prevent the penetration of the bolt in the hole. In this case, we use washers to avoid additional stresses in the most conflictive areas.

There are several types of screws and washers, all have their specific function:
Security washers: When the efforts are also dynamic.

- Calibrated screws: All those joints with dimensional tolerances, adjustments H11/h11.
- Black screws: Does not specify the surface coating that incorporates. It can be galvanized, stainless, etc. They only indicate that the level of adjustment is not necessary like with the calibrated screw.

3.2.2.1 High resistence screws

This type of screws are tightened at high pressure to exert a severe friction reaction, so they take advantage of this reaction to transmit stresses on the contact surfaces and reduce the efforts in the joint.

These unions will work with a lock nut or with a direct thread. All of them also have flat washers under the head, and also in the nut.
3.2.3 Welding

In 1910 a great disruption in the world of metal structure was created with a new way of fixing joints.

Welding is a new technique that, by means of electrical intensity and a filler material, melts both parts to fix the joint in an established position.

The filler material is supplied through the coated electrodes. The coating of these electrodes creates an inert atmosphere to avoid inclusions of other particles inside the weld.

It has a wide range of advantages, one of the most important is the best tension distribution, using the entire base to make the union.

Thanks to this type of union we can get to make lighter structures, being homogeneous and continuous in some cases.

All these advantages also have a series of disadvantages such as breakage due to fatigue and fragile breaking. These types of breaks occur due to the dynamic loads and internal stresses that we generate when creating a hardening at the time of welding.

And, in addition, one of the biggest disadvantages from the economic point of view regarding the assembling is that specialized and qualified personnel is required for this type of union. In addition to greater difficulties at the time of realization, the structure is examined with more rigorous inspections in some occasions.

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The union does not have to cause more difficulties than the ones it causes by itself, so we have to adapt to the available means. Often the place where the welding has to be made is so difficult that opting for a bolted connection will limit the number of errors caused, therefore ensuring the quality of the union and simplifying the performance of verification tests, such as the use of torque wrenches.

3.3 Steel profiles

The metallic structures are formed by steel profiles. Standard profiles are generally used to build, although profiles with specific shapes and / or special measures can also be requested from the steelworks.

In this project, as in most of them, we will use standard profiles. I will make a list of the most known structures, and I will name the other possible forms that they may have.

All of them can be used as main profiles of a single piece and there is also the possibility of joining more than one profile to perform other geometries with different properties such as, for example, to achieve a higher moment of inertia in comparison to a single profile.

This particular case is found in the hall of our university ESEIAAT, where we find a series of profiles in L joined by rivets, to form a pillar with a greater moment of inertia compared to an IPE of the same caliber. Thus we achieve a more slender profile, but more resistant to the efforts that we suppose as the main ones.

The following image illustrates the most used profiles.

All these types of profiles are processed through a hot rolling process, for which I will explain in the following section.
3.3.1 Hot rolling

Hot rolling has a great number of advantages, among which we can highlight one that differentiates it from all other models of forming metal profiles: the internal low residual tension of the section. All of this is thanks to the absence of welds or joints, generating a uniform solid with a better resistance to loads compared to other methods that have secondary processes.

Illustration 11 - Hot Rolling [8]

In addition, as a linear process, it can generate solids of large dimensions without suffering variations in its interior architecture. That causes a greater cost reduction in these profiles, which can currently be purchased in various sizes, and lengths depending on the size.

3.4 Protection against the fire

As previously mentioned, metallic structures at high temperatures (> 500°C) in case of fire may have the structural profile vary. Consequently their resistance will be reduced and the modulus of elasticity decreased; properties that would now cause the structure to collapse according to the initial calculation.

Currently, the fire safety regulations require the fireproofing of industrial buildings and ships in order to being able to start working, but it does not require the fireproofing of houses built with metal structures.
Among the most used fireproofing techniques we find intumescent paint as well as perlite and vermiculite mortar, which allow us to maintain the properties until the fire is extinguished and the building evacuated.

Flame retardant concrete has good properties envolving fire resistance, so it is still one of the most used to protect a structure against fire.

The application of this material is made through projection thanks to a compressor and a mixer.

In order to allow the adhesion of the material to the steel profile, the concrete must be damp and the structure must be clean and free of grease.

Our project will be fireproofed with fireproof concrete.

3.5 Quality controls

Welding processes are present in our environment in any type of construction, whether machines, ships, trains, airplanes, bridges, cars, or in any type of union.

Welded parts and elements require verification of how the manufacturing process was developed. These verifications are also preventive maintenance at an industrial level, as the presence of cracks, nodes, slag and imperfections can cause the pieces to break.
This is why quality controls are carried out. Especially in this section we will talk about welding analyses, which are mainly two types: destructive and non-destructive analysis.

A non-destructive testing (NDT) is a type of test applied to a material that does not permanently alter its mechanical, physical, dimensional or chemical properties.

These are the different types of controls.

- Visual test
- Tests with penetrating liquids
- Ultrasound test
- Inspection with magnetic particles
- Inspection by induced current
- Inspection by radiography
- Inspection with gamma rays
- Loss tests

4. Regulations

Nowadays there are 3 main regulations within the main Spanish legislation regarding the construction of steel structure projects.

The three documents coexist with a higher or lower rate of resemblance between them.
4.1 Eurocode [Eurocódigo (EN 1993)]

This Eurocode is formed by many parts and subparts. I will announce the most important ones in accordance to the rules that may be applied in each case:


This eurocode is commonly accepted, but its realisation is not compulsory within Spanish territory.

It is the most complete regulation of the main three, including the calculation of mixed steel & concrete structures, which CTW and EAE do not contemplate as they are reduced versions of the structural Eurocode.

4.2 Structural Steel Instruction [Instrucción de Acero Estructural (EAE)]

The Spanish Structural Steel Instruction is applicable to all structures and elements of structural steel of building or of civil engineering.

The structures formed by high resistance steels or mixed components, being concrete or other materials, will have to apply the Eurocode.

In relation to the measures for the prevention of occupational risks that may have been taken into account during the completion of the project and the implementation of the structures and the structural steel elements, the regulations that appear at the 24th of October Royal Decree 1627/1997, where minimum security & health at building works provisions are established.

4.3 Technical Building Code [Código Técnico de la Edificación (CTE DB SE-A)]

The Basic Structural Steel Security Document of the Edification Technical Code is addressed to verify the structure in terms of security within the metallic elements formed with structural steels.

It is only devoted to buildings, it must not be applied other type of works out of the area of homes or industries, such as bridges, silos, chimneys, tanks, etc.
This document references the security and the correct usage and durability conditions. The implementantion of requisites such as fire resistance of thermic & acoustic insulation are not contemplated in this document.

Quality controls, maintenance and assemblage aspects are briefly contemplated within this document.

4.4 The regulations in the calculation of resistance

Once we have been introduced to the calculation of the structure, we must apply different regulations depending on the selected profile.

- Tubular Profiles. The Structural Steel Instruction must be used (EAE). Or the Technical Code as well (CTE).
- Laminated or Reinforced Profiles. The Technical Code (CTE) must be used.

After this section, we can observe a graph that differentiates the interaction between axial & bending within the sections.

Illustration 14 - Graph of efforts [10]
On the one hand this is a clear example of how in the calculation of the section, regulations EAE and EN-1993 are much more benevolent.

On the other hand the use of CTE is much more conservative, where the resistant area is inferior, leaving an amount of less cases as possible solutions.

5. Softwares

In this section, I will explain what softwares have used for the calculation and for the design of the metallic structure of the project.

5.1 Calculation program

Nowadays calculation by hand is almost extinct (not taking into account some small applications and/or checks) and it generally tends to arrive in the professional environment by the hand of programs that provide infinitesimal, matrix or finite element calculation. Everything will depend on the level of desired precision and the application that is going to be overlooked.

For the field of our application, we will take advantage of a structural calculation program, which already includes both national and international standards for calculation, sizing and testing.

The selected program is called CYPE. It can work with structures of concrete, laminated steel, reinforced steel, formed steel, mixed, aluminium and wood, being all of them subject to gravitational actions, wind, earthquakes and snow.
Currently CYPE 3D automatically generates the weight of the introduced bars that will hence form a hypothesis of own weight. In the event of being necessary, it allows the addition of an indefinite number of additional hypotheses.

The most conflictive area of structures are the nodes. Within this section, CYPE allows the user to define the node as articulated, embedded, semiembedded, etc. And once the type of knot has been defined, we will proceed to define the type of ligation between the nodes.
The ligations between the nodes are used to indicate that two or more nodes have the same displacement in all the selected hypotheses, provided that there is an existing element that links the nodes to allow the fulfilment of the hypotheses.

CYPE 3D uses all the previously explained regulations for his calculation of welded unions, emphasising that En-1993 is their general document and CTE, DB, SE-A, & EAE are the other regulations only appliable within Spanish territory.

**5.2 Design program**

As it has already happened with calculation, the paper-pen method is rarely used to carry out large-scale projects or complex sets. We currently have a wide range of design programs, even providing plans in several dimensions.

For our project we will use the program called Tekla Structures. This design program allows us to model the structure in 3 dimensions, to make complex connections and to be able to visualize the project from any desired perspective.
Tekla structures has a parametric concept that allows you to modify and create elements quickly and easily.

When creating unions, they depend on the measures of the concomitant beam or pillar. Therefore, when modifying the dimensions, the program will automatically make the relevant changes in that area.

This can be a great advantage, but also a great point to take into consideration due to the large number of failures that depend on this function. Once a piece changes its dimensions, it is instantly considered as a new piece. Therefore, the program re-generates a totally different identification number.
If assembly plans have been delivered, it will happen that two different marks will appear for the same piece, thing which can sometimes be differentiated by only 1mm of difference in a piece of 2m in length, which on place is undetectable for the human eye or measure elements due to the % of human error.

Therefore, all assembly plans with this new numbering must be regenerated.

Once the design has been completed, you can generate the general design, the exploding and manufacturing plans and also obtain as well the list of materials and therefore part to do the pertinent purchases of the necessary material with its dimensions. With all this, the certification of the structure can be done later.

I am going to attach down below a little fragment of the lists generated by Tekla with the version used for this project (Tekla structures).
<table>
<thead>
<tr>
<th>Perfil</th>
<th>Parte</th>
<th>Material</th>
<th>Número</th>
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</table>

Illustration 22 - Tekla list of materials [13]
6. Characteristics

6.1 Materials

The material for the calculation is structural steel S 275 JR.

The fundamental values for the design of steel parts are:

- The elastic limit. The elastic limit is the unit load for which the yield stage starts, that is, from which the deformations are not recoverable.
- The breakage limit. The limit of breakage is the maximum unit load supported by the steel in the tensile test.

The values of the elastic limit and breakage of the steel that I am going to use are:

- Elasticity Module: \( E = 210 \) GPa
- Stiffness Module: \( G = 81 \) GPa
- Poisson's coefficient: \( \nu = 0.3 \)
- Coefficient of thermal expansion: \( \alpha = 1.2 \times 10^{-5} \) (ºC)-1
- Density: \( \rho = 7.850 \) kg/m³

6.2 Geometry

The geometry of the house is based on two rectangular floors of 16.6 x 9.6 and 13.55 x 6.75 meters, with a height of approximately 6.5 meters.

This is the original plan received to design and calculate the house, in which notes are found by the architecture.
In this view in 3 dimensions, they request to eliminate, if possible, some beams and pillars once the calculation is being made.

6.3 Location

The structure will be assembled in Passeig del Nard nº 101, 08195 Sant Cugat del Vallès, Barcelona.

Location in coordinates: 41°27'58.4" N, 2°02'42.0" E
7. **Structural design**

The design has been carried out by VazqueziBenages, an architecture firm specialising in houses, pools and spas.

This structure has been designed with a modern and simple base, and simple, clean lines with a focus on minimalism.

*Illustration 25 - Architecture design 1 [14]*

*Illustration 26 - Architecture design 2 [14]*

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The whole main structure will be hidden between the walls; as the structure is made of steel, the pillars can be more slender.

The joists will be the base of the roof, which will also be hidden.

7.1 Anchorage plaques

All the pillars have been anchored to the ground through a threaded M16 rebar.

Below is a detail of how the anchor plates are formed.

Illustration 27 - Anchorage plaque [13]

All the rebars are introduced into the concrete together with maximum-performance hybrid resin for heavy load anchors, in this case model HIT-HY 200-A by HILTI (see spec sheet included in the annexes).

Illustration 28 - Hilti tool [15]
7.2 Floor Distribution

The house consists of two floors; this is further explained below, together with floor and roof plans.

7.2.1 Pillar plan

The pillar distribution of this house is as follows:

Illustration 29 - Pillars plan [13]

Attempts have been made to meet architectural requests, keeping the number of maintaining inner pillars to a minimum, and when pillars are necessary, to make them as slim as possible.

There are a total of 18 pillars, all of them located with the wings in a horizontal position, as can be seen in the above image.
7.2.2 1st Floor

On this floor, although different widths of laminated profiles coexist, all of them have the same shape, namely the IPE profile.

For the main joints we use a wider profile, IPE 200.

In cases where it was necessary to increase resistance, IPE 270 and IPE 240 profiles were used. The cantilevers are formed with an 80x4mm square tubular profile.

As we can see, the shape of the house on the first floor is simple, made with a set of rectangles.

The main beams are embedded in the stiffened pillars with their corresponding stiffeners. The joists are also encased in the beams making the necessary cuts in the wings.

Below we will list the main pillars and their elements.
7.2.3 Roof

The roof of the house follows the same trend as its other parts, with simple, flat forms and welded joints.

The roof has been generated with IPE 120 profiles for horizontals and IPE 140 profiles for verticals.

The cantilevers are formed by 80x4mm square tubes, but in this case they are longer than on the first floor. Later in this study we will analyse in detail the stresses to which this cantilever is exposed and how the rectangular tube can support the loads with so much distance between supports.

Illustration 31 - Roof plan [13]

Only the outer pillars start at the base of the structure and finish on the cover, thus ensuring that the first floor is kept free of interior pillars, as requested.

All the pillars end with a plate so as to close at the top.
7.3 Double cover

The roof of the house is a gable roof towards the interior, consisting of a slope of approximately 8°.

In the following image the cover is highlighted in blue.

![Illustration 32 - Side view plan [13]](image1)

The upper part of the roof is completely hidden with the elevation of the cantilever, the entire water slope will be made through a central gutter with a downpipe.

The roof panels will be supported on the 80mm galvanized square tube upper structure.

![Illustration 33 - Cover plan [13]](image2)
In this image we can more clearly see the slope of the roof, as it carries water into the galvanised channel.

- The channel will be manufactured in 1.5mm galvanised sheet.

### 7.4 Main pillars

This is the main pillar of the house - it is the highest and thickest one.

It is a laminated profile HEB 180, corresponding to European regulations, and its section has a double T shape with a weight of 51.2kg/m. This type of profile supports large forces of torsion, bending and compression in pillars, which is why it is most commonly used in pillars of a metal structure. All this is thanks to its symmetrical exterior dimensions ensuring that the two moments of inertia are more balanced, without creating a moment change when turning the central face.

There are two very similar variants in terms of shape, but the weight per meter will vary, being lighter (HEA) or heavier (HEM) depending on the needs of each structural engineer and designer.

(If the full plan is attached under the Drawings document)
Below is the list of parts of the CN3 set:

**LISTA DE PARTES DE CONJUNTO**

<table>
<thead>
<tr>
<th>Parte</th>
<th>Perfil</th>
<th>Material</th>
<th>Número</th>
<th>Long. (mm)</th>
<th>Peso (kg)</th>
</tr>
</thead>
<tbody>
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<td>P6</td>
<td>PL10&quot;x65</td>
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<td>148</td>
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</table>

Total conjunto: 352.6 kg

Illustration 35 - CN3 parts [13]

This pillar is formed by several pieces that simulate the continuity of the concomitant beams. The pieces called stiffeners are used to stiffen the knot.

Thanks to these pieces, we could say that the pillar is composed of 3 sections, so that each floor can be welded on it in the designated area, although physically we can see that it is in fact only one section.

In addition, a 180x180x8 plate is installed in the upper part to create the pillar capital.

**7.5 Main Beams**

**7.5.1 First Floor**

This is the main beam of the structure as it is the widest. It supports several main first floor joists and it will also join two of the building’s structural pillars.

The laminated profile IPE270 used is also commonly known as the double-side profile of parallel faces, where the number following the acronym IPE indicates the total height of the profile expressed in millimetres. The weight of this profile is 36.1kg/m, maintaining a proportion between profile height and wing width less than 2/3.

(the full plan is attached under the Drawings document)
This is the list of parts of the CN9 set:

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The beams tend to have fewer elements added than the pillars, but these elements are no less important because of it; thanks to them, we can more easily weld the main joists, so the exact position can already be indicated right from the workshop, thus avoiding potential errors in the construction.
### 7.5.2 Roof

In the roof the same beam will be used, but in a higher position; in this case it is a narrower profile, but still larger compared to the rest.

It is an IPE140 profile with a weight of 12.9 kg/m. It is a leaner beam, but it still tolerates the force to which it is subjected.

The width of the wings of this profile is 73 mm, wide enough to fix the panel by mechanical anchors, without major issues.

(the full plan is attached under the Drawings document)
This is the list of parts of the CN69 set:

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Illustration 39 - CN69 parts [13]

This beam, like the previous one, does not have many additional elements, but it is of great help that it can be made in the workshop, thus again avoiding possible errors in the construction and facilitating assembly.

7.5.3 Joist

The main joist is inserted inside the section of the beam, since its positioning might lead to confusion once the work has begun if we only observe how it is located.

In this case we will not have a list of set parts nor large sizes in terms of section in the laminated profile.

The weight of this laminated profile is 15.8kg/m.
The IPE160 profile is surprising as we do not expect a joist to be wider than the main beam of the roof, but this joist is extracted from the first floor, which will constantly receive loads, while the roof will only support the snow load and one-off services, actions that we assume will be non-concomitant.

(the full plan is attached under the Drawings document)

8. Calculation

This is the three-dimensional structural set rendered by Cype3D.

Illustration 41 - Structure Cype 3D [13]

8.1 Calculation

The calculation is carried out using the matrix analysis method using the Cype3D computer program, performing all the load combinations with the coefficients recommended by the CTE-DB-SE and checking strains and deformations in all the bars. Everything is in accordance with regulations, especially the CTE and the EAE.

Ruben Extremera Hidalgo
8.2 Adopted Solution

Once we started calculating, we had to make some changes.

We respect the design but the cantilevered structure does not meet applied loads, so we reinforce that section with some extra joists.

All calculations have been based on those previously made by the company manager, with his help in drawing the structure in Cype3D and making the sets, among other things.

8.2.1 Pillars

Pillars are adopted by HEA profiles since they have a good inertia-resistance ratio to resist loads.

The following parameters are calculated using Cype3D:

- Limitation of slenderness.
- Web dent.
- Resistance to traction.
- Resistance to compression.
- Resistance to flexion.
- Cut resistance.
- Resistance to bending moment and combined shear force.
- Combined bending and axial resistance.
- Resistance to bending, axial and shear combined.
- Resistance to torsion.
- Resistance to shear and torque combined.

Below we will detail the results of the pillar with more unfavourable forces, verifying each of the points indicated above against the CTE DB SE-A regulation, thus ensuring the HEB 180 profile is adequate.
8.2.2 Beams

IPE laminated profiles are adopted for the solution of the main beams. The following parameters are calculated using Cype3D:

- Limitation of slenderness.
- Web dent.
- Resistance to traction.
- Resistance to compression.
- Resistance to buckling
- Resistance to flexion.
- Cut resistance.
- Resistance to bending moment and combined shear force.
- Combined bending and axial resistance.
- Resistance to bending, axial and shear combined.
- Resistance to torsion.
- Resistance to shear and torque combined.

Below we will detail the results of the pillar with more unfavourable forces, verifying each of the points indicated above against the CTE DB SE-A regulation, thus ensuring the IPE 270 profile is adequate.

8.2.3 Joists

IPE laminated profiles are adopted for the solution of secondary beams or joists. The following parameters are calculated using Cype3D:

- Limitation of slenderness.
- Web dent.
- Resistance to traction.
- Resistance to compression.
- Resistance to buckling
- Resistance to flexion.
- Cut resistance.
- Resistance to bending moment and combined shear force.
- Combined bending and axial resistance.
- Resistance to bending, axial and shear combined.
- Resistance to torsion.
- Resistance to shear and torque combined.

Below we will detail the results of the pillar with more unfavourable forces, verifying each of the points indicated above against the CTE DB SE-A regulation, thus ensuring the IPE 160 profile is adequate.

8.2.4 Joints

We will now look at welded joints between pillars and beams.

The joint will have the following form:

*Illustration 42 - Joint between pillar and beams [13]*
The joint between the pillar and the anchor plate is included in the calculation annexes, section 2.

8.2.5 Stiffeners

In our calculation program Cype3D we will include the option of stiffeners with cuts in the vertices. We use this option since our profiles have a curvature between the joint of the wings and the wen, so as to avoid creating conflicting points in that area.

![Illustration 43 - Cype stiffeners options [13]](image)

8.3 Actions to consider

The following actions are considered:

- **Own weight of the steel structure**: The density of the steel is 7,850 kg/m$^3$, taking this into account the calculation program automatically introduces the weight of the defined profile in each bar.

- **Snow overload**: 50 kg/m$^2$ are considered.

- **Overload of use**: 300 kg/m$^2$ are considered, with a margin of 100kg/m$^2$.

- **Permissible arrows**: We consider arrow <(L/300) for gantry beams and joists.

- **Permissible displacement**: We consider displacement at the tip of <(h/250) for pillars.
- **Forged own weight**: 360kg/m² and 20kg/m² are considered, depending on the floor.

- **Wind loads**:
  - V1: 0.065 kg/m².
  - V2: 0.045 kg/m².
  - V3: 0.065 kg/m².
  - V4: 0.045 kg/m².

### 8.4 Combinations of actions

The combinations selected for the calculation are included in the calculation annex, section 1. In section 2 of the calculation annex we can find an example of a calculation with the stress combinations in the main pillar, a HEB 180, whose selection will be justified later.

### 8.5 Resulting set

This is the summary of data obtained with the selected profiles to support the applied loads.
8.5.1 Analysis of the most unfavourable pillar

These are the different profiles that the program allows us to select, within the HEB laminated profile.

![Illustration 45 - HEB180 pillar checks [13]](image)

The first profile that meets all the demands is the profile used, in this case HEB 180.

If we analyse in terms of the arrow, the pillars do not tend to flex to a great extent, but the upper part does tend to displace as soon as they support the loads of the structure.

In terms of resistance, the first profile that complies is also within our limit of 80% resistance.

Please see attached the list of results in the calculation annexes, section 3.

In this section we can see all the test that were carried out and their results until the 0.74 safety in resistance was obtained.
8.5.2 Analysis of the most unfavourable beam – 1st Floor

These are the different profiles that the program allows us to select, within the laminated IPE profile.

![Illustration 46 - IPE270 beam checks](image)

If we select the profile in terms of the arrow, we can see that a 140mm profile would already comply with our limit of L/300, but as we observe the resistance we see that it is very thin and thus will not support the loads applied in that area. We therefore continue to oversize profiles until we find one that meets our required force demands.

The first profile that meets the resistance in this case is not the profile used for this first-floor beam, since we do not install profiles below 80% resistance; in this case we will install an IPE 270 profile.

Please see attached the list of results in the calculation annexes, section 4.

Ruben Extremera Hidalgo
8.5.3 Analysis of the most unfavourable beam – roof

The following are the different profiles that the program allows us to select, within the laminated IPE profile.

![Illustration 47 - IPE140 beam checks [13]](image)

This study is similar to that of the first-floor beam: in terms of resistance we can opt for leaner profiles, but in terms of the arrow we must opt for greater moments of inertia to resist the loads applied especially in bending.

The first profile that meets our demands is not the profile used for this roof beam, since we do not install profiles below 80% of resistance in, either in pure resistance or bending; thus we will install an IPE profile 140.

Please find attached a list of results in the calculation annexes, section 5.
8.5.4 Analysis of the most unfavourable joist

The following are the different profiles that the program allows us to select, within the laminated IPE profile.

The first profile that the program shows us as a viable option is the profile that was finally selected, as it already has a resistance below 80% and we believe that it is more than enough to support the requested loads.

Please find attached the list of results in the calculation annexes, section 6.
8.5.5 Cantilever analysis (80x4mm tubes)

Below is an analysis of the safety of this 80mm square tube to carry out the cantilevers.

![Illustration 49 - Tube 80x4 checks](image)

This tube is very oversized, but we do not know what will hang from it, therefore we can proceed to oversize, as a 40mm tube already meets our demands in terms of resistance in several areas.

In addition, we need a good base to anchor the façade panels, and since the IPE 140 profile has a base of 73mm, we select the first similar profile that is superior in terms of base, since the difference in weight and price between a 70mm and an 80mm base is trivial.

Please see attached the list of results in the calculation annexes, section 7.
8.6 Deformed structure

This is the deformed structure under the set of actions of own weight, overload of use, wind and snow.

Under the simultaneous actions of these loads, the greatest possible deformation is obtained, and it meets the demands.

Illustration 50 - Deformed structure [13]
8.7 Main pillar displacement

The maximum tolerated displacement must be less than h/250.

Below are the results of the analysis of the most unfavourable pillar.

Illustration 51 - Pillar displacement 1 [13]

Illustration 52 - Pillar displacement 2 [13]

The results are valid within the established margin.

\[(7,108\,\text{mm}) < (h/250)\]
9. Changes made

The structure has been going through constant changes before its conclusion, some of them more important, some of them less. However, I would highlight the great change that was made once all was designed.

Firstly, the incorporation of the porch.

This is the first model that we did to present it.

After this model, they changed it and included a porch, and we incorporated some extra joints to stiffen it. The joints were added in the cantilever of the main floor and in the upper floor.

You can find attached a picture of the 3D drawing, which shows how everything was distributed before the final changes.
By this moment, the structure was still very simple and had flat floors.

The whole distribution of the previously explained loads was made in accordance to these changes, but the upper cantilever was not rigid enough to withstand the loads of awnings and windows.

At that point, they made new heights for the cantilevers of the main and the upper floors, to raise the roof perimeter.
Here you can find the final distribution. You can see the changes mentioned in the cantilevers (not only in rigidity, but also in height). In some of the pillars, a supplementary piece has been added. However, some other pillars have been modified to change the length of the profile.
10. Assembly

To start with the work, we went to calculate the topography levels to buy the initial pillars to the exact length measure. Once the measures were taken, we proceeded to design and purchase the material.

![Material on the site](image)

Illustration 56 - Material on the site [13]

10.1 Process

The anchorage plaques were put with chemical HIT-HY 200-A by Hilti with threaded rod, washers and nuts. (Find attached a technical sheet in annexes)

This is the drawing design for the roof of pillars.
All the pillars have the same orientation. It is not mandatory to fulfill this method in the structures, because everything will depend on where to join the next beam.

However, as you can see in the following picture, adding some stiffeners can allow us to proceed to lengthen the wings of the beam.
The pillars and beams were made to measure, and all the pillars were assembled workshop, already incorporating stiffeners, connection angle, transverse plates, etc.

Illustration 59 - Assembly process 2 [13]

The assembly was made with a truck crane for the pillars, and manually helped by platforms for the beams.

Illustration 60 - Assembly process 3 [13]
The joints have been solved by welding. There are different types of weldings inside the structure.

The weldings within pillars and main beams have been welded through all the perimeter. In the joints, it has only been welded the web of the beam.

This is the first floor without having made the height change in the perimeter and cantelivers.

Actually, the structure has finished within the established deadlines. Currently, the structure has been finished within the established deadlines. After finishing the structure, the company asked us to prepare an additional metallic part and the composite slab. This additional job will be included in the budget as an extra part.
11. Environmental impact

This section will summarise the environmental conditions generated by this structure: from the extraction of the raw material to the final product.

11.1 Environmental impact in the steelmaking process

11.1.1 Raw material treatment

During the steelmaking process, large quantities of wastewater and atmospheric emissions are generated. This wastewater has to be treated properly since it could potentially cause earth, water and air contamination. The use of blast furnaces for the treatment of iron generates particles that contain carbon monoxide. These blast furnaces also generate slag that once cooled by water, produces carbon monoxide and hydrogen sulphide.

All this is added to with the atmospheric emissions generated in the process and released into the environment through exhaust gases.

11.1.2 Steel Transformation

During the melting of steel, solid waste, gases and liquids are generated. Most solid waste can be recycled to generate other elements with subsequent proper processing. The liquid chemical products used to clean the steel must be carefully handled due to their high level of danger; the gases generated are carbon monoxide and sulphur dioxide.

11.1.3 Effects on the environment

Steel production also contributes to the depletion of major resources. Not all the material produced is used – an amount is transformed into gases and fumes from the factories where the welds are made, generating pollution in the atmosphere due to the exchange of temperatures with the ambient temperature, which on a small scale affects the greenhouse effect. Again, there is a moral dilemma to be discussed in terms of reconciling industrial and environmental interests, since a simple change in the material used would produce positive effects on the environment, but on an industrial level this would generate costs that would not be amortized over the years.
11.2 Environmental impact on assembly

During the last phase, there are several sources of emission, both in the material supply system and the on-site assembly. CO$_2$ emissions are produced during the transport of the steelworks to the factory where the pre-assembly of the pillars and main beams takes place, and then from the factory to the location where the assembly will take place.

Once on-site, welding gases and temperature rises occur in the temperature due to welding.

12. Planning

This is the planning that I used during the project, I will use a Gantt chart to try to be as explanatory as possible.

![Gantt chart](image)
13. Conclusions

The objective of this project has been to cover both design and calculation of the structure, all helped by some partners, all the follow-up work has been carried out daily or weekly on some occasions by me.

The decision of choosing this project was thanks to the union within the engineering of metal structures and housings, a field which I had not studied in depth, and which I think it is a big step towards the future.

All the learning of the design program Autocad and Tekla Structures has been carried out in the company. During the university stage I have not been able to acquire knowledge of them, but other similar programs like Solidworks gave me an idea of how to model and work in 3 dimensions.

On the part of the calculation, during my academic stage I have been able to learn to calculate structures manually (on paper) and with finite element programs. However, all the calculations with a program dedicated to structures has had to be learnt during the project process, since I did not know how Cype3D worked.

I have learnt to calculate and design a metallic structure of these dimensions in its entirety, as well as being able to track and budget it.

This process of learning has given me a lot of knowledge about concepts which I will constantly use in my professional life, not only in the construction enviroment.

The calculation, design and budgets are daily tasks in the field of engineering, as well as the organization in the industrial environment, so this project has included the most favorable points to continue my growth as a professional in the world of engineering.

14. Future actions

As future actions we could consider studying the calculation of the foundations, the urban regulations and an exhaustive calculation of the roof area. All sanitary, electric and gas facilities would also be a broad object of study as a possible future action.
15. Bibliography

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ESEIAAT
Degree in Mechanical Engineering

Study of design and calculation of the structure of a house in the municipality of Sant Cugat del Valles.

Budget

Author: Rubén Extremera Hidalgo
Tutor: Montserrat Sánchez Romero
Terrassa, January 10, 2019
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Table 3 - Cost of the project. Own source ........................................................................................ 5
1. Budget

In this section we specify the economic calculation necessary to build the metal structure for a house of these characteristics.

1.1 Material

Please find attached a list of material for the structure.

These components were sent to more than three companies for quoting; once we receive the quotes we attempted to negotiate a better price for them together with the firm Comercial de Laminados.
### Illustration 1 - Material list. Own source

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<td>A MEDIA</td>
<td>182.5</td>
<td></td>
</tr>
</tbody>
</table>

Ruben Extremera Hidalgo
After that we proceeded to confirm this order with the previously established prices.

- All beams are made of steel with 275 quality as this is the limit of elasticity that we consider in the calculations within Cype3D.
- The budget of composite slab will be invoiced in other budget items too.
- The budget for the sheet metal is separate from the composite slab so as to avoid confusion between the linear meters of the sheet metal and the square meters of the composite slab.

The total weight of the material relating to hot laminated profiles is 6,302.1kg. The weight of tensors, anchor plates and other elements added to the structure must also be considered and invoiced accordingly.

The structure weight a total of 6661,9kg.

### 1.2 Processing cost

This is the cost of the preparation of the project, from the completion of the Project Charter to daily follow-up of the work, ending with the preparation of the documentation.

<table>
<thead>
<tr>
<th>Concept</th>
<th>Units (hours)</th>
<th>Cost per unit (€/h)</th>
<th>Total cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Preparation of the Project Charter</td>
<td>10</td>
<td>10</td>
<td>100,00 €</td>
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<tr>
<td>Search of information</td>
<td>45</td>
<td>10</td>
<td>450,00 €</td>
</tr>
<tr>
<td>Structure design</td>
<td>120</td>
<td>10</td>
<td>1,200,00 €</td>
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<tr>
<td>Choice of materials and profiles</td>
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<td>10</td>
<td>200,00 €</td>
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<tr>
<td>Calculation of the structure</td>
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<td>900,00 €</td>
</tr>
<tr>
<td>Manufacturing and assembly follow-up.</td>
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<td>10</td>
<td>300,00 €</td>
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<td>Project documentation</td>
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<td>2,000,00 €</td>
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<tr>
<td><strong>Total</strong></td>
<td><strong>515</strong></td>
<td><strong>10</strong></td>
<td><strong>5,150,00 €</strong></td>
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</tbody>
</table>

*Table 1 - Processing cost. Own source*

The total calculation of this part is **€5,150.00**.

### 1.3 Cost of materials

In this table we introduce the price of the materials, both laminated profiles and tensors, the collaborative sheet and the perimetric topping frame.
Study of design and calculation of the structure of a house in the municipality of Sant Cugat del Vallès

<table>
<thead>
<tr>
<th>Concept</th>
<th>Units</th>
<th>Cost (€/ud)</th>
<th>Total cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Metallic structure (kg)</td>
<td>6661.9</td>
<td>0.85</td>
<td>5662.62 €</td>
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<tr>
<td>Supply composite slab 0.75 (m²)</td>
<td>110</td>
<td>12.65</td>
<td>1391.50 €</td>
</tr>
<tr>
<td>Galvanized sheet metal 1.2 (m)</td>
<td>65</td>
<td>4.1</td>
<td>266.50 €</td>
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<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td><strong>7320.62 €</strong></td>
</tr>
</tbody>
</table>

*Table 2 - Cost of the material. Own source*

The price of a kilo of steel is €0.85, the rest of the components are on-site supplies for subsequent assembly.

The total calculation of this part is **€7,320.62**.

**1.4 Business profit**

We will establish a 20% business profit, calculated as 10% to cover the general expenses of the company, and a 10% of net profit.

In these small works the benefit is very small; we therefore currently carry out an assembly where we can extract a greater profit from the labour force, since the supply of materials does not cover large expenses.

**1.5 Total cost of the project**

Finally, here is a summary including the business profit.

<table>
<thead>
<tr>
<th>Concept</th>
<th>Total cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Processing cost</td>
<td>5150.00 €</td>
</tr>
<tr>
<td>Cost of the material</td>
<td>7320.62 €</td>
</tr>
<tr>
<td>Business profit</td>
<td>20%</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>14,964.74 €</strong></td>
</tr>
</tbody>
</table>

*Table 3 - Cost of the project. Own source*

The total cost of this project is **fourteen thousand nine hundred sixty-four euros and seventy-four cents (€14,964.74)**.
ESEIAAT
Degree in Mechanical Engineering

Study of design and calculation of the structure of a house in the municipality of Sant Cugat del Valles.

Drawings

Author: Ruben Extremera Hidalgo
Tutor: Montserrat Sánchez Romero
Terrassa, January 10, 2019
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LISTA DE PARTES DE CONJUNTO:

Conjunto: CN1
Número: 1
Long. (mm): 6130

<table>
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<tr>
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<th>N.º</th>
<th>Material</th>
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<th>Peso (kg)</th>
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<td>PL107/1</td>
<td>5275/R</td>
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Total por conjunto: 62.2
LISTA DE PARTES DE CONJUNTO

Conjunto: CN3
Acabado: 5540

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Total por conjunto: 350.6
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<td>A108</td>
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Total: 55.3
**LISTA DE PARTES DE CONJUNTO**

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**Acabado:**  
**Números:** 1  
**Long. (mm):** 6540

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**Total:** 218 kg

**B - B**  
**C - C**  
**A - A**
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Total por conjunto: 246.1 kg
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Total por conjunto: 2497
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Total por conjunto: 72.7
### Lista de Partes de Conjunto

**Conjunto:** CN82  
**Acabado:**  
**Número:** 1  
**Long. (mm):** 3435

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<tr>
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<th>Núm.</th>
<th>Long. (mm)</th>
<th>Peso (kg)</th>
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<tbody>
<tr>
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<td>1.19</td>
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</table>

**Total de partes:** 21,6

**Seción A - A**
ESEIAAT
Degree in Mechanical Engineering

Study of design and calculation of the structure of a house in the municipality of Sant Cugat del Valles.

Annexes

Author: Ruben Extremera Hidalgo
Tutor: Montserrat Sánchez Romero
Terrassa, January 10, 2019
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1. Components annex

PERFIL: REF. INCOFLUID
70/210/840 L.M.
PARA FORJADO COLABORANTE

DIMENSIONES

Fachada
Cubierta

TABLA DE PESOS PROPIOS

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<th>Esp (mm)</th>
<th>H (cm)</th>
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<tr>
<td>20</td>
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Por metro cuadrado de losa según el espesor "e" de la chapa y la anata total "H" de la losa; dado el daN/m².

1 daN/m² = 1kN/m²
Hormigón de resistencia característica: 25 N/mm²

NORMATIVA

EUROCÓDIGO - 4
"Reglas generales y reglas para la edificación".
UNE-ENV 1994-1-1
UNE-ENV 1994-1-2
ENV 1993-1-3
"Reglas generales. Proyecto de estructuras sometidas al fuego".
"Cold formed thin gauge members and sheeting".

MATERIAL

Acero de calidad DX51D según UNE-EN 10142.
Acero de calidad estructural S280GD según UNE-EN 10147.
Acero de calidad estructural S320GD según UNE-EN 10147.

Re = 320 N/mm
Rm = 390 N/mm

ARMADURAS DE NEGATIVOS

Es muy recomendable recurrir a una solución con malla electrosoldada continua en toda la losa.
La armadura deberá tener un reubrimiento de 20 mm.
Debe disponerse piezas especiales para su elevación respecto a la chapa.
Incluye control de fisuración y retracción.

Acero B500S
f_yk = 500 N/mm²
φ_y = 1.15

Ruben Extremera Hidalgo
Study of design and calculation of the structure of a house in the municipality of Sant Cugat del Vallès

Ruben Extremera Hidalgo
HIT-HY 200 injection mortar
Anchor design (ETAG 001) / Rods&Sleeves / Concrete

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<th>Injection mortar system</th>
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<td>Hilti HIT- HY 200-A</td>
<td>- SafeSet technology: drilling and borehole cleaning in one step with Hilti hollow drill bit</td>
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<td>500 ml foil pack (also available as 330 ml foil pack)</td>
<td>- Suitable for non-cracked and cracked concrete C 20/25 to C 50/60</td>
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| Hilti HIT- HY 200-R | - ETA Approved for seismic performance category C1, C2
| 500 ml foil pack (also available as 330 ml foil pack) | - Maximum load performance in cracked concrete and non-cracked concrete |
| Anchor rod: | - High corrosion / corrosion resistance
| HIT-V | - Small edge distance and anchor spacing possible |
| HIT-V-F | - Manual cleaning for borehole diameter up to 20mm and \( h_d \leq 10d \) for non-cracked concrete only |
| HIT-V-R | - Two mortar versions: HY 200-R for slow cure applications and HY 200-A for fast cure applications |
| HIT-V-HCR (M8-M30) | |
| Internally threaded sleeve: | |
| HIS-N | |
| HIS-RN (M8-M20) | |
| Anchor rod: | |
| HIT-Z | |
| HIT-Z-F | |
| HIT-Z-R (M8-M20) | |

a) HIS-N internally threaded sleeves not approved for Seismic.
b) High Corrosion resistant rods available only for HIT-V. Corrosion resistant rods available for HIT-V and HIS-N

c) Diamond drilling only covered for HIT-Z rods

d) HIS-N internally threaded sleeves not approved for Seismic category C2.
e) High Corrosion resistant rods available only for HIT-V. Corrosion resistant rods available for HIT-V and HIS-N.

Base material | Installation conditions
--- | ---
Concrete (non-cracked) | Concrete (cracked)
Hammer drilled holes | Diamond drilled holes
Hilti SafeSet technology | Variable embedment depth
Small edge distance and spacing

Load conditions | Other information
--- | ---
Static/ quasi-static | Seismic, ETA-C1, C2
Seismic | Fire resistance
European Technical Assessment | CE conformity
Corrosion resistance | High corrosion resistance
HCR highMo | PRORS Anchor design Software

Ruben Extremera Hidalgo
Study of design and calculation of the structure of a house in the municipality of Sant Cugat del Vallès

Ruben Extremera Hidalgo
2. Calculation annex

2.1 Section 1

 Rolled steels: CTE DB SE-A

**Category of use:** A. Residential areas

**Limit states**

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**Combinations**

- **Names of hypotheses**

  PP - Own weight
  Q 1 Q 1
  V 1 V 1
  V 2 V 2
  V 3 V 3
  V 4 V 4
  N 1 N 1

- **E.L.U. of breakage. Rolled steel**

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Ruben Extremera Hidalgo
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2.2 Section 2

2.2.1 Hypotheses

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### Study of design and calculation of the structure of a house in the municipality of Sant Cugat del Vallès

Ruben Extremera Hidalgo

#### Efforts in pillar, by hypothesis

<table>
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<td>Mt</td>
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<td>0.766 m</td>
</tr>
<tr>
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<td>My</td>
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<td>Mz</td>
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#### 2.2.2 Combinations

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<tr>
<td></td>
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<td>Mz 0.193</td>
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|        | Vz         | 0.623 m |
|        | Mt         | 0.000 m |
|        | My         | 0.704 m |
|        | Mz         | 0.193 m |

|        | Vz         | -0.174 m |
|        | Mt         | 0.000 m |
|        | My         | 0.704 m |
|        | Mz         | 0.193 m |

| 1.35·PP | N 12.09 m |
|        | Vz 0.029 |
|        | Mt 0.000 |
|        | My 0.798 |
|        | Mz 0.326 |

|        | Vz         | -0.127 m |
|        | Mt         | 0.000 m |
|        | My         | 0.798 m |
|        | Mz         | 0.326 m |

| 0.8·PP+1.5·Q1 | N 16.71 m |
|               | Vz 0.430 |
|               | Vz 1.589 |
|               | Mt 0.000 |
|               | My 1.782 |
|               | Mz 0.482 |

| 1.35·PP+1.5·Q1 | N 21.63 m |
|                | Vz 21.61 m |
|                | Vz 21.586 m |
|                | Vz 21.559 m |
|                | Vz 21.533 m |
|                | Vz 21.506 m |
|                | Vz 21.480 m |
|                | Vz 21.453 m |

Ruben Extremera Hidalgo
<table>
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## Study of design and calculation of the structure of a house in the municipality of Sant Cugat del Vallès

### Efforts in bars, by hypothesis

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<th>Description</th>
<th>Effort Positions in the pillar</th>
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<td>1.311 -0.401 -7.837 -0.188 1.559 0.001 2.511 -0.138 7.853 0.188 1.559 0.001 2.511 -0.138</td>
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<tr>
<td>1.35-PP+1.5-V2</td>
<td>N</td>
<td>1.311 -0.401 -7.837 -0.188 1.559 0.001 2.511 -0.138 7.853 0.188 1.559 0.001 2.511 -0.138</td>
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<td>0.8-PP+1.05-Q1+1.5-V2</td>
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Ruben Extremera Hidalgo
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### Efforts in bars, by hypothesis

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### Study of design and calculation of the structure of a house in the municipality of Sant Cugat del Vallès

- **Ruben Extremera Hidalgo**
### Efforts in bars, by hypothesis

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<th>Combination</th>
<th>Effort</th>
<th>Positions in the pillar</th>
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<td>m</td>
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<td>Vz 1.956</td>
<td>1.956</td>
<td>1.956</td>
</tr>
<tr>
<td></td>
<td>Mz 0.344</td>
<td>-0.225</td>
<td>-0.105</td>
</tr>
</tbody>
</table>

Ruben Extremera Hidalgo
## Study of design and calculation of the structure of a house in the municipality of Sant Cugat del Vallès

<table>
<thead>
<tr>
<th>Pillar Type</th>
<th>Pillar Combination</th>
<th>Effort Description</th>
<th>Effort (m)</th>
<th>Positions in the pillar (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>1.35·PP+1.05·Q1+1.5·V2+0.75·N1</td>
<td>Mz 0.413</td>
<td>-0.271</td>
<td>-0.128</td>
</tr>
<tr>
<td>Vz</td>
<td>2.203</td>
<td>Mt 0.001</td>
<td>0.001</td>
<td>0.001</td>
</tr>
<tr>
<td>My</td>
<td>3.243</td>
<td>0.673</td>
<td>2.399</td>
<td>1.554</td>
</tr>
<tr>
<td>Mz</td>
<td>0.546</td>
<td>2.547</td>
<td>2.718</td>
<td>1.710</td>
</tr>
<tr>
<td>Vz</td>
<td>2.632</td>
<td>Mt 0.001</td>
<td>0.001</td>
<td>0.001</td>
</tr>
<tr>
<td>My</td>
<td>3.726</td>
<td>0.585</td>
<td>2.399</td>
<td>1.554</td>
</tr>
<tr>
<td>Mz</td>
<td>0.546</td>
<td>2.547</td>
<td>2.718</td>
<td>1.710</td>
</tr>
</tbody>
</table>

| N           | 0.8·PP+1.5·Q1+0.9·V2+0.75·N1 | Mz 0.413            | -0.358    | -0.170                     |
| Vz          | 2.632              | Mt 0.001            | 0.001     | 0.001                      |
| My          | 3.726              | 0.585              | 2.399     | 1.710                      |

| N           | 1.35·PP+1.5·Q1+0.9·V2+0.75·N1 | Mz 0.413            | -0.358    | -0.170                     |
| Vz          | 2.632              | Mt 0.001            | 0.001     | 0.001                      |
| My          | 3.726              | 0.585              | 2.399     | 1.710                      |
| Mz          | 0.493              | 0.563              | 0.563     | 0.563                      |

| N           | 1.35·PP+1.5·Q1+0.9·V2+0.75·N1 | Mz 0.413            | -0.358    | -0.170                     |
| Vz          | 2.632              | Mt 0.001            | 0.001     | 0.001                      |
| My          | 3.726              | 0.585              | 2.399     | 1.710                      |
| Mz          | 0.493              | 0.563              | 0.563     | 0.563                      |

| N           | 0.8·PP+1.5·V3+0.75·N1 | Mz 0.413            | -0.358    | -0.170                     |
| Vz          | 2.632              | Mt 0.001            | 0.001     | 0.001                      |
| My          | 3.726              | 0.585              | 2.399     | 1.710                      |

| N           | 1.35·PP+1.5·V3+0.75·N1 | Mz 0.413            | -0.358    | -0.170                     |
| Vz          | 2.632              | Mt 0.001            | 0.001     | 0.001                      |
| My          | 3.726              | 0.585              | 2.399     | 1.710                      |

| N           | 0.8·PP+1.05·Q1+1.5·V3+0.75·N1 | Mz 0.413            | -0.358    | -0.170                     |
| Vz          | 2.632              | Mt 0.001            | 0.001     | 0.001                      |
| My          | 3.726              | 0.585              | 2.399     | 1.710                      |

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<table>
<thead>
<tr>
<th>Combination</th>
<th>Description</th>
<th>Effort</th>
<th>Positions in the pillar</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>0.000 m</td>
<td>0.383 m</td>
</tr>
<tr>
<td>N 16.44 1.489 1.68 0.000 1.911 2.160</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Mz 0.001</td>
<td>-1.502</td>
<td>-0.977</td>
<td>-0.453</td>
</tr>
<tr>
<td>0.8·PP+1.5·Q1+0.9·V3+0.75·N1</td>
<td>N 15.64 1.042 1.55 0.000 1.754 1.462</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Mt 0.000</td>
<td>-1.042</td>
<td>-1.042</td>
<td>-1.042</td>
</tr>
<tr>
<td>My 1.063</td>
<td>-0.663</td>
<td>-0.264</td>
<td>0.135</td>
</tr>
<tr>
<td>Vz 1.982</td>
<td>1.982</td>
<td>1.982</td>
<td>1.982</td>
</tr>
<tr>
<td>Vy 1.479</td>
<td>0.720</td>
<td>-0.040</td>
<td>-0.799</td>
</tr>
<tr>
<td>N 10.62 0.652 0.597 0.000 1.143</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Mz 1.595</td>
<td>-1.150</td>
<td>-0.705</td>
<td>-0.260</td>
</tr>
<tr>
<td>0.8·PP+1.5·Q1+0.9·V3+0.75·N1</td>
<td>N 15.55 0.533 0.925 0.001 1.169</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Mt 0.000</td>
<td>-0.001</td>
<td>-0.001</td>
<td>-0.001</td>
</tr>
<tr>
<td>My 1.010</td>
<td>0.806</td>
<td>0.602</td>
<td>0.397</td>
</tr>
<tr>
<td>Vz 1.723</td>
<td>1.273</td>
<td>1.273</td>
<td>1.273</td>
</tr>
<tr>
<td>Vz 20.418</td>
<td>20.418</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>N 17.30 0.473 0.723 0.001 1.441</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Mt 0.000</td>
<td>-0.001</td>
<td>-0.001</td>
<td>-0.001</td>
</tr>
<tr>
<td>Vz 0.491</td>
<td>0.759</td>
<td>0.578</td>
<td>0.397</td>
</tr>
<tr>
<td>N 22.23 0.353 0.353 0.001 0.380</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Mt 0.000</td>
<td>-0.001</td>
<td>-0.001</td>
<td>-0.001</td>
</tr>
<tr>
<td>My 1.925</td>
<td>1.273</td>
<td>0.621</td>
<td>-0.031</td>
</tr>
<tr>
<td>Vz 17.212</td>
<td>17.212</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Vz 17.196</td>
<td>17.196</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Vz 17.181</td>
<td>17.181</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Study of design and calculation of the structure of a house in the municipality of Sant Cugat del Vallès
2.3 Section 3

Section: HE 180 B
Material: Steel (S275)

<table>
<thead>
<tr>
<th>Nodes</th>
<th>Initial</th>
<th>Final</th>
<th>Length (m)</th>
<th>Mechanical characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Area (cm²)</td>
</tr>
<tr>
<td>N16</td>
<td>N20</td>
<td>3.200</td>
<td>65.30</td>
<td>3831.00</td>
</tr>
</tbody>
</table>

Notes:
- \((1)\) Inertia with respect to the indicated axis
- \((2)\) Uniform torsional moment of inertia

<table>
<thead>
<tr>
<th>Bar</th>
<th>( E )</th>
<th>( A )</th>
<th>( M_{b} )</th>
<th>( M_{c} )</th>
<th>( M_{y} )</th>
<th>( M_{z} )</th>
<th>( V_{y} )</th>
<th>( V_{z} )</th>
<th>( M_{t} )</th>
<th>( M_{t} )</th>
<th>( V_{y} )</th>
<th>( V_{z} )</th>
<th>( M_{t} )</th>
<th>( M_{t} )</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>N16/N20</td>
<td>( \lambda \leq 2.0 )</td>
<td>( \lambda \leq \lambda_{cr} )</td>
<td>Verified</td>
<td>( D.N.P. )</td>
<td>( \lambda_{cr} )</td>
<td>( \eta )</td>
<td>( \eta_{z} = 39.0 )</td>
<td>( \eta_{y} = 39.0 )</td>
<td>( \eta \leq 0.1 )</td>
<td>( \eta \geq 0.1 )</td>
<td>( \eta \leq 0.1 )</td>
<td>( \eta \leq 0.1 )</td>
<td>( \eta \geq 0.1 )</td>
<td>( \eta \geq 0.1 )</td>
<td>Verified</td>
</tr>
</tbody>
</table>

\( \eta = 74.4 \)
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Slenderness limit (CTE DB SE-A, Articles 6.3.1 and 6.3.2.1 - Table 6.3)

The reduced slenderness \( \bar{\lambda} \) of the bars in compression must be less than 2.0.

\[
\bar{\lambda} = \frac{A \cdot f_y}{N_{cr}}
\]

Where:
- **Class**: Section class, depending on its deformation capacity and development of plastic resistance of the compressed elements of a section.
- \( A \): Area of the gross section for class 1, 2 and 3 sections.
- \( f_y \): Yield strength. (CTE DB SE-A, Table 4.1)
- \( N_{cr} \): Critical elastic buckling axial force.

The critical elastic buckling axial force \( N_{cr} \) is the smallest of the following values a), b) and c):

a) Critical elastic buckling axial force with respect to the Y axis.

\[
N_{cr,Y} = \frac{\pi^2 \cdot E \cdot I_y}{L_{cr,Y}^2}
\]

b) Critical elastic buckling axial force with respect to the Z axis.

\[
N_{cr,Z} = \frac{\pi^2 \cdot E \cdot I_z}{L_{cr,Z}^2}
\]

c) Critical elastic buckling axial force due to torsion.

\[
N_{cr,T} = \frac{1}{I_o} \left[ G \cdot I_y + \frac{\pi^2 \cdot E \cdot I_w}{L_{cr,T}^2} \right]
\]

Where:
- \( I_y \): Moment of inertia of the gross section, with respect to the Y-axis.
- \( I_z \): Moment of inertia of the gross section, with respect to the Z-axis.
- \( I_t \): Uniform torsional moment of inertia.
- \( I_w \): Section warping constant.
- \( E \): Modulus of Elasticity.

\[
\begin{align*}
\bar{\lambda} & = 0.81 \quad \checkmark \\
\text{Class} & = 1 \\
A & = 65.30 \text{ cm}^2 \\
f_y & = 275.00 \text{ MPa} \\
N_{cr} & = 2758.77 \text{ kN} \\
N_{cr,Y} & = 7754.10 \text{ kN} \\
N_{cr,Z} & = 2758.77 \text{ kN} \\
N_{cr,T} & = \infty \\
I_y & = 3831.00 \text{ cm}^4 \\
I_z & = 1363.00 \text{ cm}^4 \\
I_t & = 42.16 \text{ cm}^4 \\
I_w & = 93750.00 \text{ cm}^6 \\
E & = 210000 \text{ MPa}
\end{align*}
\]
G: Elastic modulus of steel.

\( L_{ky} \): Effective buckling length due to bending, with respect to the Y axis.
\( L_{kz} \): Effective buckling length due to bending, with respect to the Z axis.
\( L_{kt} \): Effective buckling length due to torsion.
\( i_0 \): Polar radius of gyration of the gross section, with respect to the centre of torsion.

\[
i_0 = \left( i_y^2 + i_z^2 + y_0^2 + z_0^2 \right)^{0.5}
\]

Where:
\( i_y \), \( i_z \): Radii of gyration of the gross section, with respect to the main axes of inertia Y and Z.
\( y_0 \), \( z_0 \): Coordinates of the torsional centre in the direction of the main axes Y and Z, respectively, relative to the centre of gravity of the section.

Crushing of the web induced by the compressed flange (COPE criteria, based on: Eurocode 3 EN 1993-1-5: 2006, Article 8)

The following criteria must be satisfied:

\[
\frac{h_w}{t_w} \leq \frac{E}{f_{yf}} \sqrt{\frac{A_{wr}}{A_{fc,ef}}} \leq 17.88 \leq 164.04 \checkmark
\]

Where:
\( h_w \): Height of the web.
\( t_w \): Web thickness.
\( A_w \): Area of the web.
\( A_{fc,ef} \): Reduced area of the compressed flange.
\( k \): Coefficient which depends on the class of the section.
\( E \): Modulus of Elasticity.
\( f_{yf} \): Steel elastic limit of the compressed flange.

\[
f_{yf} = f_y
\]

Resistance to axial tension (CTE DB SE-A, Article 6.2.3)

The check does not proceed, as there is no tensile axial force.
**Compression resistance** (CTE DB SE-A, Article 6.2.5)

The following criteria must be satisfied:

\[
\eta = \frac{N_{c,Ed}}{N_{c,Rd}} \leq 1
\]

\[
\eta = \frac{N_{c,Ed}}{N_{c,Rd}} \leq 1
\]

\[
\eta = \frac{N_{c,Ed}}{N_{c,Rd}} \leq 1
\]

\[
\eta : 0.138
\]

\[
\eta : 0.210
\]

The worst case design force occurs at node N16, for load combination 1.35·SW+1.5·Q1+0.9·V4+0.75·N1.

\[N_{c,Ed} : \text{Worst case design compressive axial force.} \quad N_{c,Ed} : 235.97 \text{ kN}\]

The normal design compression force \(N_{c,Rd}\) should be taken as:

\[N_{c,Rd} = A \cdot f_{yd}\]

Where:

**Class**: Section class, depending on its deformation capacity and development of plastic resistance of the compressed elements of a section.

\[
A : \text{Area of the gross section for class 1, 2 and 3 sections.} \quad A : 65.30 \text{ cm}^2
\]

\[
f_{yd} : \text{Steel design strength.} \quad f_{yd} : 261.90 \text{ MPa}
\]

Where:

\[
f_{y} : \text{Yield strength. (CTE DB SE-A, Table 4.1)} \quad f_{y} : 275.00 \text{ MPa}
\]

\[
\gamma_{M0} : \text{Partial safety factor of the material.} \quad \gamma_{M0} : 1.05
\]

**Buckling resistance**: (CTE DB SE-A, Article 6.3.2)

The design buckling resistance \(N_{b,Rd}\) of a compressed bar is given by:

\[N_{b,Rd} = \chi \cdot A \cdot f_{yd}\]

Where:

\[
A : \text{Area of the gross section for class 1, 2 and 3 sections.} \quad A : 65.30 \text{ cm}^2
\]

\[
f_{yd} : \text{Steel design strength.} \quad f_{yd} : 261.90 \text{ MPa}
\]

Where:

\[
f_{y} : \text{Yield strength. (CTE DB SE-A, Table 4.1)} \quad f_{y} : 275.00 \text{ MPa}
\]

\[
\gamma_{M1} : \text{Partial safety factor of the material.} \quad \gamma_{M1} : 1.05
\]

\[
\chi : \text{Reduction coefficient due to buckling.} \quad \chi : 0.89
\]
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\[ \chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \left(\overline{\lambda}\right)^2}} \leq 1 \]

Where:
\[ \Phi = 0.5 \left[ 1 + \alpha \cdot (\overline{\lambda} - 0.2) + (\overline{\lambda})^2 \right] \]

\( \alpha \): Elastic imperfection coefficient.
\( \overline{\lambda} \): Reduced slenderness.

\[ \overline{\lambda} = \frac{A \cdot f_{yd}}{N_{cr}} \]

\( N_{cr} \): Critical elastic buckling axial force, obtained from the smallest of the following values:
- \( N_{cr,y} \): Critical elastic buckling axial force with respect to the Y axis.
- \( N_{cr,z} \): Critical elastic buckling axial force with respect to the Z axis.
- \( N_{cr,T} \): Critical elastic buckling axial force due to torsion.

\[ \eta = \frac{M_{Ed}}{M_{c,Rd}} \leq 1 \]

\( \eta : 0.354 \) ✔

\( M_{Ed^+} \): Worst case design bending moment.
\( M_{Ed^-} \): Worst case design bending moment.

For positive bending:
\( M_{Ed^+} \): 0.00 kN·m

For negative bending:
The worst case design force occurs at a point situated at a distance of 3.065 m from node N16, for load combination 1.35·SW+1.5·Q1+0.9·V2.

\( M_{Ed^-} \): 44.67 kN·m

The design bending moment resistance \( M_{c,Rd} \) is given by:

\[ M_{c,Rd} = W_{pl,y} \cdot f_{yd} \]

\( M_{c,Rd} : 126.08 \) kN·m

Where:
- \( \text{Class} \): Section class, depending on its deformation capacity and development of plastic resistance of the flat elements of a section submitted to simple bending.
- \( W_{pl,y} \): Plastic strength modulus corresponding to the fibre with greatest tension, for class 1 and 2 sections.
- \( f_{yd} \): Steel design strength.

\( W_{pl,y} : 481.40 \) cm³

\( f_{yd} : 261.90 \) MPa

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\[ f_{yd} = \frac{f_y}{\gamma_{Mo}} \]

Where:
- \( f_y \): Yield strength. (CTE DB SE-A, Table 4.1)
- \( \gamma_{Mo} \): Partial safety factor of the material.

\[ f_y : 275.00 \text{ MPa} \quad \gamma_{Mo} : 1.05 \]

**Lateral buckling resistance:** (CTE DB SE-A, Article 6.3.3.2)
Does not proceed, as the lateral buckling lengths are null.

**Z - Axis bending resistance** (CTE DB SE-A, Article 6.2.6)
The following criteria must be satisfied:

\[ \eta = \frac{M_{Ed}}{M_{c,Rd}} \leq 1 \]

\[ \eta : 0.390 \checkmark \]

For positive bending:
The worst case design force occurs at a point situated at a distance of 3.065 m from node N16, for load combination 1.35·SW+1.05·Q1+1.5·V3+0.75·N1.

\[ M_{Ed}^+ : \text{Worst case design bending moment.} \]

\[ M_{Ed}^+ : 23.59 \text{ kN·m} \]

For negative bending:
The worst case design force occurs at a point situated at a distance of 3.065 m from node N16, for load combination 0.8·SW+1.5·V4.

\[ M_{Ed}^- : \text{Worst case design bending moment.} \]

\[ M_{Ed}^- : 8.52 \text{ kN·m} \]

The design bending moment resistance \( M_{c,Rd} \) is given by:

\[ M_{c,Rd} = W_{pl,z} \cdot f_{yd} \]

\[ M_{c,Rd} : 60.50 \text{ kN·m} \]

Where:
- **Class**: Section class, depending on its deformation capacity and development of plastic resistance of the flat elements of a section submitted to simple bending.

\[ W_{pl,z} : 231.00 \text{ cm}^3 \]

\[ f_{yd} : \text{Steel design strength.} \]

\[ f_{yd} = \frac{f_y}{\gamma_{Mo}} \]

Where:
- \( f_y \): Yield strength. (CTE DB SE-A, Table 4.1)
- \( \gamma_{Mo} \): Partial safety factor of the material.

\[ f_y : 275.00 \text{ MPa} \quad \gamma_{Mo} : 1.05 \]
**Resistance to shear in the Z direction** (CTE DB SE-A, Article 6.2.4)

The following criteria must be satisfied:

\[ \eta = \frac{V_{Ed}}{V_{c,Rd}} \leq 1 \]

\[ \eta : 0.085 \checkmark \]

The worst case design force occurs for load combination 1.35·SW+1.05·Q1+1.5·V2.

- **V_{Ed}**: Worst case design shear force.
  - \( V_{Ed} : 26.13 \text{ kN} \)

The shear resistance \( V_{c,Rd} \) is given by:

\[ V_{c,Rd} = A_v \cdot \frac{f_{yd}}{\sqrt{3}} \]

\[ \begin{align*}
V_{c,Rd} : & 306.81 \text{ kN} \\
A_v : & 20.29 \text{ cm}^2 \\
f_{yd} : & 261.90 \text{ MPa}
\end{align*} \]

Where:
- **A_v**: Transverse shear area.
  - \( A_v = h \cdot t_w \)
  - \( h : \) Depth of the section.
  - \( t_w : \) Web thickness.
  - \( h : 180.00 \text{ mm} \)
  - \( t_w : 8.50 \text{ mm} \)

- **f_{yd}**: Steel design strength.
  - \( f_{yd} = f_y / \gamma_{M0} \)
  - \( f_y : \) Yield strength. (CTE DB SE-A, Table 4.1)
  - \( f_y : 275.00 \text{ MPa} \)
  - \( \gamma_{M0} : 1.05 \)

**Shear buckling of the web**: (CTE DB SE-A, Article 6.3.3.4)

Even though transverse stiffeners have not been provided, it is not necessary to check the buckling resistance of the web, as the following is verified:

\[ \frac{d}{t_w} < 70 \cdot \varepsilon \]

\[ \begin{align*}
14.35 \times \ & 64.71 \checkmark
\end{align*} \]

Where:
- **\( \lambda_w \)**: Slenderness of the web.
  - \( \lambda_w = \frac{d}{t_w} \)
  - \( \lambda_w : 14.35 \)

- **\( \lambda_{max} \)**: Maximum slenderness.
  - \( \lambda_{max} = 70 \cdot \varepsilon \)
  - \( \lambda_{max} : 64.71 \)

- **\( \varepsilon \)**: Reduction factor.
  - \( \varepsilon = \frac{f_{y,ref}}{f_y} \)
  - \( \varepsilon : 0.92 \)

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Where:

- $f_{\text{ref}}$: Reference elastic limit. $f_{\text{ref}} : 235.00$ MPa
- $f_y$: Yield strength. (CTE DB SE-A, Table 4.1) $f_y : 275.00$ MPa

**Resistance to shear in the Y direction** (CTE DB SE-A, Article 6.2.4)

The following criteria must be satisfied:

$$\eta = \frac{V_{Ed}}{V_{c,Rd}} \leq 1$$

$\eta : 0.018$

The worst case design force occurs for load combination

$1.35 \cdot \text{SW} + 1.05 \cdot Q1 + 1.5 \cdot V3 + 0.75 \cdot N1.$

$V_{Ed}$: Worst case design shear force. $V_{Ed} : 14.61$ kN

The shear resistance $V_{c,Rd}$ is given by:

$$V_{c,Rd} = A_v \cdot \frac{f_{yd}}{\sqrt{3}}$$

$V_{c,Rd} : 792.04$ kN

Where:

- $A_v$: Transverse shear area. $A_v : 52.38$ cm²

Where:

- $A$: Area of the gross section. $A : 65.30$ cm²
- $d$: Height of the web. $d : 152.00$ mm
- $t_w$: Web thickness. $t_w : 8.50$ mm

- $f_{yd}$: Steel design strength. $f_{yd} : 261.90$ MPa

Where:

- $f_y$: Yield strength. (CTE DB SE-A, Table 4.1) $f_y : 275.00$ MPa
- $\gamma_{M0}$: Partial safety factor of the material. $\gamma_{M0} : 1.05$

**Combined bending moment Y and shear force Z resistance** (CTE DB SE-A, Article 6.2.8)

It is not necessary to reduce the design bending resistance, as the worst case shear force $V_{Ed}$ is not greater than 50% of the design shear resistance $V_{c,Rd}$. 

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The worst case design forces occur for load combination 1.35·SW+1.05·Q1+1.5·V2.

\[ V_{Ed} \leq \frac{V_{c,Rd}}{2} \]

26.13 kN \leq 153.40 kN

\[ V_{Ed} \] Worst case design shear force.

\[ V_{c,Rd} \] Design resistant shear force.

Combined bending moment Z and shear force Y resistance (CTE DB SE-A, Article 6.2.8)

It is not necessary to reduce the design bending resistance, as the worst case shear force \( V_{Ed} \) is not greater than 50% of the design shear resistance \( V_{c,Rd} \).

\[ V_{Ed} \leq \frac{V_{c,Rd}}{2} \]

14.61 kN \leq 396.02 kN

\[ V_{Ed} \] Worst case design shear force.

\[ V_{c,Rd} \] Design resistant shear force.

Combined bending and axial resistance (CTE DB SE-A, Article 6.2.8)

The following criteria must be satisfied:

\[ \eta = \frac{N_{Ed,y}}{N_{pl,Rd,y}} + \frac{M_{Ed,y}}{M_{pl,Rd,y}} + \frac{M_{Ed,z}}{M_{pl,Rd,z}} \leq 1 \]

\[ \eta = 0.737 \]

\[ \eta = \frac{N_{c,Ed}}{f_{yd}} + \frac{M_{c,Ed,y}}{W_{pl,y}} + \frac{M_{c,Ed,z}}{W_{pl,z}} \leq 1 \]

\[ \eta = 0.633 \]

\[ \eta = \frac{N_{c,Ed}}{f_{yd}} + \frac{N_{c,Ed}}{f_{yd}} + \frac{M_{c,Ed,y}}{W_{pl,y}} + \frac{M_{c,Ed,z}}{W_{pl,z}} \leq 1 \]

\[ \eta = 0.744 \]

The worst case design forces occur at a point situated at a distance of 3.065 m from node N16, for load combination 1.35·SW+1.05·Q1+1.5·V3+0.75·N1.
Where:

\[ N_{c,Ed} \]: Worst case design compressive axial force.
\[ M_{y,Ed} \], \[ M_{z,Ed} \]: Worst case bending moments, in accordance with the Y and Z axes, respectively.
\[ Class \]: Section class, according to its deformation capacity and plastic resistance development of its flat elements, for axial load and simple bending.
\[ N_{pl,Rd} \]: Compressive resistance of the gross section.
\[ M_{pl,Rd,y} \], \[ M_{pl,Rd,z} \]: Bending resistance of the gross section in plastic conditions, with respect to the Y and Z axes, respectively.

**Buckling resistance**: (CTE DB SE-A, Article 6.3.4.2)

\[ A \]: Area of the gross section.
\[ W_{pl,y} \], \[ W_{pl,z} \]: Plastic resistant modules corresponding to the compressed fibre, about the Y and Z, respectively.
\[ f_{yd} \]: Steel design strength.
\[ f_y \]: Yield strength. (CTE DB SE-A, Table 4.1)
\[ \gamma_{M1} \]: Partial safety factor of the material.

\[ k_y \], \[ k_z \]: Interaction coefficients.

\[ k_y = 1 + (\bar{\lambda}_y - 0.2) \cdot \frac{N_{c,Ed}}{N_{c,Rd}} \]
\[ k_z = 1 + (2 \cdot \bar{\lambda}_z - 0.6) \cdot \frac{N_{c,Ed}}{N_{c,Rd}} \]

\[ \chi_y \], \[ \chi_z \]: Buckling reduction coefficients, about the Y and Z axes, respectively.
\[ \bar{\lambda}_y \], \[ \bar{\lambda}_z \]: Reduced slenderness with values no greater than 1.00, with respect to the Y and Z axes, respectively.
\[ \alpha_y \], \[ \alpha_z \]: Factors that depend on the class of the section.

**Combined bending, axial and shear resistance**: (CTE DB SE-A, Article 6.2.8)

It is not necessary to reduce the design bending and axial force resistance, as the buckling effect can be ignored due to shear. Additionally, the worst case design shear force \( V_{Ed} \) is less than or equal to 50% of the design shear resistance \( V_{c,Rd} \).
The worst case design forces occur for load combination 1.35·SW+1.05·Q1+1.5·V2.

\[ V_{Ed,z} \leq \frac{V_{c,Rd,z}}{2} \]

Where:
- \( V_{Ed,z} \): Worst case design shear force.
- \( V_{c,Rd,z} \): Design resistant shear force.

\[ 26.13 \text{ kN} \leq 153.24 \text{ kN} \]

\( V_{Ed,z} : \quad 26.13 \text{ kN} \)
\( V_{c,Rd,z} : \quad 306.49 \text{ kN} \)

**Torsional resistance** (CTE DB SE-A, Article 6.2.7)

The following criteria must be satisfied:

\[ \eta = \frac{M_{T,Ed}}{M_{T,Rd}} \leq 1 \]

\( \eta : \quad 0.003 \)

The worst case design force occurs for load combination 1.35·SW+1.05·Q1+1.5·V2.

\( M_{T,Ed} : \quad 0.01 \text{ kN·m} \)

The design torsional moment resistance \( M_{T,Rd} \) is given by:

\[ M_{T,Rd} = \frac{1}{\sqrt{3}} \cdot W_T \cdot f_{yd} \]

Where:
- \( W_T \): Torsion resistance module.
- \( f_{yd} \): Steel design strength.
  \[ f_{yd} = \frac{f_y}{\gamma_{M0}} \]

Where:
- \( f_y \): Yield strength. (CTE DB SE-A, Table 4.1)
  \[ f_y : \quad 275.00 \text{ MPa} \]
- \( \gamma_{M0} \): Partial safety factor of the material.
  \[ \gamma_{M0} : \quad 1.05 \]

\( W_T : \quad 30.11 \text{ cm}^3 \)

\( f_{yd} : \quad 261.90 \text{ MPa} \)

\( M_{T,Rd} : \quad 4.55 \text{ kN·m} \)

**Combined Z shear and torsional resistance** (CTE DB SE-A, Article 6.2.8)

The following criteria must be satisfied:

\[ \eta = \frac{V_{Ed}}{V_{pl,T,Rd}} \leq 1 \]

\( \eta : \quad 0.085 \)

The worst case design forces occur for load combination 1.35·SW+1.05·Q1+1.5·V2.

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\[ V_{Ed}: \text{Worst case design shear force.} \]

\[ M_{T,Ed}: \text{Worst case design torsional moment.} \]

The reduced design resistant shear force \( V_{pl,T,Rd} \) is given by:

\[
V_{pl,T,Rd} = \frac{1 - \frac{\tau_{T,Ed}}{1.25 \cdot f_{yd}/\sqrt{3}}}{V_{pl,Rd}}
\]

Where:

\[ V_{pl,Rd}: \text{Design resistant shear force.} \]

\[ \tau_{T,Ed}: \text{Tangential stresses due to torsion.} \]

\[
\tau_{T,Ed} = \frac{M_{T,Ed}}{W_t}
\]

Where:

\[ W_t: \text{Torsion resistance module.} \]

\[ f_{yd}: \text{Steel design strength.} \]

\[ f_{yd} = \frac{f_y}{\gamma_{M0}} \]

Where:

\[ f_y: \text{Yield strength. (CTE DB SE-A, Table 4.1)} \]

\[ \gamma_{M0}: \text{Partial safety factor of the material.} \]

**Combined Y shear and torsional resistance (CTE DB SE-A, Article 6.2.8)**

The following criteria must be satisfied:

\[
\eta = \frac{V_{Ed}}{V_{pl,T,Rd}} \leq 1
\]

\[ \eta : 0.018 \checkmark \]

The worst case design forces occur for load combination 1.35·SW+1.05·Q1+1.5·V3+0.75·N1.

\[ V_{Ed}: \text{Worst case design shear force.} \]

\[ M_{T,Ed}: \text{Worst case design torsional moment.} \]

The reduced design resistant shear force \( V_{pl,T,Rd} \) is given by:

\[
V_{pl,T,Rd} = \frac{1 - \frac{\tau_{T,Ed}}{1.25 \cdot f_{yd}/\sqrt{3}}}{V_{pl,Rd}}
\]

Where:

\[ V_{pl,Rd}: \text{Design resistant shear force.} \]

\[ \tau_{T,Ed}: \text{Tangential stresses due to torsion.} \]

\[
\tau_{T,Ed} = \frac{M_{T,Ed}}{W_t}
\]

Where:

\[ W_t: \text{Torsion resistance module.} \]

\[ f_{yd}: \text{Steel design strength.} \]

\[ f_{yd} = \frac{261.90}{275.00} \]

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\[ f_{yd} = \frac{f_y}{\gamma_{M0}} \]

Where:
- \( f_y \): Yield strength. (CTE DB SE-A, Table 4.1)
- \( f_y \): 275.00 MPa
- \( \gamma_{M0} \): Partial safety factor of the material.
- \( \gamma_{M0} \): 1.05

### 2.4 Section 4

#### Section: IPE 270
Material: Steel (S275)

<table>
<thead>
<tr>
<th>Nodes</th>
<th>Initial</th>
<th>Length (m)</th>
<th>Mechanical characteristics</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Area (cm²)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>N18</td>
<td>N20</td>
<td>1.00</td>
<td>45.90</td>
</tr>
</tbody>
</table>

Notes:
- (1) Inertia with respect to the indicated axis
- (2) Uniform torsional moment of inertia

#### Slenderness limit (CTE DB SE-A, Articles 6.3.1 and 6.3.2.1 - Table 6.3)

The reduced slenderness \( \bar{\lambda} \) of the bars in compression must be less than 2.0.

\[ \bar{\lambda} : 0.19 \]
\[
\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}}
\]

Where:

**Class**: Section class, depending on its deformation capacity and development of plastic resistance of the compressed elements of a section.

**Class**: 2

**A**: Area of the gross section for class 1, 2 and 3 sections.

**A**: 45.90 cm²

**f_y**: Yield strength. (CTE DB SE-A, Table 4.1)

**f_y**: 275.00 MPa

**N_{cr}**: Critical elastic buckling axial force.

**N_{cr}**: 34819.96 kN

The critical elastic buckling axial force \( N_{cr} \) is the smallest of the following values a), b) and c):

a) Critical elastic buckling axial force with respect to the Y axis.

\[
N_{cr,Y} = \frac{\pi^2 \cdot E \cdot I_y}{L_{ky}^2}
\]

b) Critical elastic buckling axial force with respect to the Z axis.

\[
N_{cr,Z} = \frac{\pi^2 \cdot E \cdot I_z}{L_{kz}^2}
\]

c) Critical elastic buckling axial force due to torsion.

\[
N_{cr,T} = \frac{1}{i_0} \left[ G \left( L_{kt} + \frac{\pi^2 \cdot E \cdot I_w}{L_{kt}^2} \right) \right]
\]

Where:

**I_y**: Moment of inertia of the gross section, with respect to the Y-axis.

**I_y**: 5790.00 cm⁴

**I_z**: Moment of inertia of the gross section, with respect to the Z-axis.

**I_z**: 420.00 cm⁴

**I_t**: Uniform torsional moment of inertia.

**I_t**: 15.90 cm⁴

**I_w**: Section warping constant.

**I_w**: 70600.00 cm⁶

**E**: Modulus of Elasticity.

**E**: 210000 MPa

**G**: Elastic modulus of steel.

**G**: 81000 MPa

**L_{ky}**: Effective buckling length due to bending, with respect to the Y axis.

**L_{ky}**: 1.000 m

**L_{kz}**: Effective buckling length due to bending, with respect to the Z axis.

**L_{kz}**: 0.500 m

**L_{kt}**: Effective buckling length due to torsion.

**L_{kt}**: 0.000 m

**i_0**: Polar radius of gyration of the gross section, with respect to the centre of torsion.

**i_0**: 11.63 cm

Where:

\[i_0 = \left( \left( \frac{i_y^2 + i_z^2 + y_0^2 + z_0^2}{i_y^4 + i_z^4} \right)^{0.5} \right)\]

Where:

**i_y**, **i_z**: Radii of gyration of the gross section, with respect to the main axes of inertia Y and Z.

**i_y**: 11.23 cm

**i_z**: 3.02 cm

**y_0**: 0.00 mm
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\( y_0, z_0 \): Coordinates of the torsional centre in the direction of the main axes Y and Z, respectively, relative to the centre of gravity of the section.

\( z_0 : 0.00 \) mm

**Crushing of the web induced by the compressed flange** (CYPE criteria, based on: Eurocode 3 EN 1993-1-5: 2006, Article 8)

The following criteria must be satisfied:

\[
\frac{h_w}{t_w} \leq \frac{E}{f_{Yf}} \sqrt{\frac{A_w}{A_{fc,ef}}} \leq 37.82 \leq 250.57
\]

Where:
- \( h_w \): Height of the web.
- \( t_w \): Web thickness.
- \( A_w \): Area of the web.
- \( A_{fc,ef} \): Reduced area of the compressed flange.
- \( k \): Coefficient which depends on the class of the section.
- \( E \): Modulus of Elasticity.
- \( f_{Yf} \): Steel elastic limit of the compressed flange.

Where:
- \( h_w : 249.60 \) mm
- \( t_w : 6.60 \) mm
- \( A_w : 16.47 \) cm\(^2\)
- \( A_{fc,ef} : 13.77 \) cm\(^2\)
- \( k : 0.30 \)
- \( E : 210000 \) MPa
- \( f_{Yf} : 275.00 \) MPa

**Resistance to axial tension** (CTE DB SE-A, Article 6.2.3)

The following criteria must be satisfied:

\[
\eta = \frac{N_{t,Ed}}{N_{t,Rd}} \leq 1
\]

\( \eta : 0.005 \) ✔

The worst case design force occurs for load combination 0.8·SW+1.5·V1+0.75·N1.

\( N_{t,Ed} \): Worst case design axial tensile force.

\( N_{t,Ed} : 6.32 \) kN

The design tensile resistance \( N_{t,Rd} \) is given by:

\[
N_{t,Rd} = A \cdot f_{Yd}
\]

Where:
- \( A \): Gross transverse section of the bar.
- \( f_{Yd} \): Steel design strength.

\( A : 45.90 \) cm\(^2\)

\( f_{Yd} : 261.90 \) MPa
\[ f_{yd} = f_y / \gamma_{M0} \]

Where:
- \( f_y \): Yield strength. (CTE DB SE-A, Table 4.1)
- \( \gamma_{M0} \): Partial safety factor of the material.

\[ f_y : 275.00 \text{ MPa} \quad \gamma_{M0} : 1.05 \]

**Compression resistance** (CTE DB SE-A, Article 6.2.5)

The following criteria must be satisfied:

\[ \eta = \frac{N_{c,Ed}}{N_{c,Rd}} \leq 1 \]

\[ \eta : 0.010 \quad \checkmark \]

The worst case design force occurs for load combination
1.35·SW+1.05·Q1+1.5·V2.

- \( N_{c,Ed} \): Worst case design compressive axial force.
- \( N_{c,Ed} : \) 11.92 kN

The normal design compression force \( N_{c,Rd} \) should be taken as:

\[ N_{c,Rd} = A \cdot f_{yd} \]

\[ N_{c,Rd} : \) 1202.14 kN

Where:
- **Class**: Section class, depending on its deformation capacity and development of plastic resistance of the compressed elements of a section.
- **A**: Area of the gross section for class 1, 2 and 3 sections.
- \( f_{yd} \): Steel design strength.
- \( f_{yd} = f_y / \gamma_{M0} \)

Where:
- \( f_y \): Yield strength. (CTE DB SE-A, Table 4.1)
- \( \gamma_{M0} \): Partial safety factor of the material.

\[ f_y : 275.00 \text{ MPa} \quad \gamma_{M0} : 1.05 \]

**Buckling resistance**: (CTE DB SE-A, Article 6.3.2)

For slenderness \( \overline{\lambda} \leq 0.2 \) the buckling check can be omitted, and only check the resistance of the transverse section.

\[ \overline{\lambda} : 0.19 \]

\[ \overline{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}} \]

Where:
- **A**: Area of the gross section for class 1, 2 and 3 sections.
- \( A : \) 45.90 cm²


\( f_y \): Yield strength. (CTE DB SE-A, Table 4.1)  
\( N_{cr} \): Critical elastic buckling axial force, obtained from the smallest of the following values:  
\( N_{cr,Y} \): Critical elastic buckling axial force with respect to the Y axis.  
\( N_{cr,Z} \): Critical elastic buckling axial force with respect to the Z axis.  
\( N_{cr,T} \): Critical elastic buckling axial force due to torsion.  
\( f_y \) : 275.00 MPa  
\( N_{cr} \) : 34819.96 kN  
\( N_{cr,Y} \) : 120004.52 kN  
\( N_{cr,Z} \) : 34819.96 kN  
\( N_{cr,T} \) : \( \infty \) 

**Y - Axis bending resistance** (CTE DB SE-A, Article 6.2.6)  
The following criteria must be satisfied:

\[
\eta = \frac{M_{Ed}}{M_{c,Rd}} \leq 1
\]

\( \eta \) : 0.631 \( \checkmark \)

For positive bending:
\( M_{Ed}^{+} \): Worst case design bending moment.  
\( M_{Ed}^{+} \) : 0.00 kN⋅m

For negative bending:

The worst case design force occurs at a point situated at a distance of 0.910 m from node N18, for load combination 1.35·SW+1.5·Q1+0.9·V2+0.75·N1.  
\( M_{Ed}^{-} \): Worst case design bending moment.  
\( M_{Ed}^{-} \) : 79.96 kN⋅m

The design bending moment resistance \( M_{c,Rd} \) is given by:

\[
M_{c,Rd} = W_{pl,Y} \cdot f_{yd}
\]

\( M_{c,Rd} \) : 126.76 kN⋅m  

Where:

**Class**: Section class, depending on its deformation capacity and development of plastic resistance of the flat elements of a section submitted to simple bending.  
\( W_{pl,Y} \): Plastic strength modulus corresponding to the fibre with greatest tension, for class 1 and 2 sections.  
\( f_{yd} \): Steel design strength.  
\( f_{yd} = f_y / \gamma_{M0} \)

Where:

\( f_y \): Yield strength. (CTE DB SE-A, Table 4.1)  
\( \gamma_{M0} \): Partial safety factor of the material.

\( f_y \) : 275.00 MPa  
\( \gamma_{M0} \) : 1.05

**Lateral buckling resistance**: (CTE DB SE-A, Article 6.3.3.2)  
Does not proceed, as the lateral buckling lengths are null.
**Z - Axis bending resistance** (CTE DB SE-A, Article 6.2.6)

The following criteria must be satisfied:

\[ \eta = \frac{M_{Ed}}{M_{c,Rd}} \leq 1 \]

\[ \eta : 0.080 \checkmark \]

For positive bending:

The worst case design force occurs at a point situated at a distance of 0.910 m from node N18, for load combination 1.35·SW+1.05·Q1+1.5·V3.

\[ M_{Ed}^+ : \text{Worst case design bending moment.} \]

For negative bending:

The worst case design force occurs at a point situated at a distance of 0.910 m from node N18, for load combination 0.8·SW+1.5·V4+0.75·N1.

\[ M_{Ed}^- : \text{Worst case design bending moment.} \]

The design bending moment resistance \( M_{c,Rd} \) is given by:

\[ M_{c,Rd} = W_{pl,z} \cdot f_{yd} \]

\[ M_{c,Rd} = 25.40 \text{ kN·m} \]

Where:

- **Class**: Section class, depending on its deformation capacity and development of plastic resistance of the flat elements of a section submitted to simple bending.
  
  **Class**: 1

- **W_{pl,z}**: Plastic strength modulus corresponding to the fibre with greatest tension, for class 1 and 2 sections.
  
  \( W_{pl,z} = 97.00 \text{ cm}^3 \)

- **f_{yd}**: Steel design strength.
  
  \( f_{yd} = 261.90 \text{ MPa} \)

Where:

- **f_y**: Yield strength. (CTE DB SE-A, Table 4.1)
  
  \( f_y = 275.00 \text{ MPa} \)

- **\( \gamma_{M0} \)**: Partial safety factor of the material.
  
  \( \gamma_{M0} = 1.05 \)

---

**Resistance to shear in the Z direction** (CTE DB SE-A, Article 6.2.4)

The following criteria must be satisfied:

\[ \eta = \frac{V_{Ed}}{V_{c,Rd}} \leq 1 \]

\[ \eta : 0.214 \checkmark \]

The worst case design force occurs at a point situated at a distance of 0.910 m from node N18, for load combination 1.35·SW+1.5·Q1+0.9·V2+0.75·N1.
\( V_{Ed} \): Worst case design shear force.

The shear resistance \( V_{c,Rd} \) is given by:

\[
V_{c,Rd} = A_v \cdot \frac{f_{yd}}{\sqrt{3}}
\]

Where:
- \( A_v \): Transverse shear area.
  \( A_v = h \cdot t_w \)
- \( h \): Depth of the section.
- \( t_w \): Web thickness.
- \( f_{yd} \): Steel design strength.
  \( f_{yd} = f_y / \gamma_{M0} \)
- \( f_y \): Yield strength. (CTE DB SE-A, Table 4.1)
- \( \gamma_{M0} \): Partial safety factor of the material.

\( V_{Ed} : 71.57 \) kN

\( V_{c,Rd} : 334.07 \) kN

Where:
- \( A_v : 22.09 \) cm²
- \( h : 270.00 \) mm
- \( t_w : 6.60 \) mm
- \( f_{yd} : 261.90 \) MPa
- \( f_y : 275.00 \) MPa
- \( \gamma_{M0} : 1.05 \)

**Shear buckling of the web:** (CTE DB SE-A, Article 6.3.3.4)

Even though transverse stiffeners have not been provided, it is not necessary to check the buckling resistance of the web, as the following is verified:

\[
\frac{d}{t_w} < 70 \cdot \varepsilon \quad 33.27 < 64.71 \checkmark
\]

Where:
- \( \lambda_w \): Slenderness of the web.
  \( \lambda_w = \frac{d}{t_w} \)
- \( \lambda_{max} \): Maximum slenderness.
  \( \lambda_{max} = 70 \cdot \varepsilon \)
- \( \varepsilon \): Reduction factor.
  \( \varepsilon = \frac{f_{ref}}{f_y} \)
- \( f_{ref} \): Reference elastic limit.
- \( f_y \): Yield strength. (CTE DB SE-A, Table 4.1)

Where:
- \( f_{ref} : 235.00 \) MPa
- \( f_y : 275.00 \) MPa

**Resistance to shear in the Y direction** (CTE DB SE-A, Article 6.2.4)

The following criteria must be satisfied:
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\[ \eta = \frac{V_{Ed}}{V_{c,Rd}} \leq 1 \]

The worst case design force occurs for load combination 1.35\cdot SW+1.05\cdot Q1+1.5\cdot V3.

\[ V_{Ed} \]: Worst case design shear force.

The shear resistance \( V_{c,Rd} \) is given by:

\[ V_{c,Rd} = A_v \cdot \frac{f_{yd}}{\sqrt{3}} \]

Where:

\[ A_v \]: Transverse shear area.
\[ A_v = A - d \cdot t_w \]

Where:

\[ A \]: Area of the gross section.
\[ d \]: Height of the web.
\[ t_w \]: Web thickness.

\[ f_{yd} \]: Steel design strength.
\[ f_{yd} = \frac{f_y}{\gamma_{M0}} \]

Where:

\[ f_y \]: Yield strength. (CTE DB SE-A, Table 4.1)
\[ \gamma_{M0} \]: Partial safety factor of the material.

\[ f_y = 275.00 \text{ MPa} \]
\[ \gamma_{M0} = 1.05 \]
\[ A_v = 29.43 \text{ cm}^2 \]
\[ A = 45.90 \text{ cm}^2 \]
\[ d = 249.60 \text{ mm} \]
\[ t_w = 6.60 \text{ mm} \]
\[ f_{yd} = 261.90 \text{ MPa} \]

Combined bending moment Y and shear force Z resistance (CTE DB SE-A, Article 6.2.8)

It is not necessary to reduce the design bending resistance, as the worst case shear force \( V_{Ed} \) is not greater than 50\% of the design shear resistance \( V_{c,Rd} \).

\[ V_{Ed} \leq \frac{V_{c,Rd}}{2} \]

\[ 71.14 \text{ kN} \leq 167.04 \text{ kN} \]

The worst case design forces occur for load combination 1.35\cdot SW+1.5\cdot Q1+0.9\cdot V2+0.75\cdot N1.

\[ V_{Ed} \]: Worst case design shear force.

\[ V_{Ed} : 71.14 \text{ kN} \]

\[ V_{c,Rd} \]: Design resistant shear force.

\[ V_{c,Rd} : 334.07 \text{ kN} \]

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**Combined bending moment Z and shear force Y resistance** (CTE DB SE-A, Article 6.2.8)

It is not necessary to reduce the design bending resistance, as the worst case shear force $V_{Ed}$ is not greater than 50% of the design shear resistance $V_{C,Rd}$.

$$V_{Ed} \leq \frac{V_{C,Rd}}{2} \quad 2.70 \text{ kN} \leq 222.48 \text{ kN} \quad \checkmark$$

The worst case design forces occur for load combination $1.35 \cdot \text{SW} + 1.05 \cdot \text{Q1} + 1.5 \cdot \text{V3}$.

$V_{Ed}$: Worst case design shear force.

$V_{C,Rd}$: Design resistant shear force.

$$V_{Ed} : 2.70 \quad \text{kN} \quad V_{C,Rd} : 444.96 \quad \text{kN}$$

**Combined bending and axial resistance** (CTE DB SE-A, Article 6.2.8)

The following criteria must be satisfied:

$$\eta = \frac{N_{c,Ed}}{N_{pl,Rd}} + \frac{M_{y,Ed}}{M_{pl,Rd,y}} + \frac{M_{z,Ed}}{M_{pl,Rd,z}} \leq 1 \quad \eta : 0.680 \quad \checkmark$$

$$\eta = \frac{N_{c,Ed}}{\chi_y \cdot A \cdot \alpha \cdot f_{yd}} + k_y \cdot \frac{C_{n,y} \cdot M_{y,Ed}}{W_{pl,y} \cdot f_{yd}} + \alpha_z \cdot k_z \cdot \frac{C_{n,z} \cdot M_{z,Ed}}{W_{pl,z} \cdot f_{yd}} \leq 1 \quad \eta : 0.662 \quad \checkmark$$

$$\eta = \frac{N_{c,Ed}}{\chi_z \cdot A \cdot f_{yd}} + \alpha_y \cdot k_y \cdot \frac{C_{n,y} \cdot M_{y,Ed}}{W_{pl,y} \cdot f_{yd}} + \alpha_z \cdot k_z \cdot \frac{C_{n,z} \cdot M_{z,Ed}}{W_{pl,z} \cdot f_{yd}} \leq 1 \quad \eta : 0.430 \quad \checkmark$$

The worst case design forces occur at a point situated at a distance of 0.910 m from node N18, for load combination $1.35 \cdot \text{SW} + 1.5 \cdot \text{Q1} + 0.9 \cdot \text{V2}$.

Where:

$N_{c,Ed}$: Worst case design compressive axial force.

$M_{y,Ed}$, $M_{z,Ed}$: Worst case bending moments, in accordance with the Y and Z axes, respectively.

**Class**: Section class, according to its deformation capacity and plastic resistance development of its flat elements, for axial load and simple bending.

$N_{pl,Rd}$: Compressive resistance of the gross section.

$M_{pl,Rd,y}$, $M_{pl,Rd,z}$: Bending resistance of the gross section in plastic conditions, with respect to the Y and Z axes, respectively.

$N_{c,Ed} : 8.85 \quad \text{kN} \quad M_{y,Ed}^* : 79.32 \quad \text{kN-m} \quad M_{z,Ed}^* : 1.20 \quad \text{kN-m} \quad \text{Class} : 1$

$N_{pl,Rd} : 1202.14 \quad \text{kN} \quad M_{pl,Rd,y} : 126.76 \quad \text{kN-m} \quad M_{pl,Rd,z} : 25.40 \quad \text{kN-m}$

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Buckling resistance: (CTE DB SE-A, Article 6.3.4.2)

- **A**: Area of the gross section.
  \[ A = 45.90 \text{ cm}^2 \]
- **W_{pl,y}, W_{pl,z}**: Plastic resistant modules corresponding to the compressed fibre, about the Y and Z, respectively.
  \[ W_{pl,y} = 484.00 \text{ cm}^3 \]
  \[ W_{pl,z} = 97.00 \text{ cm}^3 \]
- **f_{yd}**: Steel design strength.
  \[ f_{yd} = 261.90 \text{ MPa} \]

Where:
- **f_y**: Yield strength. (CTE DB SE-A, Table 4.1)
  \[ f_y = 275.00 \text{ MPa} \]
- **γ_M1**: Partial safety factor of the material.
  \[ γ_M1 = 1.05 \]
- **k_y, k_z**: Interaction coefficients.
  \[ k_y = 1.00 \]
  \[ k_z = 1.00 \]
- **C_{m,y}, C_{m,z}**: Equivalent uniform bending moment factors.
  \[ C_{m,y} = 1.00 \]
  \[ C_{m,z} = 1.00 \]

**χ_y, χ_z**: Buckling reduction coefficients, about the Y and Z axes, respectively.
  \[ χ_y = 1.00 \]
  \[ χ_z = 1.00 \]

**λ_y, λ_z**: Reduced slenderness with values no greater than 1.00, with respect to the Y and Z axes, respectively.
  \[ λ_y = 0.10 \]
  \[ λ_z = 0.19 \]

**α_y, α_z**: Factors that depend on the class of the section.
  \[ α_y = 0.60 \]
  \[ α_z = 0.60 \]

Combined bending, axial and shear resistance (CTE DB SE-A, Article 6.2.8)

It is not necessary to reduce the design bending and axial force resistance, as the buckling effect can be ignored due to shear. Additionally, the worst case design shear force **V_{Ed}** is less than or equal to 50% of the design shear resistance **V_{c,Rd}**.

The worst case design forces occur for load combination
\[ 1.35 \cdot SW + 1.5 \cdot Q_1 + 0.9 \cdot V_2 + 0.75 \cdot N_1 \]

\[ 71.14 \text{ kN} \leq 164.31 \text{ kN} \]

Where:
- **V_{Ed,y,z}**: Worst case design shear force.
- **V_{Ed,y,z}**: Design resistant shear force.
  \[ V_{Ed,y,z} = 71.14 \text{ kN} \]
  \[ V_{c,Rd,y,z} = 328.63 \text{ kN} \]

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**Torsional resistance** (CTE DB SE-A, Article 6.2.7)

The following criteria must be satisfied:

\[ \eta = \frac{M_{T,Ed}}{M_{T,Rd}} \leq 1 \]

\( \eta : 0.043 \)

The worst case design force occurs for load combination 1.35·SW+1.5·Q1+0.9·V4+0.75·N1.

**\( M_{T,Ed} \):** Worst case design torsional moment.

\( M_{T,Ed} : 0.10 \) kN·m

The design torsional moment resistance \( M_{T,Rd} \) is given by:

\[ M_{T,Rd} = \frac{1}{\sqrt{3}} \cdot W_T \cdot f_{yd} \]

Where:

\( W_T \): Torsion resistance module.

\( f_{yd} \): Steel design strength.

\( f_{yd} = f_y / \gamma_{M0} \)

Where:

\( f_y \): Yield strength. (CTE DB SE-A, Table 4.1)

\( \gamma_{M0} \): Partial safety factor of the material.

\( W_T : 15.59 \) cm³

\( f_{yd} : 261.90 \) MPa

\( f_y : 275.00 \) MPa

\( \gamma_{M0} : 1.05 \)

**Combined Z shear and torsional resistance** (CTE DB SE-A, Article 6.2.8)

The following criteria must be satisfied:

\[ \eta = \frac{V_{Ed}}{V_{pl,T,Rd}} \leq 1 \]

\( \eta : 0.218 \)

The worst case design forces occur at a point situated at a distance of 0.910 m from node N18, for load combination 1.35·SW+1.5·Q1+0.9·V2+0.75·N1.

**\( V_{Ed} \):** Worst case design shear force.

\( V_{Ed} : 71.57 \) kN

**\( M_{T,Ed} \):** Worst case design torsional moment.

**\( V_{pl,T,Rd} \):** Design resistant shear force.

\( V_{pl,T,Rd} : 328.63 \) kN

\( V_{pl,Rd} : 334.07 \) kN

\( \tau_{T,Ed} : 6.11 \) MPa

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Combined Y shear and torsional resistance (CTE DB SE-A, Article 6.2.8)
The following criteria must be satisfied:

\[ \eta = \frac{V_{Ed}}{V_{pl,T,Rd}} \leq 1 \]

\[ \eta = 0.006 \quad \checkmark \]

The worst case design forces occur for load combination 1.35·SW+1.05·Q1+1.5·V3.

\( V_{Ed} \): Worst case design shear force.

\( V_{pl,T,Rd} \): Design resistant shear force.

\( \tau_{T,Ed} \): Tangential stresses due to torsion.

\[ \tau_{T,Ed} = \frac{M_{T,Ed}}{W_f} \]

Where:

\( M_{T,Ed} \): Worst case design torsional moment.

\( W_f \): Torsion resistance module.

\( f_{yd} \): Steel design strength.

\( f_{yd} = f_y / \gamma_{M0} \)

Where:

\( f_y \): Yield strength. (CTE DB SE-A, Table 4.1)

\( \gamma_{M0} \): Partial safety factor of the material.
2.5 Section 5

### Section: IPE 140

**Material: Steel (S275)**

#### Mechanical characteristics

<table>
<thead>
<tr>
<th>Nodes</th>
<th>Length (m)</th>
<th>Initial</th>
<th>Final</th>
</tr>
</thead>
<tbody>
<tr>
<td>N69</td>
<td>1.900</td>
<td>16.40</td>
<td>541.00</td>
</tr>
<tr>
<td>N73</td>
<td></td>
<td>44.90</td>
<td>2.45</td>
</tr>
</tbody>
</table>

Notes:

1. Inertia with respect to the indicated axis
2. Uniform torsional moment of inertia

#### Buckling

- XY plane: \( \beta \) = 0.50, \( L_K \) = 0.950
- XZ plane: \( \beta \) = 1.00, \( L_K \) = 1.900

#### Lateral buckling

- Top fl.: \( C_m \) = 1.000
- Bot. fl.: \( C_m \) = 1.000

**Notes:**

- \( C_1 \): Critical moment modification factor
- \( \eta \): Usage coefficient (%)
- D.N.P.: Not applicable

#### Checks (CTE DB SE-A)

<table>
<thead>
<tr>
<th>Bar</th>
<th>( \lambda )</th>
<th>Checks</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>N69/N73</td>
<td>( \lambda &lt; 2.0 )</td>
<td>Verified</td>
<td>Verified</td>
</tr>
</tbody>
</table>

- **Notation:**
  - \( \lambda \): Slenderness limit
  - \( L_w \): Crushing of the web induced by the compressed flange
  - \( N_t \): Resistance to axial tension
  - \( N_c \): Compression resistance
  - \( M_Y \): Y - Axis bending resistance
  - \( M_Z \): Z - Axis bending resistance
  - \( V_Z \): Resistance to shear in the Z direction
  - \( M_{YZ} \): Combined bending moment Y and shear force Z resistance
  - \( M_{YZ} \): Combined bending moment Z and shear force Y resistance
  - \( M_{YV} \): Combined bending and axial resistance
  - \( M_{VZ} \): Combined bending, axial and shear resistance
  - \( M_t \): Torsional resistance
  - \( V_Y \): Combined Z shear and torsional resistance
  - \( M_{VY} \): Combined Y shear and torsional resistance
  - \( x \): Distance to the origin of the bar
  - \( \eta \): Usage coefficient (%)
  - D.N.P.: Not applicable

**Slenderness limit (CTE DB SE-A, Articles 6.3.1 and 6.3.2.1 - Table 6.3)**

The reduced slenderness \( \bar{\lambda} \) of the bars in compression must be less than 2.0.

\[
\bar{\lambda} = \frac{A \cdot f_y}{N_{cr}}
\]

Where:

- **Class**: Section class, depending on its deformation capacity and development of plastic resistance of the compressed elements of a section.
- **A**: Area of the gross section for class 1, 2 and 3 sections.
- **f_y**: Yield strength. (CTE DB SE-A, Table 4.1)
- **N_{cr}**: Critical elastic buckling axial force.

**Class**: 1

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>16.40 cm²</td>
<td></td>
</tr>
<tr>
<td>f_y</td>
<td>275.00 MPa</td>
<td></td>
</tr>
<tr>
<td>N_{cr}</td>
<td>1031.14 kN</td>
<td></td>
</tr>
</tbody>
</table>

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The critical elastic buckling axial force $N_{cr}$ is the smallest of the following values a), b) and c):

a) Critical elastic buckling axial force with respect to the Y axis.

$$N_{cr,Y} = \frac{\pi^2 \cdot E \cdot I_y}{L_{ky}^2}$$

b) Critical elastic buckling axial force with respect to the Z axis.

$$N_{cr,Z} = \frac{\pi^2 \cdot E \cdot I_z}{L_{kz}^2}$$

c) Critical elastic buckling axial force due to torsion.

$$N_{cr,T} = \frac{1}{\iota_0} \left[ G \cdot I_t + \frac{\pi^2 \cdot E \cdot I_w}{L_{kt}^2} \right]$$

Where:

- $I_y$: Moment of inertia of the gross section, with respect to the Y-axis.
- $I_z$: Moment of inertia of the gross section, with respect to the Z-axis.
- $I_t$: Uniform torsional moment of inertia.
- $I_w$: Section warping constant.
- $E$: Modulus of Elasticity.
- $G$: Elastic modulus of steel.
- $L_{ky}$: Effective buckling length due to bending, with respect to the Y axis.
- $L_{kz}$: Effective buckling length due to bending, with respect to the Z axis.
- $L_{kt}$: Effective buckling length due to torsion.
- $\iota_0$: Polar radius of gyration of the gross section, with respect to the centre of torsion.

$$\iota_0 = \left( i_y^2 + i_z^2 + y_0^2 + z_0^2 \right)^{0.5}$$

Where:

- $i_y$, $i_z$: Radii of gyration of the gross section, with respect to the main axes of inertia Y and Z.
- $y_0$, $z_0$: Coordinates of the torsional centre in the direction of the main axes Y and Z, respectively, relative to the centre of gravity of the section.

The following criteria must be satisfied:

$$26.85 \leq 248.60 \leq 248.60$$

Crushing of the web induced by the compressed flange (CYPE criteria, based on: Eurocode 3 EN 1993-1-5: 2006, Article 8)

The following criteria must be satisfied:

$$26.85 \leq 248.60 \leq 248.60$$
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\[
\frac{h_w}{t_w} \leq k \frac{E}{f_y} \frac{A_w}{f_{c,ef}}
\]

Where:
- \(h_w\): Height of the web.
- \(t_w\): Web thickness.
- \(A_w\): Area of the web.
- \(A_{c,ef}\): Reduced area of the compressed flange.
- \(k\): Coefficient which depends on the class of the section.
- \(E\): Modulus of Elasticity.
- \(f_{c,ef}\): Steel elastic limit of the compressed flange.

Where:
\[f_{c,ef} = f_y\]

**Resistance to axial tension** (CTE DB SE-A, Article 6.2.3)
The check does not proceed, as there is no tensile axial force.

**Compression resistance** (CTE DB SE-A, Article 6.2.5)
The following criteria must be satisfied:
\[
\eta = \frac{N_{c,Ed}}{N_{c,Rd}} \leq 1
\]

\[\eta : 0.021 \]

\[
\eta = \frac{N_{c,Ed}}{N_{c,Rd}} \leq 1
\]

\[\eta : 0.026 \]

The worst case design force occurs for load combination 1.35·SW+1.05·Q1+1.5·V2+0.75·N1.

\(N_{c,Ed}\): Worst case design compressive axial force.

\[N_{c,Ed} : 9.02 \text{ kN}\]

The normal design compression force \(N_{c,Rd}\) should be taken as:

\[N_{c,Rd} = A \cdot f_{yd}\]

\[N_{c,Rd} : 429.52 \text{ kN}\]

Where:
- **Class**: Section class, depending on its deformation capacity and development of plastic resistance of the compressed elements of a section.
- \(A\): Area of the gross section for class 1, 2 and 3 sections.
- \(f_{yd}\): Steel design strength.

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\[ f_{yd} = f_y/\gamma_{M0} \]

Where:

- \( f_y \): Yield strength. (CTE DB SE-A, Table 4.1)
- \( \gamma_{M0} \): Partial safety factor of the material.

\[ f_y = 275.00 \text{ MPa} \]
\[ \gamma_{M0} = 1.05 \]

**Buckling resistance:** (CTE DB SE-A, Article 6.3.2)

The design buckling resistance \( N_{b,Rd} \) of a compressed bar is given by:

\[ N_{b,Rd} = \chi \cdot A \cdot f_{yd} \]

Where:

- \( A \): Area of the gross section for class 1, 2 and 3 sections.
- \( f_{yd} \): Steel design strength.

\[ A = 16.40 \text{ cm}^2 \]
\[ f_{yd} = 261.90 \text{ MPa} \]

\[ \chi : \text{Reduction coefficient due to buckling.} \]
\( \chi_y = 0.96 \]
\( \chi_z = 0.81 \]

Where:

\[ \phi = 0.5 \left[ 1 + \alpha \cdot \left( \bar{\lambda} - 0.2 \right) + \left( \bar{\lambda} \right)^2 \right] \]

\[ \alpha : \text{Elastic imperfection coefficient.} \]
\( \alpha_y = 0.21 \]
\( \alpha_z = 0.34 \]

\[ \bar{\lambda} : \text{Reduced slenderness.} \]
\( \bar{\lambda}_y = 0.38 \]
\( \bar{\lambda}_z = 0.66 \]

\[ N_{cr} : \text{Critical elastic buckling axial force, obtained from the smallest of the following values:} \]

- \( N_{cr,Y} \): Critical elastic buckling axial force with respect to the Y axis.
- \( N_{cr,Z} \): Critical elastic buckling axial force with respect to the Z axis.
- \( N_{cr,T} \): Critical elastic buckling axial force due to torsion.

\[ N_{cr,Y} = 1031.14 \text{ kN} \]
\[ N_{cr,Z} = 1031.14 \text{ kN} \]
\[ N_{cr,T} = \infty \]
Y - Axis bending resistance (CTE DB SE-A, Article 6.2.6)
The following criteria must be satisfied:

\[ \eta = \frac{M_{\text{Ed}}}{M_{c,Rd}} \leq 1 \]

\[ \eta : 0.327 \checkmark \]

For positive bending:
The worst case design force occurs at node N73, for load combination 1.35·SW+0.9·V3+1.5·N1.
\[ M_{\text{Ed}+} : \text{Worst case design bending moment.} \]
\[ M_{\text{Ed}+} : 7.55 \text{ kN·m} \]

For negative bending:
\[ M_{\text{Ed}^-} : \text{Worst case design bending moment.} \]
The design bending moment resistance \( M_{c,Rd} \) is given by:
\[ M_{c,Rd} = W_{pl,y} \cdot f_{yd} \]
\[ M_{c,Rd} : 23.13 \text{ kN·m} \]

Where:
- Class: Section class, depending on its deformation capacity and development of plastic resistance of the flat elements of a section submitted to simple bending.
- \( W_{pl,y} \): Plastic strength modulus corresponding to the fibre with greatest tension, for class 1 and 2 sections.
- \( f_{yd} \): Steel design strength.
  \[ f_{yd} = f_y / \gamma_M \]
  \[ f_{yd} : 261.90 \text{ MPa} \]
  Where:
  - \( f_y \): Yield strength. (CTE DB SE-A, Table 4.1)
  \[ f_y : 275.00 \text{ MPa} \]
  - \( \gamma_M \): Partial safety factor of the material.
    \[ \gamma_M : 1.05 \]

Z - Axis bending resistance (CTE DB SE-A, Article 6.2.6)
The following criteria must be satisfied:

\[ \eta = \frac{M_{\text{Ed}}}{M_{c,Rd}} \leq 1 \]

\[ \eta : 0.044 \checkmark \]

For positive bending:
The worst case design force occurs at node N73, for load combination 0.8·SW+1.5·V4.
\[ M_{\text{Ed}+} : \text{Worst case design bending moment.} \]
\[ M_{\text{Ed}+} : 0.01 \text{ kN·m} \]

For negative bending:

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The worst case design force occurs at node N73, for load combination 1.35·SW+1.05·Q1+1.5·V3+0.75·N1. 

\[ M_{Ed}^- : \text{Worst case design bending moment.} \]

The design bending moment resistance \( M_{c,Rd} \) is given by:

\[ M_{c,Rd} = W_{pl,z} \cdot f_{yd} \]

Where:

- **Class**: Section class, depending on its deformation capacity and development of plastic resistance of the flat elements of a section submitted to simple bending. 
  \( \text{Class} = 1 \)

- **\( W_{pl,z} \)**: Plastic strength modulus corresponding to the fibre with greatest tension, for class 1 and 2 sections.
  \( W_{pl,z} = 19.30 \, \text{cm}^3 \)

- **\( f_{yd} \)**: Steel design strength.
  \( f_{yd} = 261.90 \, \text{MPa} \)

\[ f_y \]: Yield strength. (CTE DB SE-A, Table 4.1) 
- **\( f_y \)**: 275.00 MPa
- **\( \gamma_0 \)**: Partial safety factor of the material. 
  **\( \gamma_0 = 1.05 \)**

\[ \gamma M_0 = \frac{f_{yd}}{f_y} \]

\[ \mathbf{M_{c,Rd}} : 5.05 \, \text{kN} \cdot \text{m} \]

\[ \mathbf{M_{Ed}^-} : 0.22 \, \text{kN} \cdot \text{m} \]

---

**Resistance to shear in the Z direction (CTE DB SE-A, Article 6.2.4)**

The following criteria must be satisfied:

\[ \eta = \frac{V_{Ed}}{V_{c,Rd}} \leq 1 \]

\[ \eta = 0.011 \checkmark \]

The worst case design force occurs at node N69, for load combination 1.35·SW+0.9·V1+1.5·N1. 

\[ V_{Ed} : \text{Worst case design shear force.} \]

The shear resistance \( V_{c,Rd} \) is given by:

\[ V_{c,Rd} = A_v \cdot \frac{f_{yd}}{\sqrt{3}} \]

Where:

- **\( A_v \)**: Transverse shear area. 
  **\( A_v = 7.62 \, \text{cm}^2 \)**

- **\( f_{yd} \)**: Steel design strength. 
  **\( f_{yd} = 261.90 \, \text{MPa} \)**

\[ \mathbf{V_{c,Rd}} : 115.17 \, \text{kN} \]

\[ \mathbf{V_{Ed}} : 1.21 \, \text{kN} \]

Where:

- **\( h \)**: Depth of the section. 
  **\( h = 140.00 \, \text{mm} \)**

- **\( t_w \)**: Web thickness. 
  **\( t_w = 4.70 \, \text{mm} \)**

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\[
f_{yd} = \frac{f_y}{\gamma_{M0}}
\]

Where:
- \(f_y\): Yield strength. (CTE DB SE-A, Table 4.1)
- \(\gamma_{M0}\): Partial safety factor of the material.

\[
f_{yd} = \frac{f_y}{\gamma_{M0}} = \frac{275.00}{1.05} = 261.90 \text{ MPa}
\]

**Shear buckling of the web:** (CTE DB SE-A, Article 6.3.3.4)
Even though transverse stiffeners have not been provided, it is not necessary to check the buckling resistance of the web, as the following is verified:

\[
\frac{d}{t_w} < 70 \cdot \varepsilon
\]

Where:
- \(\lambda_w\): Slenderness of the web.
- \(\lambda_w = \frac{d}{t_w}\)
- \(\lambda_{max}\): Maximum slenderness.
- \(\lambda_{max} = 70 \cdot \varepsilon\)
- \(\varepsilon\): Reduction factor.

\[
\varepsilon = \frac{f_{ref}}{f_y}
\]

Where:
- \(f_{ref}\): Reference elastic limit.
- \(f_{ref} = 235.00 \text{ MPa}\)
- \(f_y\): Yield strength. (CTE DB SE-A, Table 4.1)
- \(f_y = 275.00 \text{ MPa}\)

\[
\frac{d}{t_w} < 70 \cdot \varepsilon = \frac{2387}{6471} = 0.37 < 1
\]

**Resistance to shear in the Y direction** (CTE DB SE-A, Article 6.2.4)
The following criteria must be satisfied:

\[
\eta = \frac{V_{Ed}}{V_{c,Rd}} \leq 1
\]

\[
\eta = \frac{0.12}{158.30} = 0.0007 < 1
\]

The worst case design force occurs for load combination 1.35·SW+1.05·Q1+1.5·V3+0.75·N1.

- \(V_{Ed}\): Worst case design shear force.
- \(V_{Ed} = 0.12 \text{ kN}\)

The shear resistance \(V_{c,Rd}\) is given by:

\[
V_{c,Rd} = A_v \cdot \frac{f_{yd}}{\sqrt{3}}
\]

Where:
- \(A_v\): Transverse shear area.
- \(A_v = 10.47 \text{ cm}^2\)

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\[ A_y = A - d \cdot t_w \]

Where:
- \( A \): Area of the gross section.
- \( d \): Height of the web.
- \( t_w \): Web thickness.

\[ f_{yd} = \frac{f_y}{\gamma_{M0}} \]

Where:
- \( f_y \): Yield strength. (CTE DB SE-A, Table 4.1)
- \( \gamma_{M0} \): Partial safety factor of the material.

\( f_{yd} = 261.90 \text{ MPa} \)

\( f_y = 275.00 \text{ MPa} \)

\( \gamma_{M0} = 1.05 \)

**Combined bending moment \( Y \) and shear force \( Z \) resistance** (CTE DB SE-A, Article 6.2.8)

It is not necessary to reduce the design bending resistance, as the worst case shear force \( V_{Ed} \) is not greater than 50% of the design shear resistance \( V_{c,Rd} \).

\[
V_{Ed} \leq \frac{V_{c,Rd}}{2} \quad 1.21 \text{kN} \leq 57.58 \text{kN} \]

The worst case design forces occur for load combination 1.35\( \cdot \)SW+0.9\( \cdot \)V1+1.5\( \cdot \)N1.

- \( V_{Ed} \): Worst case design shear force.
- \( V_{Ed} = 1.21 \text{kN} \)

- \( V_{c,Rd} \): Design resistant shear force.
- \( V_{c,Rd} = 115.17 \text{kN} \)

**Combined bending moment \( Z \) and shear force \( Y \) resistance** (CTE DB SE-A, Article 6.2.8)

It is not necessary to reduce the design bending resistance, as the worst case shear force \( V_{Ed} \) is not greater than 50% of the design shear resistance \( V_{c,Rd} \).

\[
V_{Ed} \leq \frac{V_{c,Rd}}{2} \quad 0.12 \text{kN} \leq 79.15 \text{kN} \]

The worst case design forces occur for load combination 1.35\( \cdot \)SW+1.05\( \cdot \)Q1+1.5\( \cdot \)V3+0.75\( \cdot \)N1.

- \( V_{Ed} \): Worst case design shear force.
- \( V_{Ed} = 0.12 \text{kN} \)


$$V_{c,Rd} : \text{Design resistant shear force.}$$

$$V_{c,Rd} : 158.30 \, \text{kN}$$

**Combined bending and axial resistance** (CTE DB SE-A, Article 6.2.8)

The following criteria must be satisfied:

$$\eta = \frac{N_{c,Ed}}{N_{pl,Rd}} + \frac{M_{y,Ed}}{M_{pl,Rd,y}} + \frac{M_{z,Ed}}{M_{pl,Rd,z}} \leq 1$$

$$\eta : 0.380 \, \checkmark$$

$$\eta = \frac{N_{c,Ed}}{X_y \cdot A \cdot f_{yd}} + k_y \cdot \frac{c_{pl,y} \cdot M_{y,Ed}}{X_{LT} \cdot W_{pl,y} \cdot f_{yd}} + \alpha_2 \cdot k_z \cdot \frac{c_{pl,z} \cdot M_{z,Ed}}{W_{pl,z} \cdot f_{yd}} \leq 1$$

$$\eta : 0.366 \, \checkmark$$

$$\eta = \frac{N_{c,Ed}}{X_z \cdot A \cdot f_{yd}} + \alpha_y \cdot k_y \cdot \frac{c_{pl,y} \cdot M_{y,Ed}}{W_{pl,y} \cdot f_{yd}} + \alpha_z \cdot k_z \cdot \frac{c_{pl,z} \cdot M_{z,Ed}}{W_{pl,z} \cdot f_{yd}} \leq 1$$

$$\eta : 0.257 \, \checkmark$$

The worst case design forces occur at node N73, for load combination 1.35·SW+1.05·Q1+0.9·V3+1.5·N1.

Where:

- **Nc,Ed**: Worst case design compressive axial force.
- **M,y,Ed, M,z,Ed**: Worst case bending moments, in accordance with the Y and Z axes, respectively.
- **Class**: Section class, according to its deformation capacity and plastic resistance development of its flat elements, for axial load and simple bending.
- **Npl,Rd**: Compressive resistance of the gross section.
- **Mpl,Rd,y, Mpl,Rd,z**: Bending resistance of the gross section in plastic conditions, with respect to the Y and Z axes, respectively.
- **Buckling resistance**: (CTE DB SE-A, Article 6.3.4.2)
  - **A**: Area of the gross section.
  - **Wpl,y, Wpl,z**: Plastic resistant modules corresponding to the compressed fibre, about the Y and Z, respectively.
  - **f,yd**: Steel design strength.
    - **f,yd** = \( f_y / \gamma_{M1} \)
    - Where:
      - **f,y**: Yield strength. (CTE DB SE-A, Table 4.1)
      - **\gamma_{M1}**: Partial safety factor of the material.
  - **k,y, k,z**: Interaction coefficients.
    $$k_y = 1 + (\alpha_y - 0.2) \cdot \frac{N_{c,Ed}}{X_y \cdot N_{c,Rd}}$$
    $$k_y : 1.00$$

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\[ k_2 = 1 + \left( 2 \cdot \chi_2 - 0.6 \right) \cdot \frac{N_{Ed}}{\chi_2 \cdot N_{c,Rd}} \]

\[ k_2 : 1.02 \]

\[ C_{m,y}, C_{m,z}: \text{Equivalent uniform bending moment factors.} \]

\[ C_{m,y} : 1.00 \]

\[ C_{m,z} : 1.00 \]

\[ \chi_y, \chi_z: \text{Buckling reduction coefficients, about the Y and Z axes, respectively.} \]

\[ \chi_y : 0.96 \]

\[ \chi_z : 0.81 \]

\[ \overline{\lambda}_y, \overline{\lambda}_z: \text{Reduced slenderness with values no greater than 1.00, with respect to the Y and Z axes, respectively.} \]

\[ \overline{\lambda}_y : 0.38 \]

\[ \overline{\lambda}_z : 0.66 \]

\[ \alpha_y, \alpha_z: \text{Factors that depend on the class of the section.} \]

\[ \alpha_y : 0.60 \]

\[ \alpha_z : 0.60 \]

**Combined bending, axial and shear resistance** (CTE DB SE-A, Article 6.2.8)

It is not necessary to reduce the design bending and axial force resistance, as the buckling effect can be ignored due to shear. Additionally, the worst case design shear force \( V_{Ed} \) is less than or equal to 50% of the design shear resistance \( V_{c,Rd} \).

The worst case design forces occur for load combination \( 1.35 \cdot SW + 0.9 \cdot V_1 + 1.5 \cdot N_1 \).

\[ V_{Ed,z} \leq \frac{V_{c,Rd,z}}{2} \]

1.21 kN \( \leq \) 57.58 kN

Where:

- \( V_{Ed,z} \): Worst case design shear force.
- \( V_{c,Rd,z} \): Design resistant shear force.

\[ V_{Ed,z} : 1.21 \text{ kN} \]

\[ V_{c,Rd,z} : 115.17 \text{ kN} \]

**Torsional resistance** (CTE DB SE-A, Article 6.2.7)

The following criteria must be satisfied:

\[ \eta = \frac{M_{T,Ed}}{M_{T,Rd}} \leq 1 \]

\[ \eta : 0.004 \]

The worst case design force occurs for load combination \( 0.8 \cdot SW + 1.5 \cdot V_3 \).

\[ M_{T,Ed}: \text{Worst case design torsional moment.} \]

\[ M_{T,Ed} : 0.00 \text{ kN-m} \]

The design torsional moment resistance \( M_{T,Rd} \) is given by:

\[ M_{T,Rd} : 0.54 \text{ kN-m} \]
Combined Z shear and torsional resistance (CTE DB SE-A, Article 6.2.8)
The following criteria must be satisfied:

\[
\eta = \frac{V_{Ed}}{V_{pl,T,Rd}} \leq 1
\]

\[
\eta : 0.009 \checkmark
\]

The worst case design forces occur at node N69, for load combination 1.35·SW+0.9·V3+1.5·N1.

\[
V_{Ed}: \text{Worst case design shear force.} \quad V_{Ed} : 1.09 \text{ kN}
\]
\[
M_{T,Ed}: \text{Worst case design torsional moment.} \quad M_{T,Ed} : 0.00 \text{ kN·m}
\]

The reduced design resistant shear force \(V_{pl,T,Rd}\) is given by:

\[
V_{pl,T,Rd} = \frac{1 - \tau_{T,Ed}}{1.25 \cdot \frac{f_{yd}}{\sqrt{3}}} V_{pl,Rd}
\]

Where:

\[
V_{pl,Rd}: \text{Design resistant shear force.} \quad V_{pl,Rd} : 115.05 \text{ kN}
\]

\[
\tau_{T,Ed}: \text{Tangential stresses due to torsion.} \quad \tau_{T,Ed} : 0.39 \text{ MPa}
\]

\[
\tau_{T,Ed} = \frac{M_{T,Ed}}{W_t}
\]

Where:

\[
W_t: \text{Torsion resistance module.} \quad W_t : 3.55 \text{ cm}^3
\]
\[
f_{yd}: \text{Steel design strength.} \quad f_{yd} = 261.90 \text{ MPa}
\]
\[
f_{yd} = f_y / \gamma_{M0}
\]

Where:

\[
f_y: \text{Yield strength. (CTE DB SE-A, Table 4.1)} \quad f_y : 275.00 \text{ MPa}
\]
\[
\gamma_{M0}: \text{Partial safety factor of the material.} \quad \gamma_{M0} : 1.05
\]

Combined Y shear and torsional resistance (CTE DB SE-A, Article 6.2.8)
The following criteria must be satisfied:
\[ \eta = \frac{V_{Ed}}{V_{pl,T,Rd}} \leq 1 \]

The worst case design forces occur for load combination \(1.35 \cdot SW + 1.05 \cdot Q1 + 1.5 \cdot V3 + 0.75 \cdot N1\).

\( V_{Ed} \): Worst case design shear force.

\( M_{T,Ed} \): Worst case design torsional moment.

The reduced design resistant shear force \( V_{pl,T,Rd} \) is given by:

\[ V_{pl,T,Rd} = \sqrt{1 - \frac{\tau_{T,Ed}}{1.25 \cdot f_{yd} / \sqrt{3}}} \cdot V_{pl,Rd} \]

Where:

\( V_{pl,Rd} \): Design resistant shear force.

\( \tau_{T,Ed} \): Tangential stresses due to torsion.

\[ \tau_{T,Ed} = \frac{M_{T,Ed}}{W_T} \]

Where:

\( W_T \): Torsion resistance module.

\( f_{yd} \): Steel design strength.

\[ f_{yd} = f_y / \gamma_{M0} \]

Where:

\( f_y \): Yield strength. (CTE DB SE-A, Table 4.1)

\( \gamma_{M0} \): Partial safety factor of the material.

\( \eta \): 0.001

\( V_{Ed} \): 0.12 kN

\( M_{T,Ed} \): 0.00 kN·m

\( V_{pl,T,Rd} \): 158.03 kN

\( V_{pl,Rd} \): 158.30 kN

\( \tau_{T,Ed} \): 0.65 MPa

\( W_T \): 3.55 cm³

\( f_{yd} \): 261.90 MPa

\( f_y \): 275.00 MPa

\( \gamma_{M0} \): 1.05
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2.6 Section 6

Section: IPE 160
Material: Steel (S275)

<table>
<thead>
<tr>
<th>Nodes</th>
<th>Length (m)</th>
<th>Mechanical characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial</td>
<td>Final</td>
<td>Area (cm²)</td>
</tr>
<tr>
<td>N61</td>
<td>N62</td>
<td>3.150</td>
</tr>
</tbody>
</table>

Notes:
(1) Inertia with respect to the indicated axis
(2) Uniform torsional moment of inertia

<table>
<thead>
<tr>
<th>XY plane</th>
<th>XZ plane</th>
<th>Top fl.</th>
<th>Bot. fl.</th>
</tr>
</thead>
<tbody>
<tr>
<td>β</td>
<td>0.50</td>
<td>1.00</td>
<td>0.00</td>
</tr>
<tr>
<td>L_c</td>
<td>1.575</td>
<td>3.150</td>
<td>0.000</td>
</tr>
<tr>
<td>C_m</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>C_1</td>
<td>0.00</td>
<td>1.000</td>
<td></td>
</tr>
</tbody>
</table>

Notes:
- XY plane: Buckling coefficient
- XZ plane: Buckling length (m)
- Top fl.: Moment coefficient
- Bot. fl.: Critical moment modification factor

<table>
<thead>
<tr>
<th>Bar</th>
<th>I_y</th>
<th>I_z</th>
<th>A</th>
<th>N_t</th>
<th>N_c</th>
<th>M_Y</th>
<th>M_Z</th>
<th>V_Z</th>
<th>V_Y</th>
<th>M_Y V_Z</th>
<th>M_Z V_Y</th>
<th>M_Y M_Z</th>
<th>M_Y M_Z V_Y</th>
<th>M</th>
<th>M</th>
<th>M</th>
<th>C_m</th>
<th>C_1</th>
</tr>
</thead>
<tbody>
<tr>
<td>N61/N62</td>
<td>0.197 m</td>
<td>0.100 m</td>
<td>1.575 m</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Status</th>
<th>Checks (CTE DB SE-A)</th>
</tr>
</thead>
<tbody>
<tr>
<td>VERIFIED</td>
<td>q = 72.2</td>
</tr>
</tbody>
</table>

Notes:
- Slenderness limit
- L_c: Crushing of the web induced by the compressed flange
- N_t: Resistance to axial tension
- N_c: Compression resistance
- M_Y: Bending resistance Y-axis
- M_Z: Bending resistance Z-axis
- V_Z: Resistance to shear in the Z direction
- V_Y: Resistance to shear in the Y direction
- M_Y V_Z: Combined bending moment Y and shear force Z resistance
- M_Z V_Y: Combined bending moment Z and shear force Y resistance
- M_Y M_Z: Combined bending and axial resistance
- M_Y M_Z V_Y V_Z: Combined bending, axial and shear resistance
- M_t: Torsional resistance
- M_t V_Z: Combined Z shear and torsional resistance
- M_t V_Y: Combined Y shear and torsional resistance
- S: Distance to the origin of the bar
- η: Usage coefficient (%)
- D.N.P.: Not applicable

Checks that do not proceed (D.N.P.):
1. The check does not proceed, as there is no bending moment.
2. The check does not proceed, as there is no shear force.
3. The check does not proceed, as there is no interaction between shear forces and bending moments for any loadcase combination.
4. The check does not proceed, as there is no torsional moment.
5. There is no interaction between torsional moment and shear force for any combination. Therefore the check does not proceed.

Slenderness limit (CTE DB SE-A, Articles 6.3.1 and 6.3.2.1 - Table 6.3)
The reduced slenderness \( \bar{\lambda} \) of the bars in compression must be less than 2.0.

\[
\bar{\lambda} = \frac{A \cdot f_y}{N_t}
\]

Where:
- \( A \): Area of the gross section for class 1, 2 and 3 sections.
- \( f_y \): Yield strength. (CTE DB SE-A, Table 4.1)

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\( N_{cr} \): Critical elastic buckling axial force.

The critical elastic buckling axial force \( N_{cr} \) is the smallest of the following values a), b) and c):

a) Critical elastic buckling axial force with respect to the Y axis.
\[
N_{cr,Y} = \frac{\pi^2 \cdot E \cdot I_y}{L_{ky}^2}
\]

b) Critical elastic buckling axial force with respect to the Z axis.
\[
N_{cr,Z} = \frac{\pi^2 \cdot E \cdot I_z}{L_{kz}^2}
\]

c) Critical elastic buckling axial force due to torsion.
\[
N_{cr,T} = \frac{1}{i_0^2} \left[ G \cdot I_t + \frac{\pi^2 \cdot E \cdot I_w}{L_{kt}^2} \right]
\]

Where:
- \( I_y \): Moment of inertia of the gross section, with respect to the Y-axis.
- \( I_z \): Moment of inertia of the gross section, with respect to the Z-axis.
- \( I_t \): Uniform torsional moment of inertia.
- \( I_w \): Section warping constant.
- \( E \): Modulus of Elasticity.
- \( G \): Elastic modulus of steel.
- \( L_{ky} \): Effective buckling length due to bending, with respect to the Y-axis.
- \( L_{kz} \): Effective buckling length due to bending, with respect to the Z-axis.
- \( L_{kt} \): Effective buckling length due to torsion.
- \( i_0 \): Polar radius of gyration of the gross section, with respect to the centre of torsion.
\[
i_0 = \left( i_y^2 + i_z^2 + y_0^2 + z_0^2 \right)^{0.5}
\]

Crushing of the web induced by the compressed flange (CYPE criteria, based on: Eurocode 3 EN 1993-1-5: 2006, Article 8)

The following criteria must be satisfied:

\[
\begin{align*}
\pi \cdot \frac{y^2}{L_{ky}^2} &= 2 \\
\pi \cdot \frac{z^2}{L_{kz}^2} &= 2 \\
\pi \cdot \frac{w^2}{L_{kt}^2} &= 2 \left( i_0^2 + y_0^2 + z_0^2 \right)^{0.5} + \frac{E}{G} \left( i_y^2 + i_z^2 \right) + \frac{G}{E} \left( i_0^2 + i_y^2 + i_z^2 \right)
\end{align*}
\]

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\[
\frac{h_w}{t_w} \leq k \frac{E}{f_{yt}} \sqrt{A_{fc,ef}}
\]

Where:
- \(h_w\): Height of the web.
- \(t_w\): Web thickness.
- \(A_w\): Area of the web.
- \(A_{fc,ef}\): Reduced area of the compressed flange.
- \(k\): Coefficient which depends on the class of the section.
- \(E\): Modulus of Elasticity.
- \(f_{yt}\): Steel elastic limit of the compressed flange.

Where:
- \(f_{yt} = f_y\)

\[
\begin{align*}
29.04 & \leq 250.58 & \checkmark 
\end{align*}
\]

**Resistance to axial tension** (CTE DB SE-A, Article 6.2.3)

The following criteria must be satisfied:

\[
\eta = \frac{N_{t,Ed}}{N_{t,Rd}} \leq 1
\]

\[
\eta : 0.003 & \checkmark 
\]

The worst case design force occurs for load combination 0.8·SW+1.5·V4+0.75·N1.

- \(N_{t,Ed}\): Worst case design axial tensile force.
- \(N_{t,Rd}\): Design tensile resistance.

\[
N_{t,Ed} = 1.62 \text{ kN}
\]

\[
N_{t,Rd} = 526.43 \text{ kN}
\]

The design tensile resistance \(N_{t,Rd}\) is given by:

\[
N_{t,Rd} = A \cdot f_{yd}
\]

Where:
- \(A\): Gross transverse section of the bar.
- \(f_{yd}\): Steel design strength.

\[
f_{yd} = \frac{f_y}{\gamma_{MO}}
\]

Where:
- \(f_y\): Yield strength. (CTE DB SE-A, Table 4.1)
- \(\gamma_{MO}\): Partial safety factor of the material.

\[
f_y = 275.00 \text{ MPa}
\]

\[
\gamma_{MO} = 1.05
\]

**Compression resistance** (CTE DB SE-A, Article 6.2.5)

The following criteria must be satisfied:
\[ \eta = \frac{N_{c,Ed}}{N_{c,Rd}} \leq 1 \]

\[ \eta = \frac{N_{b,Ed}}{N_{b,Rd}} \leq 1 \]

The worst case design force occurs for load combination 1.35·SW+1.05·Q1+1.5·V3.

**N \(_{c,Ed}\):** Worst case design compressive axial force. 

\[ N_{c,Ed} = 5.03 \text{ kN} \]

The normal design compression force **N \(_{c,Rd}\)** should be taken as:

\[ N_{c,Rd} = A \cdot f_{yd} \]

Where:

**Class:** Section class, depending on its deformation capacity and development of plastic resistance of the compressed elements of a section.

**A:** Area of the gross section for class 1, 2 and 3 sections.

**f\(_{yd}\):** Steel design strength.

\[ f_{yd} = \frac{f_y}{\gamma_{Mo}} \]

Where:

**f\(_{y}\):** Yield strength. (CTE DB SE-A, Table 4.1)

**\(\gamma_{Mo}\):** Partial safety factor of the material.

\[ f_{yd} = 261.90 \text{ MPa} \]

\[ f_{y} = 275.00 \text{ MPa} \]

\[ \gamma_{Mo} = 1.05 \]

**Buckling resistance:** (CTE DB SE-A, Article 6.3.2)

The design buckling resistance **N \(_{b,Rd}\)** of a compressed bar is given by:

\[ N_{b,Rd} = \chi \cdot A \cdot f_{yd} \]

Where:

**A:** Area of the gross section for class 1, 2 and 3 sections.

**f\(_{yd}\):** Steel design strength.

\[ f_{yd} = \frac{f_y}{\gamma_{M1}} \]

Where:

**f\(_{y}\):** Yield strength. (CTE DB SE-A, Table 4.1)

**\(\gamma_{M1}\):** Partial safety factor of the material.

\[ f_{yd} = 261.90 \text{ MPa} \]

\[ f_{y} = 275.00 \text{ MPa} \]

\[ \gamma_{M1} = 1.05 \]

**\(\chi\):** Reduction coefficient due to buckling.

\[ \chi = \frac{1}{\phi + \sqrt{\phi^2 - (\lambda)^2}} \leq 1 \]

Where:

\[ \phi = 0.69 \]

\[ \lambda = 0.91 \]

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\[
\Phi = 0.5 \left[ 1 + \alpha \cdot (\bar{\lambda} - 0.2) + (\bar{\lambda})^2 \right]
\]

\(\alpha\): Elastic imperfection coefficient.
\(\bar{\lambda}\): Reduced slenderness.

\[
\bar{\lambda} = \frac{A \cdot f_y}{N_{cr}}
\]

- \(N_{cr}\): Critical elastic buckling axial force, obtained from the smallest of the following values:
  - \(N_{cr,Y}\): Critical elastic buckling axial force with respect to the Y axis.
  - \(N_{cr,Z}\): Critical elastic buckling axial force with respect to the Z axis.
  - \(N_{cr,T}\): Critical elastic buckling axial force due to torsion.

Y - Axis bending resistance (CTE DB SE-A, Article 6.2.6)

The following criteria must be satisfied:

\[
\eta = \frac{M_{Ed}}{M_{c,Rd}} \leq 1
\]

\(\eta\): 0.716  

For positive bending:
The worst case design force occurs at a point situated at a distance of 1.575 m from node N61, for load combination 1.35·SW+1.5·Q1.

\(M_{Ed^+}\): Worst case design bending moment.

\(M_{Ed^+}\) = 23.25 kN·m

For negative bending:

\(M_{Ed^-}\): Worst case design bending moment.

\(M_{Ed^-}\) = 0.00 kN·m

The design bending moment resistance \(M_{c,Rd}\) is given by:

\[
M_{c,Rd} = W_{pl,Y} \cdot f_{yd}
\]

\(M_{c,Rd}\) = 32.48 kN·m

Where:

- **Class**: Section class, depending on its deformation capacity and development of plastic resistance of the flat elements of a section submitted to simple bending.
- \(W_{pl,Y}\): Plastic strength modulus corresponding to the fibre with greatest tension, for class 1 and 2 sections.
- \(f_{yd}\): Steel design strength.
  
\[
f_{yd} = f_y / \gamma_{M0}
\]

Where:

- \(f_y\): Yield strength. (CTE DB SE-A, Table 4.1)
- \(\gamma_{M0}\): Partial safety factor of the material.

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Lateral buckling resistance: (CTE DB SE-A, Article 6.3.3.2)
Does not proceed, as the lateral buckling lengths are null.

Z - Axis bending resistance (CTE DB SE-A, Article 6.2.6)
The check does not proceed, as there is no bending moment.

Resistance to shear in the Z direction (CTE DB SE-A, Article 6.2.4)
The following criteria must be satisfied:

\[ \eta = \frac{V_{Ed}}{V_{c,Rd}} \leq 1 \]
\[ \eta : 0.202 \checkmark \]

The worst case design force occurs at node N61, for load combination 1.35·SW+1.5·Q1.

\[ V_{Ed} : \text{Worst case design shear force.} \]
\[ V_{Ed} : 29.52 \text{ kN} \]

The shear resistance \( V_{c,Rd} \) is given by:

\[ V_{c,Rd} = A_v \cdot \frac{f_{yd}}{\sqrt{3}} \]
\[ V_{c,Rd} : 146.16 \text{ kN} \]

Where:
\[ A_v : \text{Transverse shear area.} \]
\[ A_v = h \cdot t_w \]
\[ h : \text{Depth of the section.} \]
\[ t_w : \text{Web thickness.} \]
\[ f_{yd} : \text{Steel design strength.} \]
\[ f_{yd} = f_y / \gamma_M \]
\[ f_y : 275.00 \text{ MPa} \]
\[ \gamma_M : 1.05 \]

Shear buckling of the web: (CTE DB SE-A, Article 6.3.3.4)
Even though transverse stiffeners have not been provided, it is not necessary to check the buckling resistance of the web, as the following is verified:

\[ \frac{d}{t_w} < 70 \cdot \varepsilon \]
\[ 25.44 < 64.71 \checkmark \]
Where:

\( \lambda_w \): Slenderness of the web.
\[
\lambda_w = \frac{d}{t_w}
\]

\( \lambda_{\text{max}} \): Maximum slenderness.
\[
\lambda_{\text{max}} = 70 \cdot \varepsilon
\]

\( \varepsilon \): Reduction factor.
\[
\varepsilon = \frac{f_{\text{ref}}}{f_y}
\]

Where:

\( f_{\text{ref}} \): Reference elastic limit.

\( f_{\text{y}} \): Yield strength. (CTE DB SE-A, Table 4.1)

\( \lambda_w : 25.44 \)

\( \lambda_{\text{max}} : 64.71 \)

\( \varepsilon : 0.92 \)

\( f_{\text{ref}} : 235.00 \text{ MPa} \)

\( f_{\text{y}} : 275.00 \text{ MPa} \)

**Resistance to shear in the Y direction** (CTE DB SE-A, Article 6.2.4)

The check does not proceed, as there is no shear force.

**Combined bending moment Y and shear force Z resistance** (CTE DB SE-A, Article 6.2.8)

It is not necessary to reduce the design bending resistance, as the worst case shear force \( V_{\text{Ed}} \) is not greater than 50% of the design shear resistance \( V_{c,Rd} \).

\[
V_{\text{Ed}} \leq \frac{V_{c,Rd}}{2}
\]

\( 25.83 \text{ kN} \leq 73.08 \text{ kN} \) \( \checkmark \)

The worst case design forces occur at a point situated at a distance of 0.197 m from node N61, for load combination 1.35\( \cdot \)SW+1.5\( \cdot \)Q1.

\( V_{\text{Ed}} \): Worst case design shear force.

\( V_{\text{Ed}} : 25.83 \text{ kN} \)

\( V_{c,Rd} \): Design resistant shear force.

\( V_{c,Rd} : 146.16 \text{ kN} \)

**Combined bending moment Z and shear force Y resistance** (CTE DB SE-A, Article 6.2.8)

The check does not proceed, as there is no interaction between shear forces and bending moments for any loadcase combination.

**Combined bending and axial resistance** (CTE DB SE-A, Article 6.2.8)

The following criteria must be satisfied:
The worst case design forces occur at a point situated at a distance of 1.575 m from node N61, for load combination 1.35·SW+1.5·Q1+0.9·V3.

Where:

- **Nc,Ed**: Worst case design compressive axial force.
- **My,Ed**, **Mz,Ed**: Worst case bending moments, in accordance with the Y and Z axes, respectively.
- **Class**: Section class, according to its deformation capacity and plastic resistance development of its flat elements, for axial load and simple bending.
- **Npl,Rd**: Compressive resistance of the gross section.
- **Mpl,Rd,y**, **Mpl,Rd,z**: Bending resistance of the gross section in plastic conditions, with respect to the Y and Z axes, respectively.

**Buckling resistance**: (CTE DB SE-A, Article 6.3.4.2)

- **A**: Area of the gross section.
- **Wpl,y**, **Wpl,z**: Plastic resistant modules corresponding to the compressed fibre, about the Y and Z axes, respectively.
- **f_yd**: Steel design strength.

\[ f_{yd} = f_y / \gamma_{M1} \]

Where:

- **f_y**: Yield strength. (CTE DB SE-A, Table 4.1)
- **\gamma_{M1}**: Partial safety factor of the material.

| \( \gamma_{M1} \) | 1.05 |
|--------------------|

- **k_y**, **k_z**: Interaction coefficients.

\[ k_y = 1 + (\chi_y - 0.2) \cdot \frac{N_{c,Ed}}{\chi_y \cdot N_{c,Rd}} \]

\[ k_z = 1 + (2 \cdot \chi_z - 0.6) \cdot \frac{N_{c,Ed}}{\chi_z \cdot N_{c,Rd}} \]

- **Cm,y**, **Cm,z**: Equivalent uniform bending moment factors.

| \( C_{m,y} \) | 1.00 |
|---------------|
| \( C_{m,z} \) | 1.00 |
\( \chi_y, \chi_z \): Buckling reduction coefficients, about the Y and Z axes, respectively.

\[ \chi_y : 0.91 \]
\[ \chi_z : 0.61 \]

\( \lambda_y, \lambda_z \): Reduced slenderness with values no greater than 1.00, with respect to the Y and Z axes, respectively.

\[ \lambda_y : 0.55 \]
\[ \lambda_z : 0.98 \]

\( \alpha_y, \alpha_z \): Factors that depend on the class of the section.

\[ \alpha_y : 0.60 \]
\[ \alpha_z : 0.60 \]

**Combined bending, axial and shear resistance** (CTE DB SE-A, Article 6.2.8)

It is not necessary to reduce the design bending and axial force resistance, as the buckling effect can be ignored due to shear. Additionally, the worst case design shear force \( V_{Ed} \) is less than or equal to 50% of the design shear resistance \( V_{c,Rd} \).

The worst case design forces occur at a point situated at a distance of 0.197 m from node N61, for load combination 1.35·SW+1.5·Q1.

\[ \frac{V_{Ed,z}}{2} \leq \frac{V_{c,Rd,z}}{2} \]

Where:
- \( V_{Ed,z} \): Worst case design shear force.
- \( V_{c,Rd,z} \): Design resistant shear force.

\[ 25.83 \text{ kN} \leq 73.08 \text{ kN} \]

The worst case design shear force is 25.83 kN, which is less than or equal to 50% of the design resistant shear force 146.16 kN.
### 2.7 Section 7

#### Section: SHS 80x4.0

**Material: Steel (S275)**

<table>
<thead>
<tr>
<th>Nodes</th>
<th>Length (m)</th>
<th>Mechanical characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial</td>
<td>Final</td>
<td>Area (cm²)</td>
</tr>
<tr>
<td>N54</td>
<td>N53</td>
<td>2.000</td>
</tr>
</tbody>
</table>

Notes:
- (1) Inertia with respect to the indicated axis
- (2) Uniform torsional moment of inertia

#### Buckling

<table>
<thead>
<tr>
<th>Type</th>
<th>XY plane</th>
<th>XZ plane</th>
<th>Top fl.</th>
<th>Bot. fl.</th>
</tr>
</thead>
<tbody>
<tr>
<td>β</td>
<td>0.50</td>
<td>1.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>L&lt;sub&gt;k&lt;/sub&gt;</td>
<td>1.000</td>
<td>2.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>C&lt;sub&gt;m&lt;/sub&gt;</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>C&lt;sub&gt;1&lt;/sub&gt;</td>
<td>-</td>
<td>1.000</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note:
- β: Buckling coefficient
- L<sub>k</sub>: Buckling length (m)
- C<sub>m</sub>: Moment coefficient
- C<sub>1</sub>: Critical moment modification factor

#### Slenderness limit (CTE DB SE-A, Articles 6.3.1 and 6.3.2.1 - Table 6.3)

The reduced slenderness value, \( \bar{\lambda} \), of the bars in tension must not exceed 3.0.

\[
\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}}
\]

Where:
- \( A \): Gross transverse section of the bar.
- \( f_y \): Yield strength. (CTE DB SE-A, Table 4.1)
- \( N_{cr} \): Critical elastic buckling axial force.

The critical elastic buckling axial force \( N_{cr} \) is the smallest of the following values a), b) and c):

- a) \( A \cdot f_y \)
- b) \( N_{cr} \)
- c) \( M_{t} \)

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a) Critical elastic buckling axial force with respect to the Y axis.

\[ N_{cr,y} = \frac{\pi^2 \cdot E \cdot I_y}{L_{ky}^2} \]

\( N_{cr,y} : 573.23 \) kN

b) Critical elastic buckling axial force with respect to the Z axis.

\[ N_{cr,z} = \frac{\pi^2 \cdot E \cdot I_z}{L_{kz}^2} \]

\( N_{cr,z} : 2292.94 \) kN

c) Critical elastic buckling axial force due to torsion.

\[ N_{cr,T} = \frac{1}{i_0^2} \left[ G \cdot I_t + \frac{\pi^2 \cdot E \cdot I_w}{L_{kt}^2} \right] \]

\( N_{cr,T} : \infty \)

Where:

- \( I_y \): Moment of inertia of the gross section, with respect to the Y-axis.
- \( I_z \): Moment of inertia of the gross section, with respect to the Z-axis.
- \( I_t \): Uniform torsional moment of inertia.
- \( I_w \): Section warping constant.
- \( E \): Modulus of Elasticity.
- \( G \): Elastic modulus of steel.
- \( L_{ky} \): Effective buckling length due to bending, with respect to the Y-axis.
- \( L_{kz} \): Effective buckling length due to bending, with respect to the Z-axis.
- \( L_{kt} \): Effective buckling length due to torsion.
- \( i_0 \): Polar radius of gyration of the gross section, with respect to the centre of torsion.

\( i_0 = (i_y^2 + i_z^2 + Y_0^2 + Z_0^2)^{0.5} \)

Where:

- \( i_y, i_z \): Radii of gyration of the gross section, with respect to the main axes of inertia Y and Z.
- \( Y_0, Z_0 \): Coordinates of the torsional centre in the direction of the main axes Y and Z, respectively, relative to the centre of gravity of the section.

Crushing of the web induced by the compressed flange (CYPE criteria, based on: Eurocode 3 EN 1993-1-5: 2006, Article 8)

The following criteria must be satisfied:

\[ \frac{h_w}{t_w} \leq k \cdot \frac{E \cdot A_y}{f_y \cdot \sqrt{A_{fc,ef}}} \]

\( 18.00 \leq 307.36 \)

Where:

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\( h_w \): Height of the web.  
\( t_w \): Web thickness.  
\( A_w \): Area of the web.  
\( A_{fc,ef} \): Reduced area of the compressed flange.  
\( k \): Coefficient which depends on the class of the section.  
\( E \): Modulus of Elasticity.  
\( f_{yf} \): Steel elastic limit of the compressed flange. Where:  
\( f_{yf} = f_y \).

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( h_w )</td>
<td>72.00 mm</td>
<td>( t_w )</td>
<td>4.00 mm</td>
</tr>
<tr>
<td>( A_w )</td>
<td>5.76 cm²</td>
<td>( A_{fc,ef} )</td>
<td>3.20 cm²</td>
</tr>
<tr>
<td>( k )</td>
<td>0.30</td>
<td>( E )</td>
<td>210000 MPa</td>
</tr>
<tr>
<td>( f_{yf} )</td>
<td>275.00 MPa</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Resistance to axial tension** (CTE DB SE-A, Article 6.2.3)

The following criteria must be satisfied:

\[
\eta = \frac{N_{t,Ed}}{N_{t,Rd}} \leq 1
\]

\( \eta = 0.028 \checkmark \)

The worst case design force occurs for load combination 1.35·SW+1.05·Q1+0.9·V3+1.5·N1.

\( N_{t,Ed} \): Worst case design axial tensile force.  
\( N_{t,Ed} = 8.58 \) kN

The design tensile resistance \( N_{t,Rd} \) is given by:

\[ N_{t,Rd} = A \cdot f_{yd} \]

\( N_{t,Rd} = 307.43 \) kN

Where:  
\( A \): Gross transverse section of the bar.  
\( f_{yd} \): Steel design strength.  
\( f_{yd} = f_y / \gamma_M \)

Where:  
\( f_y \): Yield strength. (CTE DB SE-A, Table 4.1)  
\( \gamma_M \): Partial safety factor of the material.

\( f_y = 275.00 \) MPa  
\( \gamma_M = 1.05 \)

**Compression resistance** (CTE DB SE-A, Article 6.2.5)

The check does not proceed, as there is no compressive axial force.

**Y - Axis bending resistance** (CTE DB SE-A, Article 6.2.6)

The following criteria must be satisfied:
\[ \eta = \frac{M_{Ed}}{M_{c,Rd}} \leq 1 \]

\[ \eta : 0.108 \]

For positive bending:
The worst case design force occurs at a point situated at a distance of 0.424 m from node N54, for load combination 1.35·SW+1.05·Q1+0.9·V2+1.5·N1.

\[ M_{Ed}^+ : \text{Worst case design bending moment.} \]

\[ M_{Ed}^+ : 0.93 \text{ kN·m} \]

For negative bending:
\[ M_{Ed}^- : \text{Worst case design bending moment.} \]

\[ M_{Ed}^- : 0.00 \text{ kN·m} \]

The design bending moment resistance \( M_{c,Rd} \) is given by:
\[ M_{c,Rd} = W_{pl,y} \cdot f_{yd} \]

Where:

- **Class**: Section class, depending on its deformation capacity and development of plastic resistance of the flat elements of a section submitted to simple bending.
- **W_{pl,y}**: Plastic strength modulus corresponding to the fibre with greatest tension, for class 1 and 2 sections.
- **f_{yd}**: Steel design strength.

\[ f_{yd} = \frac{f_y}{\gamma_{M0}} \]

Where:

- \( f_y \): Yield strength. (CTE DB SE-A, Table 4.1)
- \( \gamma_{M0} \): Partial safety factor of the material.

\[ f_y : 261.90 \text{ MPa} \]

\[ \gamma_{M0} : 1.05 \]

\[ W_{pl,y} : 33.01 \text{ cm}^3 \]

\[ M_{c,Rd} : 8.64 \text{ kN·m} \]

\[ \eta = \frac{M_{Ed}}{M_{c,Rd}} \leq 1 \]

\[ \eta : 0.091 \]

For positive bending:
\[ M_{Ed}^+ : \text{Worst case design bending moment.} \]

\[ M_{Ed}^+ : 0.00 \text{ kN·m} \]

For negative bending:
The worst case design force occurs at a point situated at a distance of 1.960 m from node N54, for load combination 1.35·SW+1.05·Q1+0.9·V4+1.5·N1.

\[ M_{Ed}^- : \text{Worst case design bending moment.} \]

\[ M_{Ed}^- : 0.79 \text{ kN·m} \]

The design bending moment resistance \( M_{c,Rd} \) is given by:
\[ M_{c,Rd} = W_{pl,z} \cdot f_{yd} \]

\[ M_{c,Rd} : 8.64 \text{ kN·m} \]
Where:

- **Class**: Section class, depending on its deformation capacity and development of plastic resistance of the flat elements of a section submitted to simple bending.
- **\( W_{pl,z} \)**: Plastic strength modulus corresponding to the fibre with greatest tension, for class 1 and 2 sections.
- **\( f_{yd} \)**: Steel design strength.
  - \( f_{yd} = f_y / \gamma_M \)
  - Where:
    - \( f_y \): Yield strength. (CTE DB SE-A, Table 4.1)
    - \( \gamma_M \): Partial safety factor of the material.

**Resistance to shear in the Z direction** (CTE DB SE-A, Article 6.2.4)

The following criteria must be satisfied:

\[
\eta = \frac{V_{Ed}}{V_{c,Rd}} \leq 1
\]

\( \eta \) : 0.005

The worst case design force occurs at a point situated at a distance of 1.960 m from node N54, for load combination 1.35·SW+1.05·Q1+1.5·V2+0.75·N1.

- **\( V_{Ed} \)**: Worst case design shear force.
- **\( V_{c,Rd} \)**: Shear resistance.

The shear resistance \( V_{c,Rd} \) is given by:

\[
V_{c,Rd} = A_v \cdot \frac{f_{yd}}{\sqrt{3}}
\]

Where:

- **\( A_v \)**: Transverse shear area.
- **\( d \)**: Height of the web.
- **\( t_w \)**: Web thickness.
- **\( f_{yd} \)**: Steel design strength.

The shear resistance \( V_{c,Rd} \) is given by:

\[
V_{c,Rd} = A_v \cdot \frac{f_{yd}}{\sqrt{3}}
\]

Where:

- **\( A_v \)**: Transverse shear area.
- **\( d \)**: Height of the web.
- **\( t_w \)**: Web thickness.
- **\( f_{yd} \)**: Steel design strength.
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Shear buckling of the web: (CTE DB SE-A, Article 6.3.3.4)
Even though transverse stiffeners have not been provided, it is not necessary to check the buckling resistance of the web, as the following is verified:

\[
\frac{d}{t_w} < 70 \cdot \varepsilon \quad \text{18.00} \times 64.71 \checkmark
\]

Where:
- \( \lambda_w \): Slenderness of the web.
- \( \lambda_w = \frac{d}{t_w} \) \( \lambda_w : 18.00 \)
- \( \lambda_{\text{max}} \): Maximum slenderness.
- \( \lambda_{\text{max}} = 70 \cdot \varepsilon \) \( \lambda_{\text{max}} : 64.71 \)
- \( \varepsilon \): Reduction factor.
- \( \varepsilon = \frac{f_{\text{ref}}}{f_y} \)

Where:
- \( f_{\text{ref}} \): Reference elastic limit.
- \( f_{\text{ref}} : 235.00 \text{ MPa} \)
- \( f_y \): Yield strength. (CTE DB SE-A, Table 4.1)
- \( f_y : 275.00 \text{ MPa} \)

Resistance to shear in the Y direction (CTE DB SE-A, Article 6.2.4)
The following criteria must be satisfied:

\[
\eta = \frac{V_{\text{Ed}}}{V_{c,Rd}} \leq 1 \quad \eta : 0.008 \checkmark
\]

The worst case design force occurs for load combination 1.35·SW+1.05·Q1+0.9·V4+1.5·N1.

- \( V_{\text{Ed}} : 0.73 \text{ kN} \)

The shear resistance \( V_{c,Rd} \) is given by:

\[
V_{c,Rd} = A_v \cdot \frac{f_{\text{yd}}}{\sqrt{3}} \quad V_{c,Rd} : 90.40 \text{ kN}
\]

Where:
- \( A_v \): Transverse shear area.
- \( A_v = A - 2 \cdot d \cdot t_w \) \( A_v : 5.98 \text{ cm}^2 \)

Where:
- \( A \): Area of the gross section.
- \( d \): Height of the web.
- \( t_w \): Web thickness.
- \( A : 11.74 \text{ cm}^2 \)
- \( d : 72.00 \text{ mm} \)
- \( t_w : 4.00 \text{ mm} \)
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$f_{yd}$: Steel design strength.

\[
  f_{yd} = \frac{f_y}{\gamma_m}
\]

Where:

- $f_y$: Yield strength. (CTE DB SE-A, Table 4.1)
- $\gamma_m$: Partial safety factor of the material.

\[
  f_y = 275.00 \text{ MPa}
\]

\[
  \gamma_m = 1.05
\]

Shear buckling of the web: (CTE DB SE-A, Article 6.3.3.4)

Even though transverse stiffeners have not been provided, it is not necessary to check the buckling resistance of the web, as the following is verified:

\[
  \frac{b}{t_r} < 70 \cdot \varepsilon
\]

Where:

- $\lambda_w$: Slenderness of the web.
- $\lambda_{max}$: Maximum slenderness.
- $\varepsilon$: Reduction factor.

\[
  \lambda_w = \frac{b}{t_r}
\]

\[
  \lambda_{max} = 70 \cdot \varepsilon
\]

\[
  \varepsilon = \frac{f_{ref}}{f_y}
\]

\[
  f_{ref} = 235.00 \text{ MPa}
\]

\[
  f_y = 275.00 \text{ MPa}
\]

Combined bending moment $Y$ and shear force $Z$ resistance (CTE DB SE-A, Article 6.2.8)

It is not necessary to reduce the design bending resistance, as the worst case shear force $V_{Ed}$ is not greater than 50% of the design shear resistance $V_{c,Rd}$.

\[
  V_{Ed} \leq \frac{V_{c,Rd}}{2}
\]

\[
  0.34 \text{ kN} \leq 43.55 \text{ kN}
\]

The worst case design forces occur for load combination 1.35·SW+0.9·V1+1.5·N1.

- $V_{Ed}$: Worst case design shear force.
- $V_{c,Rd}$: Design resistant shear force.

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Combined bending moment Z and shear force Y resistance (CTE DB SE-A, Article 6.2.8)

It is not necessary to reduce the design bending resistance, as the worst case shear force $V_{Ed}$ is not greater than 50% of the design shear resistance $V_{c,Rd}$.

$$V_{Ed} \leq \frac{V_{c,Rd}}{2} \quad 0.73 \text{ kN} \leq 45.20 \text{ kN}$$

The worst case design forces occur for load combination $1.35 \cdot SW + 1.05 \cdot Q1 + 0.9 \cdot V4 + 1.5 \cdot N1$.

- $V_{Ed}$: Worst case design shear force.
- $V_{c,Rd}$: Design resistant shear force.

Combined bending and axial resistance (CTE DB SE-A, Article 6.2.8)

The following criteria must be satisfied:

$$\eta = \frac{N_{t,Ed}}{N_{p,pl,Rd}} + \frac{M_{y,Ed}}{M_{p,pl,Rd,y}} + \frac{M_{z,Ed}}{M_{p,pl,Rd,z}} \leq 1 \quad \eta : 0.204$$

$$\eta = \frac{M_{y,Ed}}{M_{p,pl,Rd,y}} + \frac{M_{z,Ed}}{M_{p,pl,Rd,z}} \leq 1 \quad \eta : 0.158$$

The worst case design forces occur at a point situated at a distance of 1.960 m from node N54, for load combination $1.35 \cdot SW + 1.05 \cdot Q1 + 0.9 \cdot V4 + 1.5 \cdot N1$.

Where:

- $N_{t,Ed}$: Worst case design axial tensile force.
- $M_{y,Ed}$, $M_{z,Ed}$: Worst case bending moments, in accordance with the Y and Z axes, respectively.

Class: Section class, according to its deformation capacity and plastic resistance development of its flat elements, for axial load and simple bending.

- $N_{p,pl,Rd}$: Resistance to axial tension.
- $M_{p,pl,Rd,y}$, $M_{p,pl,Rd,z}$: Bending resistance of the gross section in plastic conditions, with respect to the Y and Z axes, respectively.

Buckling resistance: (CTE DB SE-A, Article 6.3.4.1)

- $M_{ef,Ed}$: Worst case design bending moment.
  $$M_{ef,Ed} = W_{y,com} \cdot \sigma_{com,Ed}$$

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Combined bending, axial and shear resistance (CTE DB SE-A, Article 6.2.8)

It is not necessary to reduce the design bending and axial force resistance, as the buckling effect can be ignored due to shear. Additionally, the worst case design shear force $V_{Ed}$ is less than or equal to 50% of the design shear resistance $V_{c,Rd}$.

The worst case design forces occur for load combination $1.35\cdot SW+1.05\cdot Q1+0.9\cdot V4+1.5\cdot N1$.

\[
V_{Ed,y} \leq \frac{V_{c,Rd,y}}{2}
\]

Where:
- $V_{Ed,y}$: Worst case design shear force.
- $V_{c,Rd,y}$: Design resistant shear force.

\[
0.73 \text{ kN} \leq 44.80 \text{ kN}
\]

Torsional resistance (CTE DB SE-A, Article 6.2.7)

The following criteria must be satisfied:

\[
\eta = \frac{M_{T,Ed}}{M_{T,Rd}} \leq 1
\]

\[
\eta : 0.009
\]

The worst case design force occurs for load combination $1.35\cdot SW+0.9\cdot V4+1.5\cdot N1$.

$M_{T,Ed}$: Worst case design torsional moment.

\[
M_{T,Ed} : 0.06 \text{ kN-m}
\]

The design torsional moment resistance $M_{T,Rd}$ is given by:

\[
M_{T,Rd} = \frac{1}{\sqrt{3}} \cdot W_T \cdot f_{yd}
\]

Where:
- $W_T$: Torsion resistance module.
- $f_{yd}$: Steel design strength.

\[
W_T : 46.21 \text{ cm}^3
\]

$M_{T,Rd} : 6.99 \text{ kN-m}$

\[
f_{yd} : 261.90 \text{ MPa}
\]

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\[ f_{yd} = f_y / \gamma_{M0} \]

Where:
- \( f_y \): Yield strength. (CTE DB SE-A, Table 4.1)
- \( \gamma_{M0} \): Partial safety factor of the material.

\[ f_y = 275.00 \text{ MPa} \]
\[ \gamma_{M0} = 1.05 \]

**Combined Z shear and torsional resistance** (CTE DB SE-A, Article 6.2.8)

The following criteria must be satisfied:

\[ \eta = \frac{V_{Ed}}{V_{pl,T,Rd}} \leq 1 \]
\[ \eta = 0.005 \checkmark \]

The worst case design forces occur at a point situated at a distance of 1.960 m from node N54, for load combination 1.35·SW+1.05·Q1+1.5·V2+0.75·N1.

- \( V_{Ed} \): Worst case design shear force.
- \( V_{Ed} = 0.44 \text{ kN} \)
- \( M_{T,Ed} \): Worst case design torsional moment.
- \( M_{T,Ed} = 0.05 \text{ kN-m} \)

The reduced design resistant shear force \( V_{pl,T,Rd} \) is given by:

\[ V_{pl,T,Rd} = \left[ 1 - \frac{\tau_{T,Ed}}{f_{yd} \sqrt{3}} \right] V_{pl,Rd} \]
\[ V_{pl,T,Rd} = 86.45 \text{ kN} \]

Where:
- \( V_{pl,Rd} \): Design resistant shear force.
- \( V_{pl,Rd} = 87.10 \text{ kN} \)
- \( \tau_{T,Ed} \): Tangential stresses due to torsion.
- \( \tau_{T,Ed} = 1.12 \text{ MPa} \)
- \( W_T \): Torsion resistance module.
- \( W_T = 46.21 \text{ cm}^3 \)
- \( f_{yd} \): Steel design strength.
- \( f_{yd} = 261.90 \text{ MPa} \)

\[ f_{yd} = f_y / \gamma_{M0} \]

Where:
- \( f_y \): Yield strength. (CTE DB SE-A, Table 4.1)
- \( \gamma_{M0} \): Partial safety factor of the material.
- \( f_y = 275.00 \text{ MPa} \)
- \( \gamma_{M0} = 1.05 \)

**Combined Y shear and torsional resistance** (CTE DB SE-A, Article 6.2.8)

The following criteria must be satisfied:

\[ \eta = \frac{V_{Ed}}{V_{pl,T,Rd}} \leq 1 \]
\[ \eta = 0.008 \checkmark \]

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The worst case design forces occur for load combination 1.35·SW+1.05·Q1+0.9·V4+1.5·N1.

\[ V_{Ed} \text{: Worst case design shear force.} \]
\[ M_{T,Ed} \text{: Worst case design torsional moment.} \]

The reduced design resistant shear force \( V_{pl,T,Rd} \) is given by:

\[
V_{pl,T,Rd} = \left[ 1 - \frac{\tau_{T,Ed}}{f_{yd}/\gamma_{M0}} \right] \cdot V_{pl,Rd}
\]

Where:

\( V_{pl,Rd} \text{: Design resistant shear force.} \)
\( \tau_{T,Ed} \text{: Tangential stresses due to torsion.} \)
\( \tau_{T,Ed} = \frac{M_{T,Ed}}{W_T} \)

Where:

\( W_T \text{: Torsion resistance module.} \)
\( f_{yd} \text{: Steel design strength.} \)
\( f_{yd} = \frac{f_y}{\gamma_{M0}} \)

Where:

\( f_y \text{: Yield strength. (CTE DB SE-A, Table 4.1)} \)
\( \gamma_{M0} \text{: Partial safety factor of the material.} \)

\[ V_{Ed} = 0.73 \text{ kN} \]
\[ M_{T,Ed} = 0.06 \text{ kN·m} \]
\[ V_{pl,T,Rd} = 89.59 \text{ kN} \]
\[ V_{pl,Rd} = 90.40 \text{ kN} \]
\[ \tau_{T,Ed} = 1.34 \text{ MPa} \]
\[ W_T = 46.21 \text{ cm}^3 \]
\[ f_{yd} = 261.90 \text{ MPa} \]
\[ f_y = 275.00 \text{ MPa} \]
\[ \gamma_{M0} = 1.05 \]