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Assessment of Robustness for Composite Steel-Concrete Frame Buildings

Dissertation of Master of Science in construction of Steel and Composite Structure Faculty advisor Professor Aldina Maria Cruz Santiago and Professor Ruis António Duarte Simões

Coimbra, September 2019

Faculdade de Ciências e Tecnologia da Universidade de Coimbra
Departamento de Engenharia Civil

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ASSESSMENT OF ROBUSTNESS FOR COMPOSITE STEEL-CONCRETE FRAME BUILDINGS

AVALIAÇÃO DA ROBUSTEZ DE EDIFICIOS MISTOS AÇO-BETÃO EM ESTRUTURA PORTICADA

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Summary

Robustness for buildings is a compilation of lessons learned from past experiences in the engineering world. The main purpose is always the same: to avoid the progressive collapse on buildings second to an accidental load like fire, explosion, impact or the consequence of a human error. This is why the construction norms for Europe, EN 1991 part 1-7, is focused in that matter. However, preparing a building to withstand the additional stress caused by these events, can make the final cost of the structure out of reach for the investor. This is what motivated the research at hand, so that we could design a structure, on top of the building, capable of redistributing the loads.

In order to reach this objective, a parametric numerical study was done, where two buildings were designed under three different conditions. The first one was a simple structural design following the consideration on the European norms for ultimate and serviceability limit state, used as a refence point. On the second one, the buildings were designed following the Eurocode 1 part 1-7 for accidental load, making the structure with enough redundancy in order to tolerate the stress applied. On the third one, the latter was applied by using a truss superstructure to redistribute the load. The solutions were compared based on the final weight and connection rigidity of the building. On both cases the building with the truss superstructure was verified for the accidental combination loads. However, the shortest one, when checked for normal conditions for the ULS and SLS, was not satisfactory, having to be redesigned for the additional weight of the truss superstructure.

Making a comparison of the final solution for both buildings, we arrived at the conclusion that the truss superstructure that was considered, helped the structural design. Even though for the shorter building the solution was not lighter, the connection rotational stiffness was considerably lower, whereas for the second building, both conditions were satisfactory.

Keyword

Robustness | Progressive collapse | Truss superstructure | Connections rigidity

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Contents

Symbols

$"+"$ implies "to be combine with"

Σ implies "to be combine with"

G_{kj} are the characteristic values of the permanent actions

A_d is the design value of an accidental action

$Q_{k,1}$ is the characteristic value of the leading variable actions

$Q_{k,i}$ are the characteristic values of the other variable actions

$\psi_{1,1}$ is the factor for the frequent value of the leading variable $Qk,1$

$\psi_{2,I}$ is the factor for the quasi-permanent value of the i-th variable action Qk,i

s is the spacing of ties.

L is the span of the tie.

ψ is the relevant factor in the expression for combination of action effects for the accidental design situation.

F_t is $60kNmor20 + 4ns\ kNm, which\ ever\ is\ less$

n_s is the number of stories

z is the lesser distance between ties

A The cross-sectional area in $mm2$ of the wall measured on plan, excluding the non-load-bearing leaf of a cavity wall.

H is the story height in meters.

k_i The stiffness coefficient for basic joint component i.

z The lever arm see figure 6.15 EN 1993-1-8.

μ The stiffness ration $Sj,ini/Sj$.

$F_{tr,Rd}$ The effective design resistance of a bolt-row r.

h_r The distance from bolt-row r to the center of compression.

r The bolt-row number.

d is the nominal diameter of the bolt.

f_{ub} is the yield strength of the relevant basic component.

f_y is the ultimate tensile strength of the bolt material.

h_c The depth of the column

h_b The depth of the beam

k_b The mean value of $Ib\ /\ Lb$ for all the beam at the top of that story.

k_c The mean value of $Ic\ /\ Lc$ for all the column in that story.

I_b The second moment of area of a beam.

I_c The second moment of area of a column.

L_b The spam of a beam (center-to-center of columns)

L_c The span of a column

$M_{b,Pl,Rd}$ The design plastic moment resistance of the beam.

$M_{c,Pl,Rd}$ The design plastic moment resistance of the column.

z The lever arm.

β_1 The value of transformation parameter β for the right-hand side joint.

β_2 The value of transformation parameter β for the left-hand side joint.

$M_{j,b1,Ed}$ The moment at the intersection from the right-hand beam.

$M_{j,b2,Ed}$ The moment at the intersection from the left-hand beam.

Vx Shear stress on the section

S Static moment of the homogenized concrete section in relation with the neutral elastic axis.

I The moment of inertia of the homogenized section.

q The longitudinal shear stresses.

γ_v Partial coefficient to consider material imperfections, use 1.25.

d Diameter of the shear stud.

f_u Ultimate tensile stress of the shear stud, less than $500\ N\ /\ mm2$

f_{ck} Ultimate compression tension for concrete at 28 days.

h_{sc} Total height of the shear stud

P_B The force due to the flexion of the stud

P_z The force due to the inclination of the stud

h_{sc} Total height of the shear stud

h_p Total height of the steel sheet

n_r The number of shear stud in the nerve, not higher than 2.

Where,

n The number of connectors

L_x Distance between the support and any point in the beam.

P_{Rd} Shera stud resistance.

N_c Force applied to the concrete.

$V_{L,Rd}$ Longitudinal shear design resistance.

$N_{c,f}$ The compression force resisted by the concrete flange.

R_c The maximum compression resistance of the concrete flange.

R_a The maximum tensile resistance of the steel section.

S_k Characteristic value of snow on the ground at a relevant site.

C_z Coefficient depending on the zone.

H Altitude of the local in meters

S Snow load on the roof.

μ_i Snow load shape coefficient.
c_e Exposure coefficient.
c_t Thermal coefficient.
q_p The peak velocity pressures.
q_{p20} The peak velocity pressures at a height of 20 meters.
q_{p32} The peak velocity pressures at a height of 32 meters.
q_{p40} The peak velocity pressures at a height of 40 meters.
$" + "$ Implies "to be combine with".
Σ Implies "to be combine with".
G_{kj} The characteristic values of the permanent actions.
$\gamma_{G,j}$ Partial factor for permanent action j.
P Relevant representative value of a prestressing action.
γ_p Partial factor for prestressing actions.
$Q_{k,1}$ The characteristic value of the leading variable actions.
$\gamma_{Q,1}$ Partial value factor for the leading variable action.
$Q_{k,i}$ The characteristic values of the accompanying variable actions i.
$\gamma_{Q,i}$ Partial value factor for the accompanying variable actions i.
$\psi_{o,i}$ The factor for combination for the accompanying variable action i.

k_s Coefficient that allows to take into consideration the effect of the reduction of the normal force of the concrete slab due to initial cracking and local slip of shear connection, which may be taken as 0.9

k_c Coefficient which considers of the stress distribution within the section immediately prior to cracking and is given by:

h_c The thickness of the concrete flange.

Z_0 The vertical distance between the centroids of the un-cracked concrete flange and the un-cracked composite section, calculated using the modular ration $n0$ for short term loading.

k Coefficient which allows for the effect of non-uniform self-equilibrating stress which may be taken as 0.8.

$F_{ct,eff}$ The mean value of the tensile strength of the concrete effective at the time when cracks may first be expected to occur. This value may be taken as $fctm$ from table 3.1 of NP EN 1992-1-1.

A_{ct} The area of the tensile zone immediately prior of the cracking of the cross section
σ_s The maximum stress permitted in the reinforcement immediately after cracking.
MPl, Rd The plastic resistance moment of the section.
VPl, Rd The plastic shear resistance of the section.

A_v The shear area of the section.
f_y The yield strength of the structural steel.
V_{Ed} The shear stress applied to the beam.
$M_{Ed,max}$ The maximum moment applied to the beam.
w The combination load applied to the beam.
L The length of the beam.
b_0 The distance between connectors.
b_{ei} The effective width of the concrete flange.
δ_{max} Maximum deformation allowed
L Beam length
u_i Overall horizontal displacement over the building height H
u Horizontal displacement over a story height Hi

1. Introduction

1.1. Relevance and scope

The construction industry has witness various improvements based on the knowledge of the materials, the construction method and the efficiency and cost of production of the structure. The composite structures are a perfect example of this evolution, where different materials are combined in order to take advantages of specific mechanical characteristics of each element. For this reason, steel structures, thanks to their high resistance to tensile stress, are often combined with concrete, which has high resistance to compression, allowing both materials to be used with a high efficiency. As a result, there are lightweight structural elements that can super pass large span. This, combined with other characteristics, has allowed the growth of steel structures use in the world.

The construction process of steel structures is made from standardized elements. The installation process is simple and has a big impact on the execution time and, furthermore, in the final cost of the construction. Besides, the steel structure is 100% recyclable making it very suitable to lessen the environmental impact.

For the structural design of a building, each country has established its own code. The European Union (EU) has the Eurocodes, which give the parameters on how the design should be conducted within the EU. This set of codes stablishes the Ultimate Limits States, when the structure is limited by the stress experienced in all the materials involved; and the Serviceability Limits State, when the condition of the structure is in service and involves other verifications like cracking, vibration, deformation, durability and overall stability. These limits are verified after submitting the building with different loads scenarios that are also summarized in the Eurocode.

The first part of the Eurocode EN 1990 makes a differentiation between the permanent load, the variable loads and the accidental loads on a building. The permanent load refers to the self-weight of structural and non-structural elements that are fixed in the building. The nominal density used for the structural elements is defined on the codes for each material and in the case of the non-structural elements, it can be defined by the provider.

The variable loads, according to Eurocode 0, are the ones whom the variation in magnitude with time is neither negligible nor monotonic; for example, snow, wind, thermal, imposed variable load. The imposed load varies according to the usage of the building. The environmental loads are defined in Eurocode 1 part 1-3, 1-4 and 1-5 and they depend on the shape, height and if is a composite structure or not, among others.

The accidental loads are actions of short duration but of significant magnitude, that are unlikely to occur on a given structure during the design working life; for example, fire, explosion, seismic (CEN, 2009). These loads are more complex when taken into consideration, because, they may never occur on the life span of the structure and there is not a way to determine the expected damage or probability of occurrence. They are represented in Eurocode 1 part 1-2 for fire, part 1-7 for impacts and explosion and Eurocode 8 part 1-1 for seismic action.

In this thesis, we will focus on the Eurocode 1 part 1-7 where “robustness” is defined as the ability of a structure to withstand events like fire, explosions, impact or the consequence of human error, without being damaged to an extent that is disproportionate to the original cause. This norm proposes two different approaches for the structural design: one through the identification and quantification of the accidental action (explosion, impact) and another one based on limiting the damage. In the case at hand, the design is going to be based on limiting the damages, which has three ways in which it can be achieved. The first one is to identify key elements in the structure on which its stability depends on an accidental design situation and design. The second one is to design the structure so that it’s stability won’t be affected in case there is a localized damage. The third one is using design rules in order to get enough robustness (for example, ductility of the elements, traditional tying to increase the integrity of the structure) (Way, 2011).

The study of robustness has been emphasized, revised and evolved due to a series of events around the world. In the document “Best Practices for Reducing the Potential for Progressive Collapse in Building” from the National Institute of Standards and Technology in the United States of America (Technology, 2007), there are five maior cases presented where the result of not taking into consideration this capacity on the structure lead to partial or total failure. These examples help to understand and give an introduction on how robustness pretend to enhance structural design.

In May 16th, 1968, in a 22-story building in the east side of London known as the Ronan Point Tower, an incident took place; the light of a match produced a gas explosion on the 18th floor caused a partial structural collapse. Thanks to this occurrence, the prevention of disproportionate collapse was introduced in the engineering world. In this event, one of the external walls was severely damaged and, with this, the lack vertical support for the floors above caused everything from the top to collapse and, due to the weight, all the floors underneath as well. According to investigations, this was due to the new construction methodology implemented at the time. It could be shown that it did not have structural integrity to withhold this type of situations. In other words, there was not an alternative path for the load to be distributed to other structural members of the building (Technology, 2007).

Figure 1-1: Ronan Point Tower partial collapse (Cook, 2018)

In July 17th, 1981, the Hyatt Regency Hotel's walkway collapse in Kansas City, Missouri, was another case study for the lack redundancy in the structure. In the original drawings there where two pavements, one on top of each other, hanging from the ceiling inside the lobby both using a continuous rod as support. In this case, due to a lack of communication between the project designer and the company in charge of the construction, this support was modified, and the lower pathway was set to hang directly from the upper one with a different rod. This doubled the stress in the structural element on the upper bridge causing a total collapse of the structure (Technology, 2007).

Figure 1-2 Hyatt Regency Hotel walkway collapse (MURPHY, 2014)

In April 23, 1987, the L'Ambiance Plaza in Bridgeport, Connecticut had a total collapse due to the insufficient reinforcement of the slab. This structure was using a different technique for the construction method known as the "Lift Slab Method". This consisted on having the slab cast on the ground one on top of each other and then they were lifted to its position with jacks installed on top of the columns. Many hypotheses exist of when and where the collapse started; however, it is clear, that it started with a local failure of the structure and the lack of resistance of the slab due to poor reinforcement for cracking, leading to the total collapse of the structure (Technology, 2007).

Figure 1-3 L'Ambiance Plaza total collapse (CARTER, 2018)

In April 19, 1995 a truck bomb was detonated on the north side of the Alfred P. Murrah Building in Oklahoma City, Oklahoma, that caused the partial collapse of the building. This was a terrorist attack aimed to harm a Unites Stated government office that was located there. This case, as opposed to the other ones, the structural design and detailing was according to the constructions codes that existed at the moment of erection; however, this building was not designed to sustain accidental loads like earthquake, blast, or any type of extreme loading because it was not mandatory. This was a nine-story reinforced concrete building with ordinary moment frames and, according to investigations, just by modifying the structure, to a moment frames like the one used in seismic design, would have limit the collapse to a 50% (Technology, 2007).

Figure 1-4 Alfred P. Murrah Federal Building partial collapse (Jenkins, 2019)

In a heavy snowstorm in the winter of 1996 the Jackson Landing Skating Rink came to a complete collapse in Durham, New Hampshire. This was a result of an unheated dome that, due to the low angle of inclination, stored a large amount of snow. This resulted in a local failure, given that one of the anchorages from one of the rods'bends failed suddenly. This was pre-engineered rigid frame structure of 64 m by 30.5 m open on the laterals. The roof structure was a metal-deck carried by Z-shape and C-shape purlins anchored to bents separated every 6.4 m, that resisted the lateral forces by cross-bracing cable on three non-sequential spans. On this day, the progressive collapse was due to overload of the purlin pulling down or by lateral instability of the structure. This case study is different from the others, since is not the typical vertical progressive failure, but a horizontal version of it, where open structure requires low resistance to lateral loads; therefore, it was not prepared to resist the sudden increase of load causing the instability and, eventually, a structure collapse.

Figure 1-5 Jackson Landing Skating Rink collapse (Technology, 2007)

These are five well documented and studied cases where the structural design did not took into consideration the possibility of a rapid increase or change of loads, where the buildings had a lack of structural integrity and there was not a redundancy in the structure or an alternative path for the load in case of an extreme condition. As a result, there was loss of human life and large amounts of money. Cases like the ones presented here have helped shape what is known today as robustness in the structure.

The robustness in steel structures have different ways of being assessed. According to Eurocode 1 part 1-7, one of them is the instantaneous loss of a column that helps to design a structure capable of having an internal redistribution of the loads to avoid a progressive collapse. The purpose of this method is to simulate the column loss due to a car collision, gas explosion, fire or malicious intent.

The main purpose of this thesis is the study of two different scenarios of a composite steel-concrete building to evaluate robustness through European norms, with the objective to propose an alternative load path due to the loss of a column. The first approach will be done by making the sections of the buildings resist the stress applying EN 1991 part 1-7. The second approach will be to use a truss superstructure to redistribute these loads. It will be done only by numerical verification supported by the Eurocode. The solution will be validated according the structural behavior of the building. To ensure the necessary redundancy of a structure, the cost becomes a major factor on the final decision. With this thesis, a comparison of the final weight for two different solutions will be presented, as well as the structural solutions and benefits.

2. Literature review: EN 1991-1-7

2.1. Introduction

In this chapter will be describe in extension the use of the Eurocode 1 part 1-7, for this review it will be necessary to outline other sections of the Eurocode (EN). It is important to point out that in this project the study will focus an office building.

2.2. Robustness design approach

The Eurocode, as was stated before, dedicated to take into consideration the structural robustness is the part 1-7 of the Eurocode 1. Where robustness is defined as "the ability of a structure to withstand events like fire, explosions, impact, or the consequences of human error, without being damaged to an extent disproportionate to the original cause" (CEN, 2006). This term can also be found in the Eurocode 0 and Eurocode 3. Is important to point out that every country may vary some of the parameters in this code through the National Annex; however, Portugal has not made any.

Design for robustness involves considering an accidental load and, according to Eurocode 0 section 3.2, This situation refers to an "exceptional conditions applicable to the structure or to its exposure, example to fire, explosion, impact or localized failure" (CEN, 2009). In other words, the building should be designed to ensure that a disproportionate collapse will not occur.

Figure 2-1 is a direct extraction from EN 1991 part 1-7 where the strategies for the structural design in accidental situations are described. The consideration will be taken in accordance with the client and relevant authority. The path should be chosen so the basis of structural projects are accomplished; specially what is stated in Eurocode 0, in section 2.1, where it specifies that the structure should be designed in a way that will not be damaged by explosion, impact, and the consequence of human error. Also, that the potential damage of a structure should be avoided or limited by appropriately selecting one of the following:

- Avoiding, eliminating or reducing the hazards to which the structure can be subjected;
- Selecting a structural form which has low sensitivity to the hazards considered;
- Selecting a structural form and design that can survive adequately the accidental removal of an individual member or limited part of the structure, or the occurrence of acceptable localized damage;
- Avoiding as far as possible structural systems that can collapse without warning;

- Tying the structural member together;

As it can be seen this is basically the definition for "robustness" describe earlier making a connection between the norms.

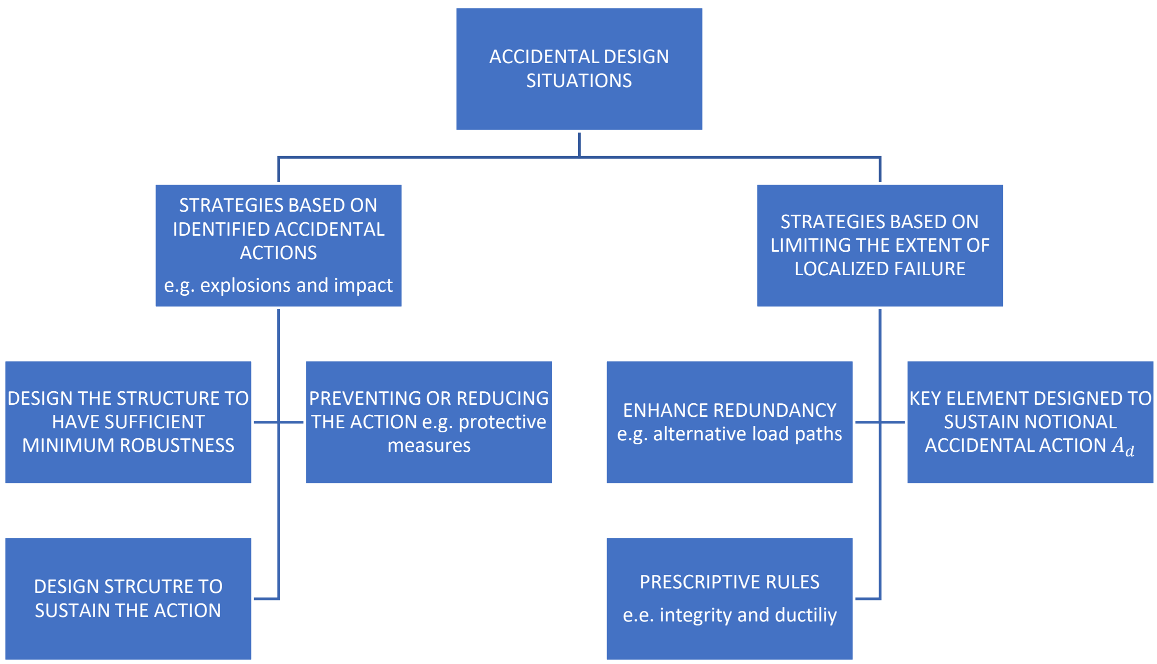


Figure 2-1 Strategies for accidental design situations (CEN, 2006)

In this situation the Eurocodes proposes a combination of loads when designing for Ultimate Limit State. This combination should be used when verifying for robustness, especially when taking the notional removal or key element method approach, that later will be explained.

$$\sum_{j\geq 1} G_{kj} \text{ "} + \text{" } A_d \text{ "} + \text{" } \psi_{1,1} Q_{k,1} \text{ "} + \text{" } \sum_{i\geq 1} \psi_{2,i} Q_{k,i} \qquad (2.1)$$

Where,

" + " *implies "to be combine with"*

Σ *implies "to be combine with"*

G_{kj} *are the characteristic values of the permanent actions*

A_d *is the design value of an accidental action*

$Q_{k,1}$ *is the characteristic value of the leading variable actions*

$Q_{k,i}$ *are the characteristic values of the other variable actions*

$\psi_{1,1}$ *is the factor for the frequent value of the leading variable* $Q_{k,1}$

$\psi_{2,I}$ *is the factor for the quasi-permanent value of the i-th variable action* $Q_{k,i}$

The value for the factor for the frequent and quasi-permanent value ψ are taken form table A1.1 from the Eurocode 0, shown here in Table 2-1. This value will depend on the category of use for the building and the type of load. Each country may vary this value in the National Annex, but, once again, Portugal does not make any changes on this.

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

Table 2-1 Recommended values of ψ factors for buildings (CEN, 2009)

The design rules and specific guidance analyzing steel structures are expressed in the Eurocode 3 part 1-1; however, there is no description for structural robustness. Here, in section 2.1.3, where the title is “Design working life, durability and robustness”, the parameters for the durability and the design work life are stablished and it states that the structure should be resistant to accidental action and then makes reference to the EN 1991 part 1-7.

2.2.1. Accidental design strategies for identified actions

The first strategy on Figure 2-1 is when the accidental load is identified. When starting to consider which accidental action to use when choosing the strategy of approach, the Eurocode 1, part 1-7 section 3.2, gives a pre-determine list of things to look for:

- the measures taken for preventing or reducing the severity of an accidental action;
- the probability of occurrence of the identified accidental action;
- the consequences of failure due to the identified accidental action;
- public perception;
- the level of acceptable risk.

These will help categorize the structure depending on the risk level. To design a building with no accidental-load consequences, is impracticable; for this reason, and in most cases, a certain level of risk will have to be accepted. It is important to emphasize that localized failure may be tolerable, if this will not jeopardize the stability of the construction, and the overall load-bearing capacity is maintained, allowing the necessary emergency measures to be taken.

Also, some actions should be considered to mitigate the risk of accidental actions. These can be done while making the structural design or by adding some protection to the elements. In the EN 1991 part 1-7 there are three considerations to select at least one when designing for robustness.

A. Prevent the action from occurring or reducing the probability and/or magnitude of the action to an acceptable level through the structural design process (CEN, 2006). This can be as simple as selecting a different paint for a steel structure to help the resistance to a fire in a building or providing with enough space in a bridge between the trafficked lanes and the structure.
B. Protecting the structure against the effects of an accidental action by reducing the effects of the action on the structure (CEN, 2006). Some example for this will be barriers to prevent car crashing on the structures when the construction is vulnerable.
C. Ensuring that the structure has enough robustness by adopting one or more of the following approaches:
    i. By designing certain components of the structure, upon which stability depends, as key elements, to increase the likelihood of the structure's survival following an accidental event (CEN, 2006);

ii. Designing structural members, and selecting materials, to have enough ductility capable of absorbing significant strain energy without rupture (CEN, 2006);
iii. Incorporating enough redundancy in the structure to facilitate the transfer of actions to alternative load paths following an accidental event (CEN, 2006).

These are the basis for a structural design when the action is known, the other approach is to design to limit the extent of localized failure.

2.2.2. Accidental design strategies for unidentified actions

This path is taken when the action is unknown, and the potential failure of the structure should be mitigated adopting one of the following methods:

A. Designing key elements, on which the stability of the structure depends, to sustain the effects of model of accidental action A_d (CEN, 2006);
B. Designing the structure so that in the event of a localized failure the stability of the whole structure or a significant part of it would not be endangered (CEN, 2006); for this, the norm imposes a limit of the minimum for acceptance of “localized failure” to 100 m^2 or 15% of the area of the floor, whichever is less.
C. Applying prescriptive design/detailing rules that provide acceptable robustness for the structure (CEN, 2006).

When taking this approach, the norm gives some recommendation of how to proceed in the Annex A of EN 1991 part 1-7. These recommendations are tied to the building consequences class that is assessed by the engineer. For this reason, when starting a project and making the decision of the strategy that will be adopted, all the entities involved should be part of it.

2.2.3. Use of consequence classes

In the EN 1990 gives each building a consequence class that it is used to design. This classification is based on the human life that can be loss and the social, economic or environmental consequence. This is resumed in table B1 of Annex B and it is represented in Table 2-2.

Consequences Class	Description	Examples of buildings and civil engineering works
CC3	**High** consequence for loss of human life, or economic, social or environmental consequences **very great**	Grandstands, public buildings where consequences of failure are high (e.g. a concert hall)
CC2	**Medium** consequences for loss of human life, economic, social or environmental consequences **considerable**	Residential and offices buildings, public buildings where consequences of failure are medium (e.g. an office building)
CC1	**Low consequences** for loss of human life, and economic, social or environmental consequences **small or negligible**	Agricultural buildings here people do not normally enter (e.g. storage buildings), greenhouses

Table 2-2 Definition of consequence class (CEN, 2009)

It is possible to considered different consequence class on different parts of the construction. Also, if some action is applied to minimize the risk of certain accidental load, for the design purpose, it might be considered to lower the consequence class, but the most appropriate thing to do is to reduce de forces applied to the structure. Therefore, when having the building characterized:

- CC1: No specific consideration is necessary for accidental actions except to ensure that the robustness and stability rules given in the norms, as applicable, are met (CEN, 2006);
- CC2: depending upon the specific circumstances of the structure, a simplified analysis by static equivalent action models may be adopted or prescriptive design/detailing rules may be applied (CEN, 2006);
- CC3: an examination of the specific case should be carried out to determine the level of reliability and the depth of structural analyses required. This may require a risk analysis to be carried out and the use of refined methods such as dynamic analyses, non-linear models and interaction between the load and the structure (CEN, 2006).

2.2.4. Impact

Impacts are considered an accidental action that it could be used in buildings. In the EN 1991 part 1-7 the following events are defined as impact in a structure:

- Impact from road vehicles (excluding collisions on lightweight structures);
- Impact from forklift trucks;
- Impact from trains (excluding collisions on lightweight structures);
- Impact from ships;
- The hard landing of helicopters on roof;

For buildings, action due to impact shall be considered for:

- Buildings used for car parking;
- Buildings in which vehicles or forklift trucks are permitted, and
- Buildings that are located adjacent to either road or railway traffic.

As it can be seen, the used of this type of load is very limited. This will be used in a higher extend when designing a bridge or structures where the interactions with vehicles is more often. As it was said before, for this project it is considered a conventional building and to elaborate on this topic is not considered necessary.

2.2.5. Internal explosions

The internal explosions are considered for buildings design when there is a part of the building exposed to gas, where there is any type of explosive materials, the possibility of any liquid forming explosive vapor or gas is stored or transported. EN 1991-1-7 does not cover the effect of the explosion or the cascade effects from several connected rooms filled with explosive gas, dust or vapor and is limited to the effect of the internal explosions.

Explosion is defined as the consequence of a rapid chemical reaction of dust, gas or vapor in air that results in a high temperature and overpressures response. The pressure created on structural members should consider the forces transmitted by the non-structural member. When measuring the pressure, there several parameters to contemplate, according to EN 1991 part 1-7:

- Type of dust, gas or vapor;
- The percentage of dust, gas or vapor in the air
- The uniformity of the dust, gas or vapor air mixture;
- The ignition source;
- The presence of obstacles in the enclosure;
- The size, shape and strength of the enclosure in which the explosion occurs;
- The amount of venting or pressure release that may be available.

These bounds will help to have a better estimation of the load that needs to be applied when designing. However, is very difficult to come with an exact number.

When analyzing a construction work classified as CC1, there is no specific consideration of the effect of the explosion, other than complying with what is stated in the norms about connections and interaction between components. However, for the ones classified as CC2 or CC3, one of the methods applied in Annex A and D should be considered. Additionally, for CC3 structures a dynamic analysis is required as well.

For the explosion, the design consideration, may allow the partial failure of the structure if it does not include key element where the stability of the structure can be threatened. There are some actions that can be taken in order to minimize the damage caused. Again, the Eurocode 1 part 1-7 in section 5.3 gives some measures that can be taken to control this:

- Designing the structure to resist the explosion peak pressure;
- Using venting panels with defined venting pressures;
- Separating adjacent sections of the structure that contain explosive materials;
- Limiting the area of structures that are exposed to explosion risks;
- Providing specific protective measures between adjacent structures exposed to explosions risks avoiding propagation of pressures.

When using venting panels additional consideration are important for the well-functioning of the system. For example, they should be located near the ignition sources, if known, or where pressures are high. Also, put in a location where it will not cause the expansion of the explosion, be restraint so it does not become a missile and threaten personnel. For these reasons, the recommendations are that they should be design by an expert.

2.2.6. Design for consequences of localized failure in buildings from an unspecified cause (Annex A)

This section gives all the design approaches for consequences of a localized failure in buildings from an unspecified cause, with the main purpose of limiting the extent of damage or failure. Making the structure sufficiently robust and avoiding disproportionate collapse. In this aspect, the building is only required to survive minimum amount of time that is needed to ensure the safe evacuation and rescue of personnel from the building and its surroundings. This may change for buildings used for handling hazardous materials, provision of essential services, or for national security reasons (CEN, 2006).

Also, the EN1991-1-7 gives a categorization to the consequence class that relates to the consequence class previously reviewed, see Table 2-3. At the same time, it states that a building with multiple categorization should be classified with the more onerous type and that in the case of a basement they may be included if they meet with the requirements of “Consequence Class 2 Upper Risk Group”. These will help to select the approach that should be taken when considering robustness in buildings.

Consequences Classes	**Example of categorization of building type and occupancy**
1	Single occupancy houses not exceeding 4 stories. Agricultural buildings. Buildings into which people rarely go, provided no part of the building is closer to another building, or area where people do go, than a distance of $1\ ^1/_2$ time the building height.
2a Lower Risk Group	5 story single occupancy house. Hotel not exceeding 4 stories. Flats, apartments and other residential buildings no exceeding 4 stories. Offices not exceeding 4 stories. Retailing premises not exceeding 3 stories of less than 1,000 m^2 floor area in each story. Single story educational buildings. All buildings not exceeding two stories to which the public are admitted, and which contain floor areas not exceeding 2,000 m^2 at each story.
2b Upper Risk Group	Hotels, flats, apartments and other residential buildings greater than 4 stories but not exceeding 15 stories. Educational buildings greater than single story but not exceeding 15 stories. Retailing premises greater than 3 stories but not exceeding 15 stories. Hospital not exceeding 3 stories. Offices greater than 4 stories but not exceeding 15 stories. All buildings to which the public are admitted, and which contain floor areas exceeding 2,000 m^2 but not exceeding 5,000 m^2 at each story. Car parking not exceeding 6 stories.

3	All buildings defined above Class 2 lower and Upper Consequences Class that exceed the limits on area and number of stories. All buildings to which members of the public are admitted in significant numbers. Stadia accommodating more than 5,000 spectators. Buildings containing hazardous substances and/or processes.

Table 2-3 Categorization of consequences classes (CEN, 2006)

2.2.6.1. Recommended strategies

The norm gives recommendations for the strategy to be adopted according to the categorization of the consequence class. This will help limiting the resources used in the structural design for buildings. Also, will allow to have a standardization in order to validate robustness. Following these, buildings will have an acceptable level of robustness to sustain localized failure without disproportionate level of collapse (CEN, 2006).

a) For buildings in Consequence Class 1:

For this classification there are not extra consideration needed; if, the structural design has followed what is stated in the other Eurocodes.

b) For buildings in Consequence Class 2a (Lower Group):

Beyond what is required for buildings class 1, the provision of effective horizontal ties, or effective anchorage of suspended floor to walls are for framed and load-bearing wall construction respectively. This will be explained in extension later.

c) For buildings in Consequence Class 2b (Upper Group):

In addition to the recommendation for Class 1 buildings, there are two other provisions, from which on shall be selected:

- Horizontal and vertical ties in all supporting columns and walls should be provided.
- The building should be checked to ensure that upon the notional removal of each supporting column and each beam supporting a column, or any nominal section of load-bearing wall (one at the time in each story of the building) the building stays stable and that any local damage does not exceed the limit that was established (CEN, 2006).

When the second strategy exceed the limits of damage in the structures, these elements should be designed as “key elements”. The design of these elements will be explained latter on this chapter.

d) For buildings in Consequence Class 3:

A systematic risk assessment of the building should be undertaken where foreseeable and unforeseeable hazards should be taken into consideration.

2.2.6.2. Horizontal ties

Framed structures

The horizontal ties are rolled steel section, steel bar reinforcement in concrete slabs, or steel mesh reinforcement and profiled steel sheeting in composite steel/concrete floors or a combination of two or more of the previous. They should be provided in order that all structural load-bearing section of the building are tied together and should be continuous and be arranged as closely as practicable to the edges of floors and lines of the column and walls, see Figure 2-2.

For the design of these ties and its end connections, should be able to absorb a tensile stress load for the accidental limit state. The internal and perimeter ties will have different loads depending on the following:

$- for\ internal\ ties,$

$$T_i = 0.8(g_k + \psi q_k)sL\ or\ 75kN, whichever\ is\ greater \tag{2.2}$$

$- for\ perimeter\ ties,$

$$T_p = 0.4(g_k + \psi q_k)sL\ or\ 75kN, whichever\ is\ greater \tag{2.3}$$

Where,

s *is the spacing of ties.*

L *is the span of the tie.*

ψ *is the relevant factor in the expression for combination of action effects for the accidental design situation.*

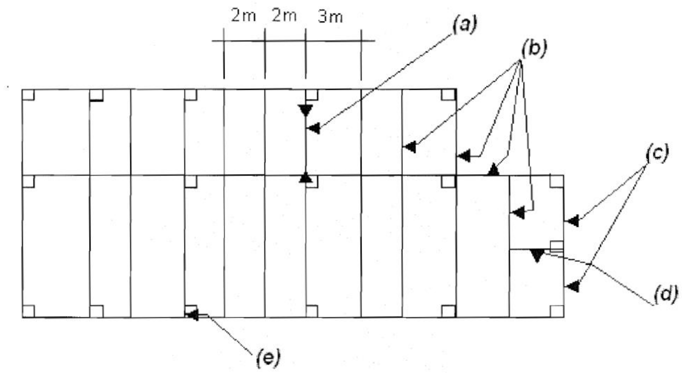


Figure 2-2 Example of horizontal tying of a 6 stories department store (CEN, 2006)

Key,

(a) 6 m span beam as internal tie
(b) All beams designed to act as ties
(c) Perimeter ties
(d) Tie anchored to a column
(e) Edge column

Load-bearing wall construction

For the categorization of the upper and lower consequence class 2 buildings the requirements for load bearing walls varies. For the lower class, robustness is provided by adopting a cellular form of construction designed to facilitate interaction of all components including an appropriate means of anchoring the floor to the walls (CEN, 2006). The main purpose for this is to ensure that the load will be transmitted through the structure in the design.

However, for upper consequence class these ties must be continuous. For this reason, the internal ties should be distributed throughout the floors in both orthogonal directions and peripheral ties, extending around the perimeter of the floor slabs within a 1.2 m width of the slab (CEN, 2006). The tensile stress for the design of the tie should be determined as follow:

$$For\ internal\ ties$$

$$T_i = \ the\ greather\ of\ F_t\ kN\ /\ m\ or\ \frac{F_t(g_k + \psi q_k)z}{7.5 \times 5} kN\ /\ m \qquad (2.4)$$

$$For\ peripheral\ ties\ T_p = F_t \qquad (2.5)$$

Where,

F_t $is\ 60kN|mor 20 + 4n_s\ kN/m, which\ ever\ is\ less$

n_s *is the number of stories*

z *is the lesser distance between ties*

- 5 time the clear story height H, or
- The greatest distance in meters in the direction of the tie, between the centers of the columns of other vertical load-bearing member whether this distance is spanned by:
  - A single slab or
  - By a system of beams and slabs.

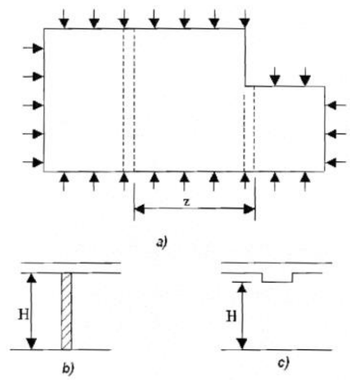


Figure 2-3 Illustration of factors H and z (CEN, 2006)

Key,

a) Plan
b) Section: Flat slab
c) Section: Beam and slab

2.2.6.3. Vertical ties

For the vertical ties it is recommended that, each column and wall, to be continuously from the foundations to the roof level. This is to guarantee a continuity of the path of the accidental load throughout the structure. When considering a framed building, the load-bearing elements should be able to sustain the accidental design tensile force that is equal to the largest design vertical permanent and variable load reaction applied to the column from any one story. When calculating the accidental design loading it should not be assumed to act simultaneously with permanent and variable actions that may be acting on the structure (CEN, 2006).

For this consideration to be considered effective, EN 1991 part 1-7 has some parameters that must be matched:

a) For masonry wall their thickness is at least 150 mm and if they have a minimum compressive strength of 5 $N\ /\ mm^2$ in accordance to EN 199-1-1
b) The clear height of the wall, H, measured in meters between faces of floor or roof does not exceed $20t$, where t is the thickness of the wall in meters.
c) If they are designed to sustain the following vertical tie force T

$$T = \frac{34A}{8000}\left(\frac{H}{T}\right)^2 N, or 100 KN/m of wall, whichever is greater, \qquad (2.6)$$

Where,

A *The cross-sectional area in* mm^2 *of the wall measured on plan, excluding the non-load-bearing leaf of a cavity wall.*

d) The vertical ties are grouped at 5 m maximum centers along the wall and occur no greater than 2.5 m from an unrestrained end of the wall.

2.2.6.4. Nominal section of load-bearing wall

The length for the nominal section of a load-bearing wall has some limitations according to EN 1991 part 1-7 and are as follow;

- For a reinforced concrete wall, a length not exciding 2.25 H.
- For an external masonry, timber or steel stud wall, the length measured between lateral supports provided by the vertical building components.
- For an internal masonry, timber of steel stud wall, a length not exciding 2.25 H.

Where,

H *is the story height in meters.*

2.2.6.5. Key elements

In this approach, the key elements, as it was stated for the strategy when designing for consequence class 2.b, should be capable of resisting an additional accidental load, A_d, applied in both direction to the element and any other attached, one at the time, taking into account the connection between them. This load should be applied using the combination for accidental load. The EN 1991 part 1-7 recommend the value of $A_d = 34\ KN\ /\ m^2$ (CEN, 2006).

2.2.7. Information on risk assessment (Annex B)

This section of the Eurocode 1 part 1-7 is dedicated to how to plan and execute risk assessment for civil engineering structures, see Figure 2-4 Overview of risk analysis . This will be done along with the design for consequences of localized failure for buildings that have a consequence class of 3. To understand this topic is important to define some concept that will be used throughout its development.

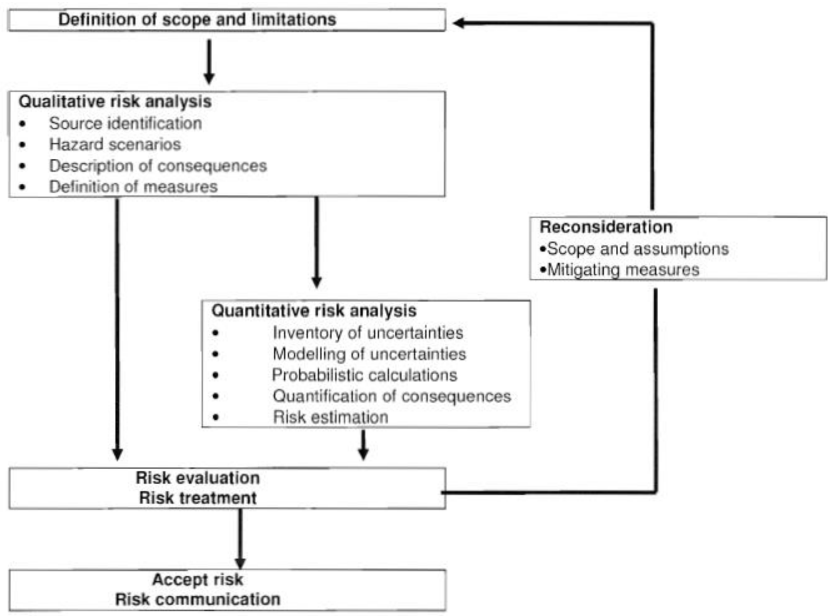


Figure 2-4 Overview of risk analysis (CEN, 2006)

These definitions are giving in the Annex B of the EN 1991 part 1-7 and are described next:

- Consequences: A possible result of an event. This can be express verbally or numerically as the economic loss, human loss, injuries, environmental damage, among others.
- Hazard scenario: A critical situation at a particular time consisting of a leading hazard together with one or more accompanying conditions which lead to an unwanted event. For example, the complete collapse of the structure.
- Risk: A measure of the combination of the probability or frequency of occurrence of a defined hazard and the magnitude of the consequences of the occurrence.
- Risk acceptance criteria: Acceptable limits to probabilities of certain consequences of an undesired event and are expressed in terms of annual frequencies. These criteria are normally determined by the authorities to reflect the level of risk considered to be acceptable by people and society.

- Risk analysis: A systematic approach for describing and/or calculating risk. Risk analysis involves the identification of undesired events, and the causes, likelihoods and consequences of these events.
- Risk evaluation: A comparison of the results of a risk analysis with the acceptance criteria for risk and other decision criteria.
- Risk management: Systematic measures undertaken by an organization in order to attain and maintain a level of safety that complies with defined objectives.
- Undesired event: An event of condition that can cause human injury, environmental or material damage.

2.2.7.1. Description of the scope of a risk analysis

To start the risk analysis all technical, environmental, organizational and human circumstances must be identified and detailed for further evaluation. Also, these tasks have a descriptive part, known as qualitative; and may, where relevant and practicable, also have a numerical part, known as quantitative.

Qualitative risk analysis

The most important task in the qualitative risk analysis is to identify all hazards and corresponding hazard scenarios. It requires a detailed examination and understanding of the system. For the importance of this step, some technics have been developed to help engineers when making the analysis.

In structural risk analysis some conditions that can present hazards to the structures and that have been listed in the Eurocode are:

- High values of ordinary actions.
- Low values of resistances, possibly due to errors or unforeseen deterioration.
- Ground and other environmental conditions different from those assumed in the design.
- Accidental actions like fire, explosion, flood, impact or earthquake.
- Unspecified accidental actions.

This list gives some specific cases to pay attention when doing a qualitive risk analysis in a structural design. However, it is not limit to only this, there might be others, but every case is different.

Also, the norm gives a list of some hazard scenarios that should be considered in the risk analysis, that are presented here; once again, in real life is not limit to only this.

- The anticipated or known variable actions on the structure.
- The environment surrounding the structure.
- The propose or known inspection regime of the structure.
- The concept of the structure, its detailed design, materials of constructions and possible points of vulnerability to damage or deterioration.
- The consequences of type and degree of damage due to the identified hazard scenario.

Quantitative risk analysis

The quantitative risk analysis the probabilities for all undesired events and their subsequent consequences should be estimated. This is usually based on judgment and, for this reason, it might differ from actual failure frequencies. When the risk can be express numerically it can be presented as the mathematical expectation of the consequences of an undesired vent. The EN 1991-1-7 has a possible way of presenting the quantitative analysis in figure B.2a, in this project is Figure 2-5. The risk analysis can be terminated at any time depending on the following:

- The objective of the risk analysis and the decisions to be made
- The limitations made at an earlier stage in the analysis.
- The availability of relevant or accurate data.
- The consequences of the undesired events.

An important aspect of the risk analysis that at any point, when relevant information is gather, the assumptions made must be revised for the more effectiveness of the method.

Severe	X				
High	X				
Medium		X			
Low			X		
Very low				X	
consequence probability	0,00001	0,0001	0,001	0,01	> 0,1
X represents examples of maximum acceptable risk levels					

Figure 2-5 Possible presentation diagram for the outcome of a quantitative risk analysis (CEN, 2006)

This diagram helps classify every hazard scenario into Severe, High, Medium, Low or Very Low the potential of failure of the risk identified. Where Severe is that the building can have a sudden collapse, resulting on the loss of life and injury and Very Low are local damages of small importance. This will be done by estimations by the engineer.

2.2.7.2. Risk acceptance and mitigating measures

The next step, after identifying the level of risk, is deciding whether the risk is acceptable or mitigating measures should be taken. For this, the ALARP (as low as reasonably practicable) principle is used, where risk can be above or below the ALARP region. When the risk falls below or the tolerable region no measures are need it, in the other hand, the risk most be mitigated. When the risk is between these two boundaries an economical optimal solution should be sought.

When the risk is accepted, they should be classified by the following criteria:

- ➢ The individual acceptance of the risk, which is presented as fatal accident rates.
- ➢ The socially acceptable level of risk, which is presented as F-N curve, for probability of F having an accident versus N the number of casualties.

The acceptance criteria may come from national regulation or requirements, certain codes and standards, or from experience and/or theorical knowledge that may be used as a basis for decisions on acceptable risk. They also can be expressed qualitatively or numerically.

The EN 1991 part 1-7 have some criteria that need to be meet for qualitative risk analysis:

a) The general aim should be to minimize the risk without incurring a substantial cost penalty.
b) For the consequence within the vertically hatched area of Figure 2-6, the risks associated with the scenario can normally be accepted.
c) For the consequences within the diagonally hatched area of Figure 2-6, a decision on whether the risk of the scenario can be accepted and whether risk mitigation measures can be adopted at an acceptable cost should be made.
d) For the consequence considered to be unacceptable, or the horizontally hatched area of Figure 2-6, appropriate risk mitigation measures should be taken.

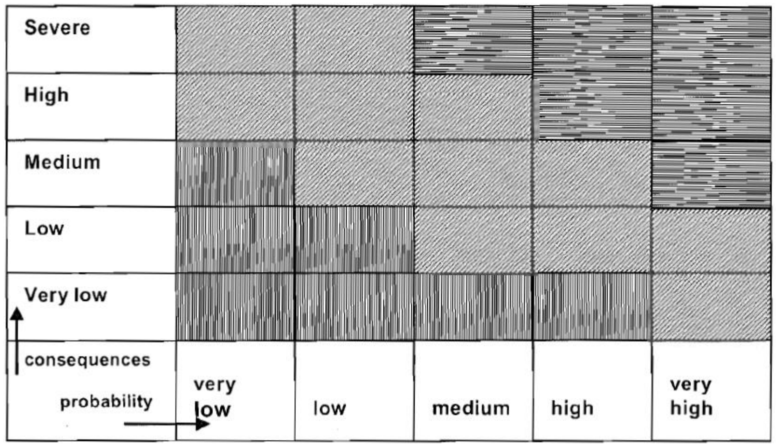


Figure 2-6 Possible presentation diagram for the outcome of a qualitative risk analysis (CEN, 2006)

2.2.7.3. Risk mitigation measures

The norm, once again, gives a list of mitigating measures that might be used for the mitigation measures and one or more can be used.

a) Elimination or reduction of the hazard.
b) By-passing the hazard by changing the design concepts or occupancy.
c) Controlling the hazard.
d) Overcome the hazard.
e) Permitting controlled collapse of the structure where the probability of injury or fatality may be reduced.

2.2.7.4. Application to buildings and civil engineering structures

This section gives a summary of the application for robustness for buildings and civil engineering structures in order to mitigate risk for extreme events. For the structural measures, recommends that the members are designed in a way to have reserves of strength or an alternative load path in case of local failures. For non-structural measures, assorts the reduction of the probability of the event occurring, the strength of the action or the chance of failure.

For the structural design the probabilities and effects of all accidental and extreme action happening at the same time should be considered. In these cases, the consequences should be presented in terms of number of casualties and economic losses. This approach has its complicity when considering unforeseeable hazards. For this reason, the more global damage tolerance design previously described (key elements design, horizontal and vertical ties, notional removal) should be used.

Structural risk analysis

The approach for the structural risk analysis due to accidental action can follow the sequences in Figure 2-7.

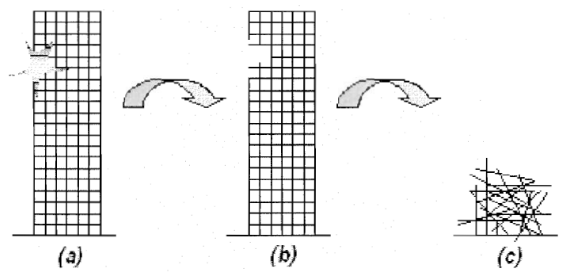


Figure 2-7 Illustration of steps in risk analysis of structures subject to accidental actions (CEN, 2006)

Key,

- Step 1: Identification and modelling of relevant accidental hazards. Assessment of the probability of occurrence of different hazards with different intensities.
- Step 2: Assessment of damage states to structures from different hazards. Assessment of the probability of different states of damage and corresponding consequences for given hazards.
- Step 3: Assessment of the performance of the damaged structure. Assessment of the probability of inadequate performance of the damaged structure together with the corresponding consequence.

The total risk R can be assessed by:

$$R = \sum_{i=1}^{N_H} P(H_i) \sum_{j}^{N_D} \sum_{k=1}^{N_S} P(H_i)\, P(D_j|H_i) P(S_k|D_j) C(S_k) \qquad (2.7)$$

Where is assumed that the structure is subjected to N_H different hazards, that the hazards may damages the structure in N_D different ways (can be dependent on the considered hazards) and that the performance of the damages structure can be discretized into N_S adverse states S_K with corresponding consequences $C(S_k)$. $P(H_i)$ is the probability of occurrence (within a reference time interval) of the i^{th} hazard, $P(D_j|H_i)$ is the conditional probability of j^{th} damage state of the structure given the i^{th} hazard and $P(S_k|D_j)$ is the conditional probability of the k^{th} adverse overall structural performance S given the i^{th} damage state.

Also, when performing a risk analysis, different strategies for the risk control and the risk reduction have to be investigated for economic feasibility:

- The risk may be reduced by reducing the probability that the hazards occurs, $P(H_i)$. For example, removing explosive materials from buildings, can reduce the probability of an explosion happening inside.
- The risk may be reduced by reducing the probability of significant damages for given hazards, $P(D_j|H_i)$. For example, using passive or active fire control for the structure.
- The risk may be reduced by reducing the probability of adverse structural performance given structural damage, $P(S_k|D_j)$. For example, designing the structure with enough redundancy.

2.2.8. Dynamic design for impact (Annex C)

The EN 1991 part 1-7 dedicates Annex C to dynamic design for impact in structures. Where impact is defined as an interaction between a moving object and a structure, in which the kinetic energy of the object is suddenly transformed into energy of deformation. For this phenomenon to be study, the mechanical properties of both elements must be determined. When designing for this event, equivalent static forces are commonly used.

Advanced design of structures to sustain actions due to impact may include explicitly one or several of the following aspects:

- Dynamic effects.
- Non-linear material behavior.

However, since this does not form part of the study that is been worked in this thesis, this will not be detailed.

2.2.9. Internal explosions (Annex D)

When taking into consideration internal explosions, the Eurocode 1 part 1-7 annex D gives recommendation for the opening area in dust explosions in rooms, vessels and bunkers. Also, helps calculate the pressure caused by natural gas explosions in buildings and explosions in road and rail tunnels. Once again, this will not be the scoop of this investigation, for this reason, this will not be expanded in this project.

3. Structural connections

In this section will be explained the relevant elements for structural connections. This will include beam-to-beam, beam-to-column and concrete-to-steel connection, since all become responsible for the re-distribution of the load when considering robustness in a structure. For these, multiple sources will be used and referenced in this work.

3.1. Beam-to-column and beam-to-beam connections

According to Eurocode 3 part 1- 8, a connection is where two or more elements meet. This represent the point where internal forces and moment are transfer and when considering robustness, since the stress amplified, is a critical point when designing. There are multiples element that need to be defined in order to have a better understanding of a connection.

In Figure 3-1, there are two type of beam-to-column configurations, single-sided and double-sided joint. Also, it is point out the relevant part of the connection that represent the mechanical and rotational resistance of the connection. Both of this drawing are joints in the strong axis of the elements.

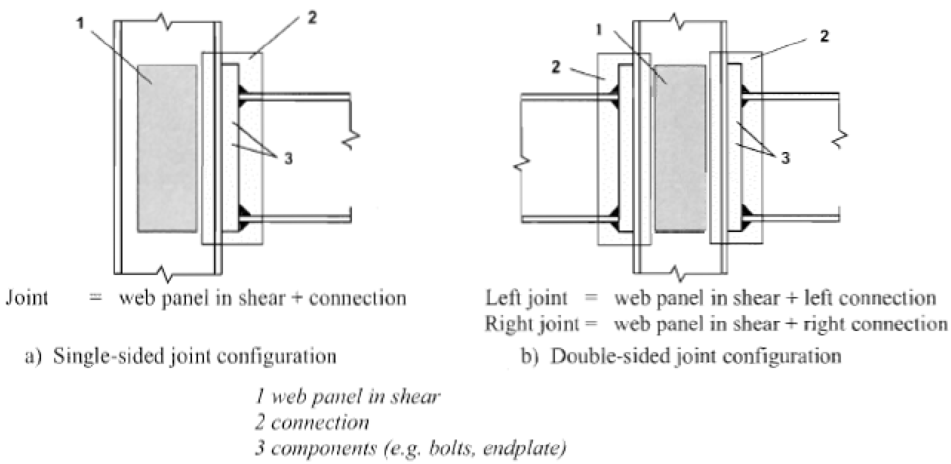


Figure 3-1 Parts of a beam to column joint configuration (CEN, 2005)

In a steel structures joints can be found in different places, see Figure 3-2. This can happen for multiple reasons, for example: limits in the fabrication of the elements, transportation problems, viability when erecting the structures, among others. For these reasons, in this project the focus will be beam-to-column in the strong axis, as was shown before, and beam-to-beam in the weak axis. The connection will be design to resist the strength of the forces applied and the rotational capacity.

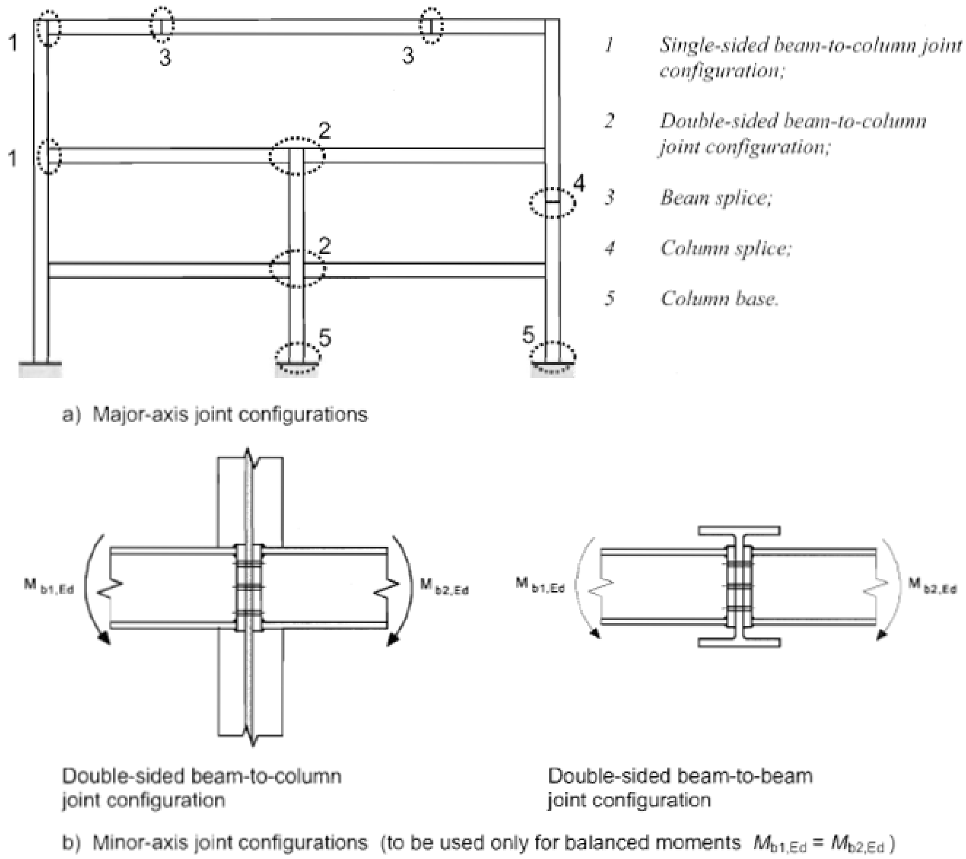


Figure 3-2 Joint configuration (CEN, 2005)

3.1.1. Analysis, classification and modeling of joints

There are three distinctions that need to be made when modeling a joint, to consider the effect of its behavior on the distribution of internal forces and on the structure.

- Simple, in which the joint may be assumed not to transmit bending moments.
- Continuous, in which the behavior of the joint may be assumed to have no effect on the analysis.
- Semi-continuous, in which the behavior of the joint need to be taken into account in the analysis.

These peculiarities will have different implication in the structure depending on the global analysis method used and the classification of the joint. The Eurocode 3 part 1-8, helps on the selection of the modeling using Table 3-1. The will be explained latter in this chapter.

Method of global analysis	Classification of joint		
Elastic	Nominally pinned	Rigid	Semi-rigid
Rigid-Plastic	Nominally pinned	Full-strength	Partial-strength
Elastic-Plastic	Nominally pinned	Rigid and full-strength	Semi-rigid and partial-strength Semi-rigid and full-strength Rigid and partial-strength
Type of joint model	Simple	Continuous	Semi-continuous

Table 3-1 Type of joint model (CEN, 2005)

3.1.1.1. Elastic global analysis

When using a global elastic analysis, the joints will have to be classified in function of its rotational stiffness in nominally pinned, rigid or semi-rigid. Also, they will have to be able to resist the nominal stress from the structural analysis (CEN, 2005).

If a semi-rigid joint is used, the rotational stiffness S_j corresponding to the bending moment $M_{j,Ed}$ should be used in the analysis. If $M_{j,Ed}$ does not exceed ${}^{2}/_{3}\ M_{j,Rd}$ the initial rotational stiffness $S_{j,ini}$ may be taken in the global analysis, see Figure 3-3 (a) (CEN, 2005).

However, the norm has a simplification where the rotational stiffness can be considered equal to $S_{j,ini}\ /\ \eta$, for all the values of $M_{j,Ed}$, as it can be seen in Figure 3-3 (b). The coefficient for the stiffness modification η can be found in Table 3-2 (CEN, 2005).

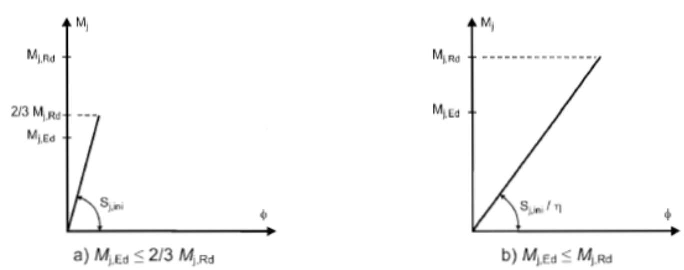


Figure 3-3 Rotational stiffness to be used in elastic global analysis (CEN, 2005)

Type of connection	Beam-to-column joints	Other types of joints (beam-to-beam joints, beam splices, column base joints)
Welded	2	3
Bolted end-plates	2	3
Bolted flange cleats	2	3,5
Base plates	-	3

Table 3-2 Stiffness modification coefficient (CEN, 2005)

For joint of element type H or I the value of S_j is taken based on the flexibility of their basic components, each one represented with an elastic stiffness coefficient, k_i, obtained in section 6.3.2, table 6.11 in EN 1993 part 1-8. This does not apply for column base, since in this project this will not be part of the study, column base connection will not be emphasized.

$$S_j = \frac{Ez^2}{\mu \sum_i \frac{1}{k_i}} \quad (3.1)$$

Where,

k_i *The stiffness coefficient for basic joint component* i.

z *The lever arm see figure 6.15 EN 1993-1-8.*

μ *The stiffness ration* $S_{j,ini}/S_j$.

The stiffness ratio μ should be determined as follow:

$- if\ M_{j,Ed} \leq 2\ /\ 3M_{j,Rd}$

$$\mu = 1 \quad (3.2)$$

$- if\ 2\ /\ 3M_{j,Rd} < M_{j,Ed} \leq M_{j,Rd}$ (3.3)

$$\mu = \left(1.5M_{j,Ed}\ /\ M_{j,Rd}\right)^{\psi}$$

Where ψ is obtain from

Type of connection	ψ
Welded	2,7
Bolted end-plate	2,7
Bolted angle flange cleats	3,1
Base plate connections	2,7

Table 3-3 Value of the coefficient ψ (CEN, 2005)

3.1.1.2. Rigid-plastic global analysis

For rigid-plastic global analysis, the joints will have to be classified according to its resistance, this will be explained latter. In the case of section type H or I, $M_{j,Rd}$ will be equal to:

$$M_{j,Rd} = \sum_{r} h_r \, F_{tr,Rd} \tag{3.4}$$

Where,

$F_{tr,Rd}$ *The effective design resistance of a bolt-row r.*

h_r *The distance from bolt-row r to the center of compression.*

r *The bolt-row number.*

To calculate the rotational capacity for section type H or I when using a rigid-plastic analysis are:

Bolted joint

For a beam-to-column connection where the design resistance moment $M_{j,Rd}$ is limited by the resistance of the web plate of the column to shear stress, can be considered to have a rotational capacity for a plastic analysis is $d_{wc} \,/\, t_w \leq 69\varepsilon$.

In the case of a bolted joint with an endplate or an angle flange may be assumed to have enough rotation capacity for a plastic analysis, as long as it satisfies the following:

- The design moment resistance of the joint is governed by the design resistance of either:
  - The column flange in bending.
  - The beam endplate or tension flange cleat in bending.

- The thickness t of either the column flange or the beam endplate or tension flange cleat (not necessarily the same basic component as in (a)) satisfies:

$$t \leq 0.36\, d \sqrt{f_{ub} \,/\, f_y} \qquad (3.5)$$

Where,

d *is the nominal diameter of the bolt.*

f_{ub} *is the yield strength of the relevant basic component.*

f_y *is the ultimate tensile strength of the bolt material.*

For a joint where the design resistance moment $M_{j,Rd}$ is limited by the shear resistance of the bolt, can't be considered as having enough rotational capacity for the plastic analysis.

Welded joints

For welded beam-to-column joints where the column web is reinforced for compression and not for tension, and the resistant moment is not limited due to shear stress of the web of the column, the rotational capacity of the joint can be considered not less than:

$$\phi_{Cd} = 0.025 h_c \,/\, h_b \qquad (3.6)$$

Where,

h_c *The depth of the column*

h_b *The depth of the beam*

Although, if the joint fulfills what is stated in this section and is welded but not reinforced, it can be considered that it has a rotational capacity ϕ_{Cd} not less than 0.015 radian.

All these considerations are verified only for steel type S 235, S 275 and S 355 and for joints where the design value of axial force N_{Ed} in the connected members does not exceed 5% of the design plastic resistance $N_{pl,Rd}$ of its cross section. Also, this verification is not necessary if $M_{j,Rd}$ of the joint is at least 1.2 times the design plastic moment resistance $M_{pl,Rd}$ of the cross section of the connected members.

In case of hollow section, it is necessary to review section 7 of the EN 1993 part 1-8. Since this connection is not critical in the project, this will not be emphasized.

3.1.1.3. Elastic-plastic global analysis

When using an elastic-plastic global analysis for the structure, the joint will have to be classified by their stiffness and resistance. In this case the same methods previously mentioned will be used, for the $M_{j,Rd}$ will be the same as it was mentioned in section 3.1.1.2 of this text, the value of S_j will be the same as mentioned in section 3.1.1.1 and ϕ_{Cd} will be found the same way as mention in section 3.1.1.2. Also, for hollow section the method explained in section 7 of the Eurocode 3 part 1-8 will be used, this part will not be detailed since is not a critical part of the project.

The design of the joint for this type of analysis has a bi-linear simplification for the relation of the moment and rotation of the joint represented in Figure 3-4. Where η will be taken from Table 3-3.

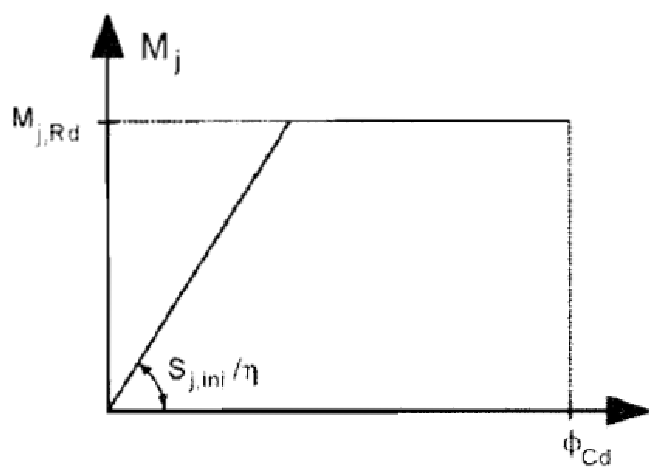


Figure 3-4 Simplified bi-linear design moment-rotation characteristic (CEN, 2005)

3.1.1.4. Global analysis of lattice girders

3.1.2. Joint classification

As it was stated before, all joints can be classified according to their stiffness or to their resistance and these joints must respect the assumptions made on the analysis method chosen, without adversely affecting any other part of the structure.

Classification according to its stiffness

A joint will be classified as rigid, nominally pinned or semi-rigid according to its rotational stiffness. For this, the initial rotational stiffness $S_{j,ini}$ must be compare with the limits indicated in Figure 3-5. The joint can be catalogized by calculation as it was explained in the previous section or by experimental results.

A nominally pinned joint should be capable to transmit the internal forces, without creating significant moment that can adversely affect the structure. This type of joint should be able to accept the rotation imposed by the load combination. For a rigid joint, should have enough rotational stiffness to guarantee a full continuity for the analysis. If the joint cannot be qualified as neither of these types, the joint will be considered semi-rigid.

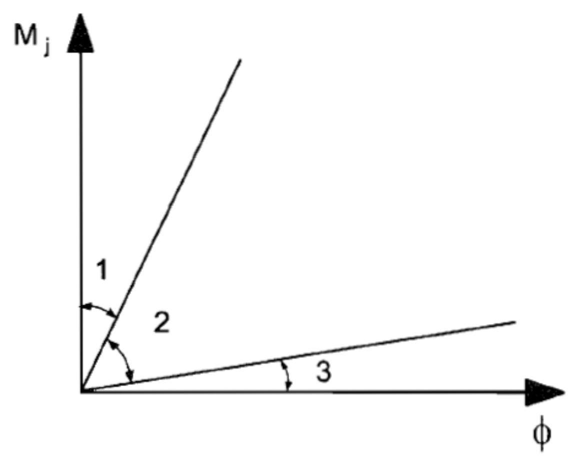


Figure 3-5 Classification of joint by stiffness (CEN, 2005)

Zone 1: rigid, if $S_{j,ini} \geq \frac{k_b E I_b}{L_b}$

Where,

$k_b = 8$ for frame where the bracing system reduces the horizontal displacement by at least 80%.

$k_b = 25$ for other frames, provided that in every story $k_b \ / \ k_c \geq 0.1$.

Zone 2: semi-rigid

All joints in zone 2 should be classified as semi-rigid. Joints in Zone 1 or 3 may optionally also be treated as semi-rigid.

Zone:3 nominally pined, if $S_{j,ini} \geq \frac{0.5EI_b}{L_b}$

Where,

k_b *The mean value of* I_b / L_b *for all the beam at the top of that story.*

k_c *The mean value of* I_c / L_c *for all the column in that story.*

I_b *The second moment of area of a beam.*

I_c *The second moment of area of a column.*

L_b *The spam of a beam (center-to-center of columns)*

L_c *The span of a column*

The base of a column can be classified as well but since is not part of this investigation will not be detailed.

Classification according to its resistance

When classifying a connection due to its resistance, it will be based on making a comparison between is resistance moment, $M_{j,Rd}$, with the resistance moment of the element joint. They can be arranged into three main groups:

Nominally pined joints

This type of joints should be able to transmit the internal forces of the structure without creating and adverse moment that can affect other members. Joints that its design resistance is not greater than 25% of a full-strength joint and that has sufficiently rotation will be considered pin.

Full strength joints

For full strength joints, the design resistance cannot be less than the resistance of the element connected. For a joint to be classified as full strength must meet the criteria stablished in Figure 3-6. Where the moment resistance of the connection, $M_{j,Rd}$, should be greater than the moment resistance of the beam, M_{b,Pl,R_d}, or the moment resistance of the column, M_{c,Pl,R_d}, when the connection is on the top of the column, or $2M_{c,Pl,R_d}$ when the connection is within the column height.

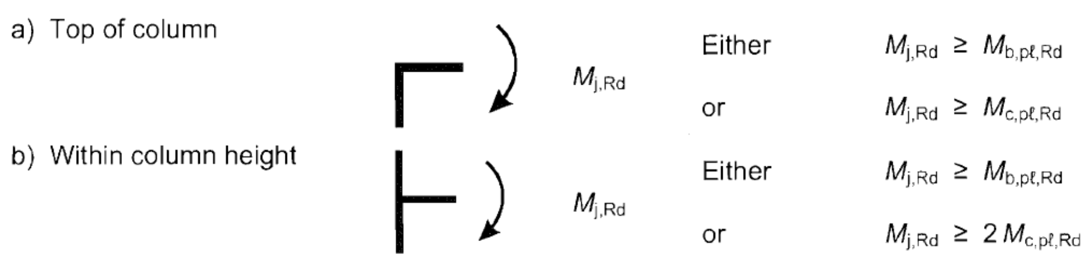


Figure 3-6 Full strength joints (CEN, 2005)

Where,

$M_{b,Pl,Rd}$ *The design plastic moment resistance of the beam.*

$M_{c,Pl,Rd}$ *The design plastic moment resistance of the column.*

Partial strength joints

When the joint does not follow on any of the criteria before mentioned, the joint will be considered with partial strength.

3.1.3. Modeling beam-to-column connection

When modeling a beam-to-column connection, the deformation due to shear stress and the rotational deformation should be considered. The joint should be designed to resist the shear, normal and moment stress applied as shown in Figure 3-7.

The resulting shear force applied to the web of the column in a connection should be calculated according to:

$$V_{wp,Ed} = \frac{\left(M_{b1,Ed} - M_{b2,Ed}\right)}{z} - \frac{\left(V_{c1,Ed} - V_{c2,Ed}\right)}{2} \tag{3.7}$$

Where,

z *The lever arm.*

For simplification, in order to get more truthfully results from the joint, two different models will have to be done, one for the shear stress acting on the web panel of the column and a second one with the resultant of the forces acting on the beams, see Figure 3-7 (a) and Figure 3-8 (a). Also, in a beam-to-column connection, with a beam in one side only, can be model as a single joint, and a connection with two beams can be model as two single beam model, but with interacting joints, on each side. As a result, a double-side beam-to-column joint configuration has two moment-rotation characteristics, one for each side, having to model two different rotational mole, see Figure 3-9 (CEN, 2005).

When calculating the moment resistance and the rotational stiffness of the connection, the influence of the web panel summited to shear will be considered through the transformation parameters β_1 and β_2, where:

β_1 *The value of transformation parameter* β *for the right-hand side joint.*

β_2 *The value of transformation parameter* β *for the left-hand side joint.*

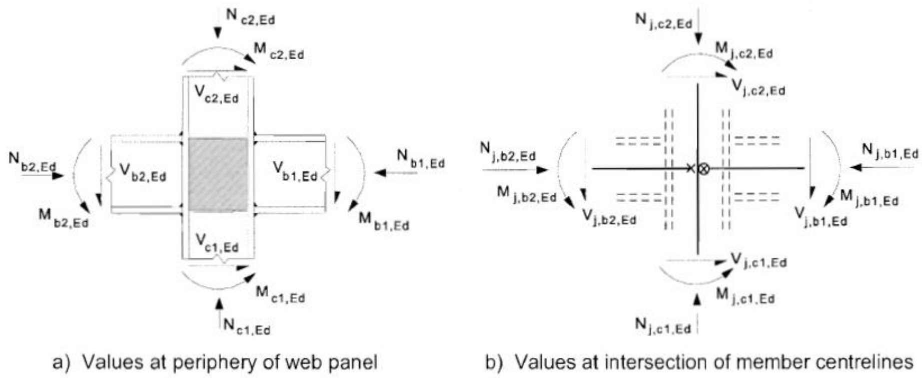


Figure 3-7 Forces and moments acting on a joint (CEN, 2005)

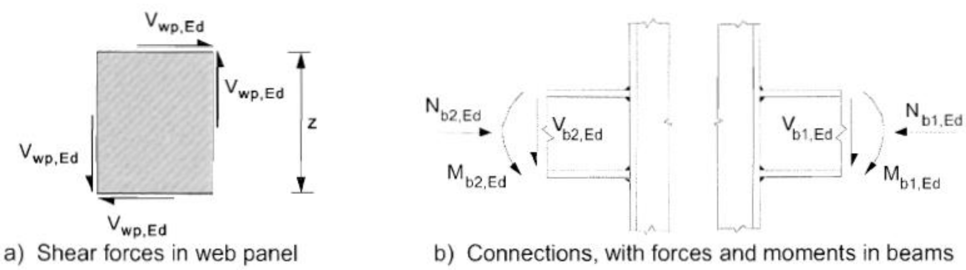


Figure 3-8 Force and moment acting on the web panel at the connections (CEN, 2005)

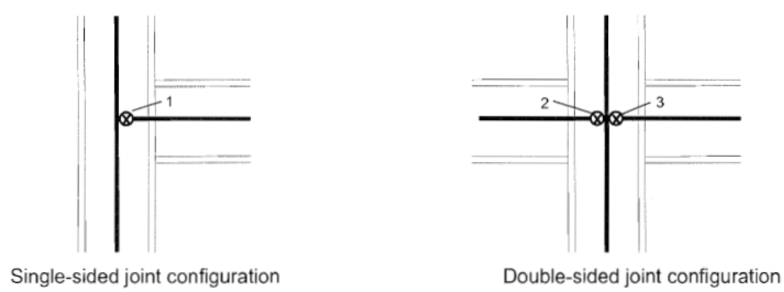


Figure 3-9 Modelling of the joint (CEN, 2005)

Where,

1. Joint
2. Joint 2: left side
3. Joint 2: right side

In Table 3-4 Approximate values for the transformation parameter β there are approximate values for β_1 and β_2. However, for more accurate values can be found following the next equations. These equations will be mandatory in the case of joint where the beam do not have the same designs.

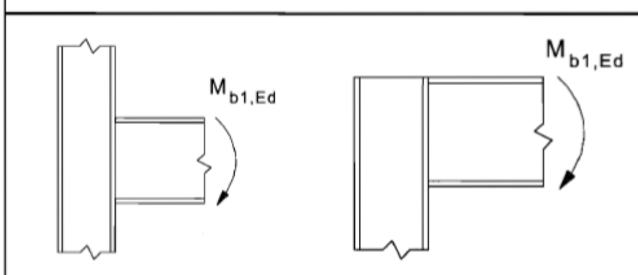
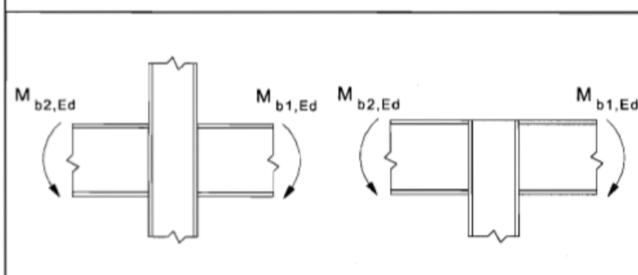
$$\beta_1 = \left|1 - \frac{M_{j,b2,Ed}}{M_{j,b1,Ed}}\right| \leq 2 \qquad (3.8)$$

$$\beta_2 = \left|1 - \frac{M_{j,b1,Ed}}{M_{j,b2,Ed}}\right| \leq 2 \qquad (3.9)$$

Where,

$M_{j,b1,Ed}$ *The moment at the intersection from the right-hand beam.*

$M_{j,b2,Ed}$ *The moment at the intersection from the left-hand beam.*

Type of joint configuration	Action	Value of β
	$M_{b1,Ed}$	$\beta \approx 1$
	$M_{b1,Ed} = M_{b2,Ed}$	$\beta = 0$ *)
	$M_{b1,Ed} / M_{b2,Ed} > 0$	$\beta \approx 1$
	$M_{b1,Ed} / M_{b2,Ed} < 0$	$\beta \approx 2$
	$M_{b1,Ed} + M_{b2,Ed} = 0$	$\beta \approx 2$
*) In this case the value of β is the exact value rather than an approximation.		

3

Table 3-4 Approximate values for the transformation parameter β (CEN, 2005)

After considering these, the resistance of the connection must be calculated taking into consideration the bolts and the distribution. To do so, what is established in EN 1993 part 1-8 section 6 have to be followed, this will not be explain since what is important for this case study is the rotational stiffness more than the plastic resistance of the connection.

3.2. Steel-to-concrete connection

The connection between the steel and the concrete is important to guarantee the re-distribution of the forces in the structure. This connection can be ignored, where the two materials work separately; partial, where only a percentage of the interaction is considered; or total, where the two materials work together to create a composite element with better mechanical characteristics, see Figure 3-10. For robustness this can be a method to tie the elements among each other and will help with the deformation of the structure.

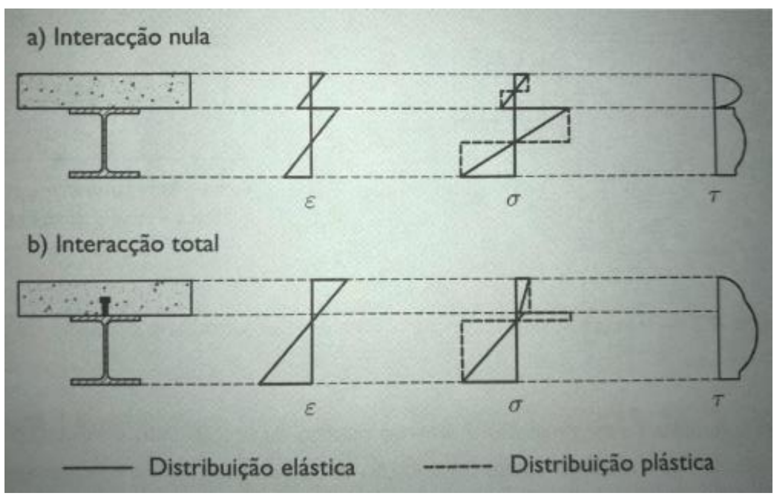


Figure 3-10 Composite beam deformation with full interaction and no interaction between materials (LUIS CALADO, 2015)

3.2.1. Longitudinal shear stress between concrete and steel

When calculating the longitudinal stress of the connection between the concrete and the steel, and considering that it has a total interaction between the two materials has a lineal-elastic behavior, the longitudinal shear stress, q, can be determine following:

$$q(x) = \frac{V(x)S}{I} \tag{3.10}$$

Where,

$V(x)$ *Shear stress on the section*

S *Static moment of the homogenized concrete section in relation with the neutral elastic axis.*

I *The moment of inertia of the homogenized section.*

q *The longitudinal shear stresses.*

However, this will be true if the beam does not exceed its elastic moment. When this happens, the distribution of the longitudinal shear stress is no longer lineal-elastic and the beam develops plasticity in the mid-span, where the forces applied to the connectors will increase (LUIS CALADO, 2015).

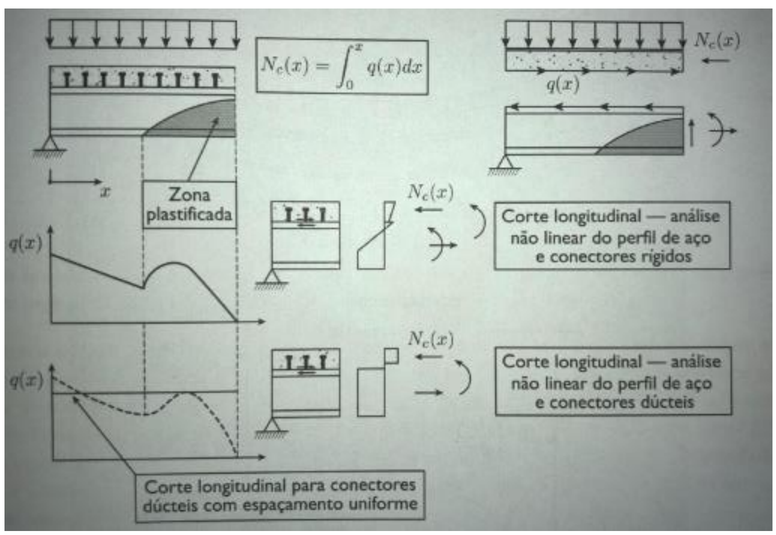


Figure 3-11 Variation of the longitudinal shear stress, q, according to the span of the beam (LUIS CALADO, 2015).

When designing a building, it can be considered that the connectors are ductile and, because of the capacity of the redistribution of the forces for the longitudinal shear, it can be admitted a uniform shear force throughout the beam and equal spacing between connectors.

Shear stud connectors

The shear stud connectors have an ultimate tensile strength resistance between $450\ N\ /\ mm^2$ and $600\ N\ /\ mm^2$, even though the norm does not allow resistance higher than $500\ N\ /\ mm^2$, and a geometrical characteristics with diameters between 13 and 25 mm and height between 75 and 150 mm. Using these values, the shear resistance on a solid slab for the stud can depend on the resistance of the concrete or the resistance of the stud using the following equations (CEN, 2009):

$$P_{Rd} = \min(P_{Rd,1}; P_{Rd,2}) \tag{3.11}$$

Where,

$$P_{Rd,1} = \frac{0.8 f_u \pi \frac{d^2}{4}}{\gamma_v} \tag{3.12}$$

Or

$$P_{Rd,2} = \frac{0.29\alpha\, d^2 \sqrt{f_{ck} E_{cm}}}{\gamma_v} \tag{3.13}$$

Where,

γ_v *Partial coefficient to consider material imperfections, use 1.25.*

d *Diameter of the shear stud.*

f_u *Ultimate tensile stress of the shear stud, less than* $500\ N\ /\ mm^2$

f_{ck} *Ultimate compression tension for concrete at 28 days.*

h_{sc} *Total height of the shear stud*

Where α will be calculates as:

$$\alpha = 0.2\left(\frac{h_{sc}}{d} + 1\right)\ for\ 3 \leq \frac{h_{sc}}{d} \leq 4 \tag{3.14}$$

$$\alpha = 1\ for\ \frac{h_{sc}}{d} > 4 \tag{3.15}$$

According to the Eurocode 4 part 1-1, the shear stud, when in between the parameters of height and diameter before mentioned, can be considered ductile when their total height after the weld is not smaller than 4 times its diameter. Also, sets a limit to the minimum height of the stud to be 3 times its diameter after the welding (CEN, 2009).

However, when a composite slab it's considered with a profiled steel sheet, the resistance for the shear stud will minimized depending on the geometry of the sheet, see Figure 3-12. On the case of a trapezoidal steel sheet, the steel stud will present a behavior like the one presented in Figure 3-13. Where the stud has its ultimate tensile strength when the material gets plasticized on the core. This will only be possible if the stud is, at least two times its diameter, greater than the height of the steel sheet.

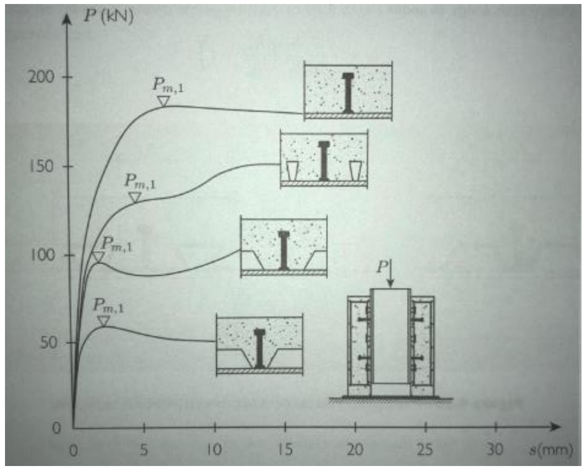


Figure 3-12 Shear stud resistance according to the geometry of the steel sheet (LUIS CALADO, 2015)

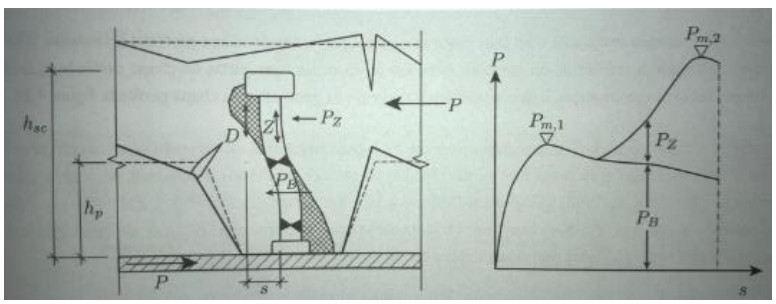


Figure 3-13 Shear stud behavior in a trapezoidal steel sheet (LUIS CALADO, 2015)

Where,

P_B — *The force due to the flexion of the stud*

P_z — *The force due to the inclination of the stud*

h_{sc} — *Total height of the shear stud*

h_p — *Total height of the steel sheet*

After several experimental tests for different type of steel sheet it was concluded that the resistance of the shear stud considering the steel sheet profile, $P_{i,Rd}$, will be equal to the one calculated for a solid slab, multiply for a reduction factor, k_i, that depends on orientation of the nerves, parallel or transvers to the beam.

$$P_{i,Rd} = k_i P_{Rd} \tag{3.16}$$

Where,

- For the nerves are parallel to the beam

$$k_l = 0.6\frac{b_0}{h_P}\left(\frac{h_{sc}}{h_p} - 1\right) \leq 1.0 \tag{3.17}$$

- For the nerves are transverse to the beam

$$k_t = \frac{0.7}{\sqrt{n_r}}\frac{b_0}{h_P}\left(\frac{n_{sc}}{h_p} - 1\right) \tag{3.18}$$

Where,

n_r *The number of shear stud in the nerve, not higher than 2.*

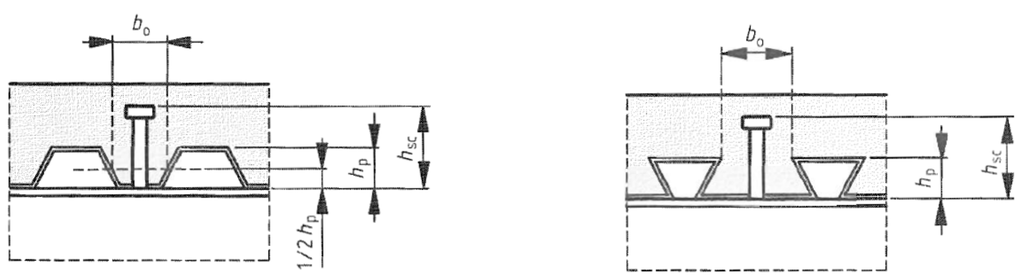


Figure 3-14 Steel sheet geometrical components (CEN, 2009)

Partial/total shear force connection

When calculating the resistance moment of a composite section, M_{Rd}, it is related to the resistance force of the longitudinal shear, $V_{L,Rd}$, and concomitant to the number of connector and the respective force strength, P_{Rd}. In a composite beam the moment applied depends on the load and the support condition, the resistance moment depends, among others, on the connection since this will condition the force applied to the concrete, N_c. As a result, the longitudinal shear design resistance will be equal to the number of connectors times the resistance force of the connector in a distance in the bema, and this will have to be equal or greater than the compression force applied to the concrete.

$$N_c \leq V_{L,Rd} = nL_x P_{Rd} \tag{3.19}$$

Where,

n *The number of connectors*

L_x *Distance between the support and any point in the beam.*

P_{Rd} *Shera stud resistance.*

N_c *Force applied to the concrete.*

$V_{L,Rd}$ *Longitudinal shear design resistance.*

If the connection between the two materials, $N_{c,f}$, is considered total and the neutral plastic axis is in the concrete, means that $R_c \geq R_a$. In this case $N_{c,f} = R_a$. In the other hand, if the neutral plastic axis is in the steel, $R_a > R_c$ and $N_{c,f} = R_c$. As a result, the value for the compression force on the concrete flange will be equal to (LUIS CALADO, 2015):

$$N_{c,f} = \min(R_c; R_a) \tag{3.20}$$

Where,

$N_{c,f}$ *The compression force resisted by the concrete flange.*

R_c *The maximum compression resistance of the concrete flange.*

R_a *The maximum tensile resistance of the steel section.*

As a result, if $N_c = N_{c,f}$ the connection between the elements will be considered total, in the other hand, if $N_c < N_{c,f}$ the connection will be considered partial. The ratio between these two components will represent the degree of connection, η (LUIS CALADO, 2015).

4. Structural design of buildings: two case studies

For this project, two different type of buildings where selected to be designed following what it is described in the Eurocodes. For each case, there will be three situations so the results can be compared at the end. On the first model, the building will be designed without taking into consideration robustness. It will be a simple design to Ultimate Limited State and Serviceability Limit State. The second model will be taking into consideration robustness with the removal of key elements. In this case, the key elements considered will be the columns. The third model will use the first solution and combine it with a truss structure in the top, that will allow it to resist the notional removal of key elements.

This chapter will present the definition of the materials, structural layouts, actions, action combinations and verification of the limit states and the linear elastic analysis.

4.1. Parametric values and analysis case

4.1.1. Parametric values

The parametric values where selected in order to have buildings with similar characteristics to the everyday erected. For both situations the designs where set as seen in Table 4-1. To help make a fair comparison with the final solutions, some parameters were set fixed in all cases.

The floor system was considered to have a composite response only for the secondary beams. The primary beams where designed to be able to resist the loads during the construction phase, without using any propping device. The column base connections where considered fixed allowing rotation on the X direction on the global coordinates of the structure. The beam to column connections that belong on the moment resistant frame where considered as totally rigid on the X direction and the beam to column connection, on the Y direction, where considered perfectly pinned. The building will be considered to have a moment resistance frame (MRF) on the X direction and a braced system on the Y direction.

Each building will be designed three times, the first time following the normal regulation for a project design, the second time using EN 1991 part 1-7 with the removal of the critical element and a third time using a truss superstructure to withstand the additional stress from the removal of the columns.

Parametric	Building 1	Building 2
Floor height	4 m	4 m
Moment resistance frame span	6 x 8 m	4 x 8 m
Braced frame span	6 x 5 m	4 x 5 m
Lateral force design scenario	Wind	Wind
Concrete	C30/37	C30/37
Steel	S355	S355
Utilization	Office	Office
Nomenclator		
Building Normal Condition	1-1	2-1
Building following EN 1991-1-7	1-2	2-2
Building with truss superstructure	1-3	2-3

Table 4-1 Parametric consideration for buildings type

4.1.2. Analysis case

The structures will be analyzed for the loss of the column taking only into considerations the wind load for horizontal load. For each building, the most critical situation was identified by removing one by one column on the first floor of the structure; since, here the elements had a greater rate of utilization.

4.2. Actions

For the permanent action and the live load for the building it was used the parameters established on the EN 1990 and the EN 1991 part 1-1. The snow and wind where considered following the recommendations from EN 1991 part 1-3 and EN 1991 part 1-4 respectively. Identified accidental loads where not taken into consideration and will not be part of this evaluation.

The primary beam was not considered to work as a composite member in order to be able to resist the stresses for the construction phase, at ultimate limit state and serviceability state. This will allow a faster construction process.

4.2.1. General actions for live load

Buildings with a utilization for office, fit in category type B. This will allow to select the standardization load applicable for the design of this construction. A summary of table 6.1 from the EN 1991 part 1-1 is given in this thesis in Table 4-2. Acknowledging this, the load used for this categorization are summarized in Table 6.2 from the Eurocode and can also be seen in Table 4-2. For the design a load due to the building utilization of 3 $KN\ /\ m^2$ will be used.

Category	Specific Use	Load (KN/m^2)
A	Area for domestic and residential activities	2 to 3
B	Office areas	2 to 3
C	Areas where people may congragate	It has a sub category 2 to 7.5
D	Shopping areas	4 to 5

Table 4-2 Category of use (CEN, 2009)

The roof of the building can be considered of three types according to the specific use that will have. The first one, is for roofs that cannot be access except for normal maintenance and repair. The second one is for roofs that are accessible with an occupancy that can be categorize along with the other floors. The last classification is for roof that will have special used like the landing of a helicopter, see Table 4-3. In this case a category type H was used where the recommendations from the Eurocode is to use a load of 0.40 $KN\ /\ m^2$.

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

Table 4-3 Categorization of roofs (CEN, 2009)

4.2.2. General action for permanent load

For the structural element, annex A for the EN 1991 part 1-1 stablish the recommended values. As a result, we proceeded to select these values to take into consideration in the structural design of the structure, see Table 4-4.

Materials	Density $\gamma(KN\ /\ m^3)$
Concrete	24.0
Structural steel	78.5

Table 4-4 Construction materials density (CEN, 2009)

Materials	Density $\gamma(KN\ /\ m^2)$
Composite slab	2.17

Table 4-5 Structural element load

4.2.3. General action for permanent non-structural elements

For the non-structural element what is recommended is to ask for technical properties to the distributer to have a more accurate value. For this project, the values where selected following annex A of the EN 1991 part 1-1.

Elements	$q(kN\ /\ m^2)$
Floor covering	1.40
Services	0.40
False ceiling	0.12
Division walls	1.00
Σ	2.92

Table 4-6 Non-structural element load

4.2.4. Other permanent action considered

To have a more realistic design a glass facade was simulated. This element has a density of $25\ KN\ /\ m^3$ according to annex A of the EN 1991-1-1. A covering for the ceiling was also considered.

Element	Load
Glass faced	$2.5\ KN\ /\ m$
Roof covering	$2.00\ KN\ /\ m^2$

Table 4-7 Other permanent action considered

4.2.5. Snow load

When considering the snow action, the EN 1991 part 1-3 has a national annex that must be considered when calculating the load due to snow. In the national annex for Portugal the country is divided into three zone, making Coimbra part of zone one with a coefficient, C_z, equal to 0.3. Also, it gives an equation, see equation 4.1, to calculate the characteristic value of the snow on the ground. This equation is based on the altitude of the site.

$$S_k = C_z\left[1+\left(\frac{H}{500}\right)^2\right] = 0.3\left[1+\left(\frac{499}{500}\right)^2\right] = 0.5988\ KN/m^2 \tag{4.1}$$

Where,

S_k *Characteristic value of snow on the ground at a relevant site.*

C_z *Coefficient depending on the zone.*

H *Altitude of the local in meters*

After calculating the characteristic value of snow on the ground, the norm recommends calculating the snow load for persistent/transient design situation with the following equation (CEN, 2003). This equation has some coefficient that allows to take into consideration the shape, exposure and thermal conditions of the roof. In table 5.1 of the EN 1993 part 1-1 have the recommended value for the exposure, and the thermal coefficient; that are 0.8 and 1.0 respectively.

$$S = \mu_i c_e c_t S_k = (0.8)(1.0)(1.0)(0.5988) = 0.479KN\ /\ m^2 \tag{4.2}$$

Where,

S *Snow load on the roof.*

μ_i *Snow load shape coefficient.*

c_e *Exposure coefficient.*

c_t *Thermal coefficient.*

4.2.6. Wind load

For the calculation of the wind action, the EN 1991-1-4 has a national annex that must be considered when calculating the wind pressure on buildings. For the project the mean wind velocity due to the height was elaborated taken into consideration the location of the building and other the parameters of the norm. This will not be explained in great detailed since is not relevant for the investigation, only the calculation will be shown. Since we have two buildings with different designs and the area of application depends on this, the pressure due to the wind was stablished separately.

Knowing the wind mean velocity we proceeded to calculate the value for the pressure created using the height of the buildings as reference. Since the geometry of the buildings differ from one another two different approach were taken. In Figure 4-1 the sign convention for the pressures is shown.

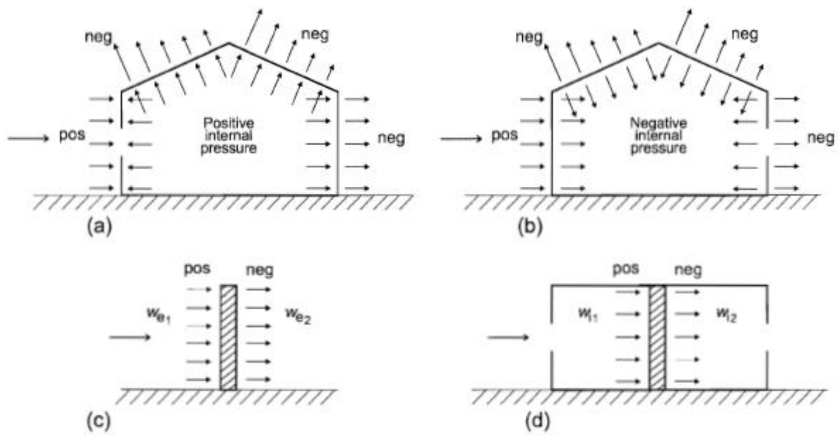


Figure 4-1 Sing convention for pressure due to the wind load (CEN, 2010)

4.2.6.1. Building 1 wind load

Since this building height is less than any of the longitudinal or transversal dimensions, only one pressure was needed it at the building height of 20 m, and this was used for both wind in the X or Y direction. Using these parameters, the mean wind velocity graphic was developed, see Figure 4-2. For modeling purpose, the wind in the X direction will be considered acting in the same direction as the MFR and the Y will be acting perpendicular.

1-Wind pressures			**2.2-Roughness factor**		
1.1-Basic parameters acc. National Annex			z_0 =	0.30	m (Tabela 4.1)
$v_{b,0}$ =	27.00	m/s	z_{min} =	8.00	m (Tabela 4.1)
c_{dir} =	1.00	(recommended)	k_r =	0.215	
c_{season} =	1.00	(recommended)	**3-Orography Coefficient**		
ρ =	1.25	kg/m^3	ϕ =	0	(figura A.1 or A.2)
1.2-Basic wind velocity EN 1991-1-4 (4.1)			S =	0	(figura A.2 or A.3)
v_b =	27.00	m/s	c_o =	1	
1.3-Basic Velocity Pressure			**4-Wind turbulance**		
q_b =	455.63	N/m^2	K_I =	1	(Recommended)
2-Wind force			**5-Exposure coefficeint**		
2.1-Terrain category (Table 4.1)			Peak factor=	3.50	
Category:	III				

Table 4-8 Calculation for the wind pressure building 1 (CEN, 2010)

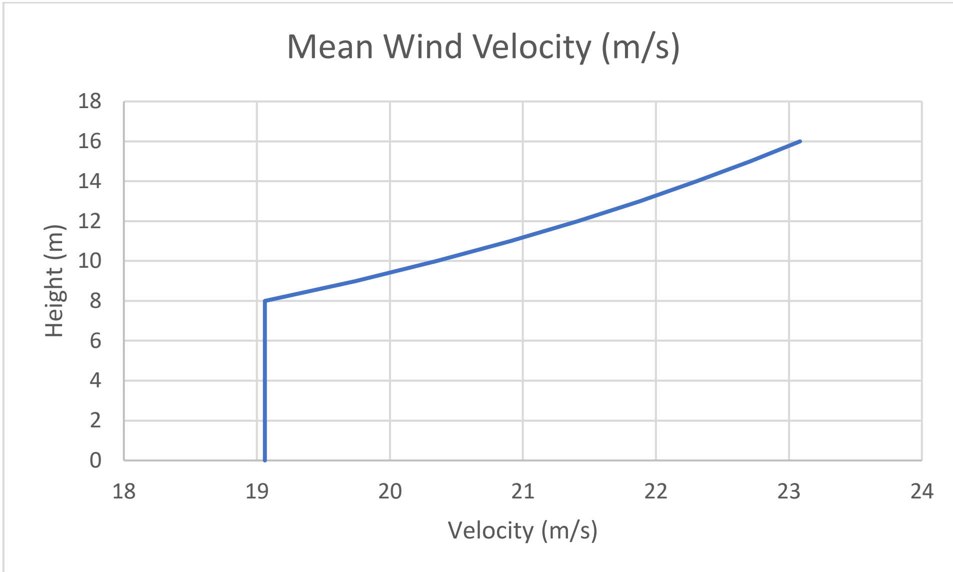


Figure 4-2 Wind mean velocity building 1

As a result, the pressure at a height of 16 meters was calculated.

$$q_p = 0.922k\,N/m^2 \tag{4.3}$$

Where,

q_p *The peak velocity pressures.*

	A	B	C	D	E	F	G	H	I
Wind X	-1.11	-0.74	-0.46	0.66	-0.3	-1.66	-1.11	-0.65	-0.18
Wind Y	-1.11	-0.74	0	0.68	-0.35	-1.66	-1.11	-0.65	-0.18

Table 4-9 Wind pressure for each area building 1

4.2.6.1. Building 2 wind load

For this building, since the height is greater than both X and Y dimensions, a different approach was needed. Following what is stated in the norms, three peak velocity pressures where needed depending on the direction of the wind pressures that it was going to be calculated, see Figure 4-3. Taking all these into consideration the pressures due to the wind where calculated.

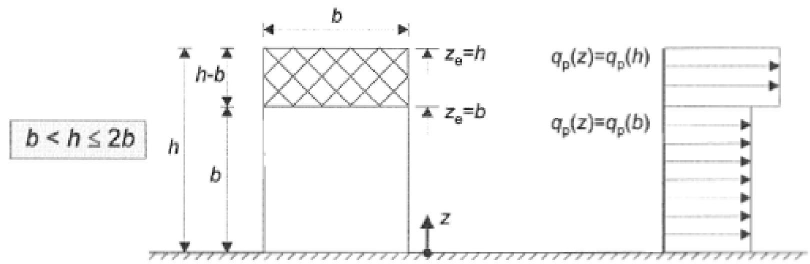


Figure 4-3 Reference height, Z_e, depending on h and b, and corresponding velocity pressure profile (CEN, 2010)

1-Wind pressures			2.2-Roughness factor		
1.1-Basic parameters acc. National Annex			z_0 =	1.00	m (Tabela 4.1)
$v_{b,0}$ =	27.00	m/s	z_{min} =	15.00	m (Tabela 4.1)
c_{dir} =	1.00	(recommended)	k_r =	0.234	
c_{season} =	1.00	(recommended)	**3-Orography Coefficient**		
ρ =	1.25	kg/m^3	ϕ =	0	(figura A.1 or A.2)
1.2-Basic wind velocity EN 1991-1-4 (4.1)			S =	0	(figura A.2 or A.3)
v_b =	27.00	m/s	c_o =	1	
1.3-Basic Velocity Pressure			**4-Wind turbulance**		
q_b =	455.63	N/m^2	K_I =	1	(Recommended)
2-Wind force			**5-Exposure coefficeint**		
2.1-Terrain category (Table 4.1)			Peak factor=	3.50	
Category:	III				

Table 4-10 Calculation for the wind pressure building 2 (CEN, 2010)

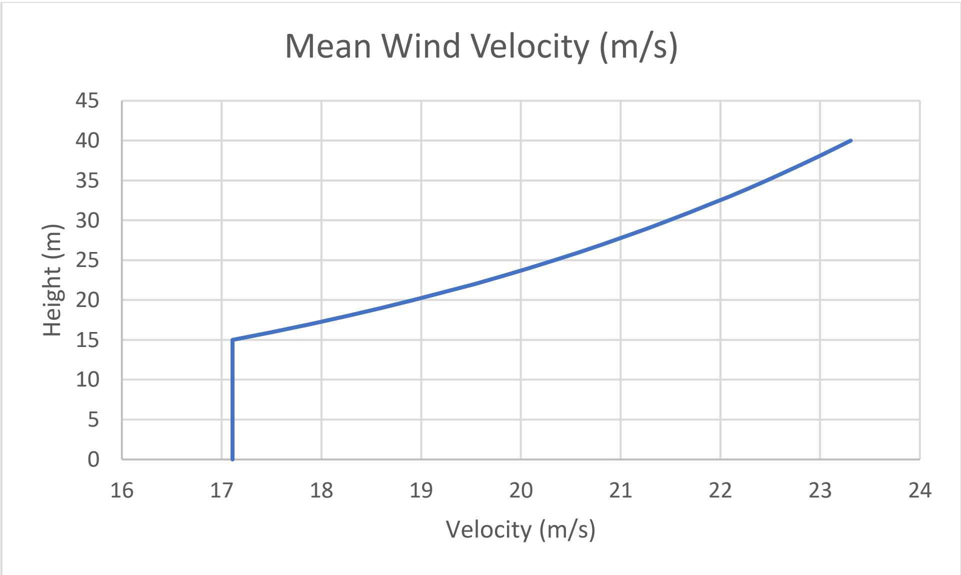


Figure 4-4 Wind mean velocity building 2

$$q_{p20} = 0.749k\,N/m^2 \tag{4.4}$$

$$q_{p32} = 0.907k\,N/m^2 \tag{4.5}$$

$$q_{p40} = 0.986\,N/m^2 \tag{4.6}$$

q_{p20} *The peak velocity pressures at a height of 20 meters.*

q_{p32} *The peak velocity pressures at a height of 32 meters.*

q_{p40} *The peak velocity pressures at a height of 40 meters.*

	A_{20}	A_{32}	A_{40}	B_{20}	B_{32}	B_{40}	C_{20}	C_{32}	C_{40}	D_{20}	D_{32}	D_{40}	E_{20}	E_{32}	E_{40}	F	G	H	I
WIND X	0.00	-1.09	-1.18	0.00	-0.73	-0.79	0.00	-0.45	-0.49	0.00	0.73	0.79	0.00	-0.47	-0.51	-1.78	-1.18	-0.69	-0.20
WIND Y	-0.90	0.00	-1.18	-0.60	0.00	-0.79	0.00	0.00	0.00	0.60	0.00	0.79	-0.41	0.00	-0.54	-1.80	-1.20	-0.70	-0.20

Table 4-11 Wind pressure for each area building 2

4.2.7. Load summary

In Table 4-12, we present a summary of the loads that will be applied to each building for design purposes. These loads will be applied in the same way for all buildings, even when considering robustness.

Load	Unit
Self-weight	Considered for each element
Rest of permanent load Glass facade Roof covering	2.92 KN/m^2 2.00 KN/m 2.00 KN/m^2
Live load	3.00 KN/m^2
Wind load	Table 4-9 and Table 4-11
Snow load	0.479 KN/m^2
Roof load	0.40 KN/m^2

Table 4-12 Applied load summary

4.2.8. Load combination

The load combinations that were considered for the ultimate limit states was the fundamental combination.

$$\sum_{j\geq 1} \gamma_{G,j} G_{k,j} \text{ "} + \text{"} \gamma_{Q,1} Q_{k,1} \text{ "} + \text{"} \sum_{i>1} \gamma_{Q,1} \psi_{0,i} Q_{k,i} \tag{4.7}$$

Where it must be verified:

- EQU – Lost of static equilibrium

Design Situation	$\gamma_{G,\text{sup}}$ Unfavorable	$\gamma_{G,inf}$ Favorable	$\gamma_{Q,1}$ Unfavorable	$\gamma_{Q,1}$ Favorable	$\gamma_{Q,i}$ Unfavorable	$\gamma_{Q,i}$ Favorable
Persistent	1.10	0.90	1.50	0.00	1.50	0.00
Transient	1.10	0.90	1.50	0.00	1.50	0.00

Table 4-13 Design value for equation EQU

- STR – Structure collapse

Design Situation	$\gamma_{G,\text{sup}}$ Unfavorable	$\gamma_{G,inf}$ Favorable	$\gamma_{Q,1}$ Unfavorable	$\gamma_{Q,1}$ Favorable	$\gamma_{Q,i}$ Unfavorable	$\gamma_{Q,i}$ Favorable
Persistent	1.35	1.00	1.50	0.00	1.50	0.00
Transient	1.35	1.00	1.50	0.00	1.50	0.00

Table 4-14 Design value for equation STR

- GTR – Excessive deformation or collapse of the foundations (in this investigation, the foundations are not considered)

The combination factors that will be used where selected from Table 2-1 from this project.

4.3. Design assumptions

4.3.1. Structural modeling

For the structural modeling the column base were considered to be fixed only allowing rotation on the global axis x, along the moment resistance frame (MRF), see Figure 4-5 Global modeling considerations. The primary beam where considered continuous and where used to complete the MRF and the secondary beam was idealized as perfectly pin at both ends. To simulate the rigidity of the concrete slab and create a diaphragm effect, concrete beams were place in between the secondary beams. These beams will not contribute to the final resistance of the structure or change the distributions of the loads. These considerations where taken for both buildings.

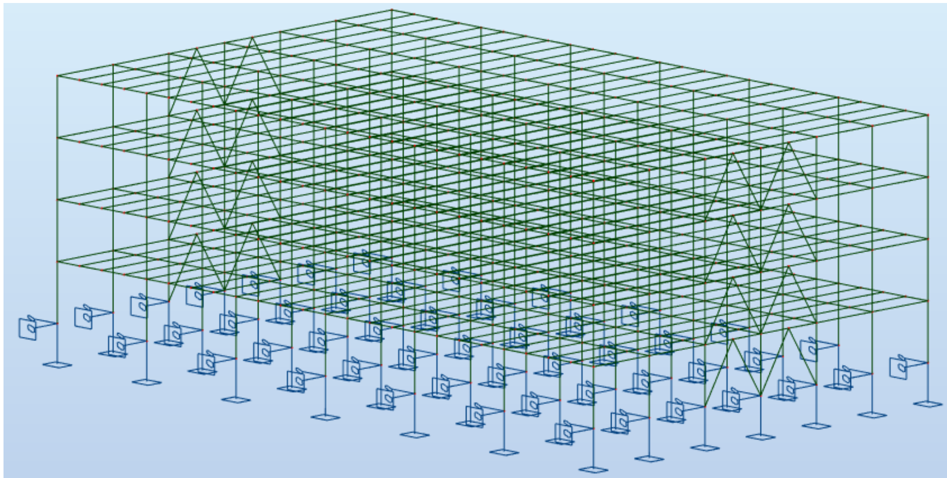


Figure 4-5 Global modeling considerations

4.3.2. Building characterization

The classification for robustness starts with the categorization for the design of the building. The first one is in section 2.1 of the EN 1990, where it has to make a distinction from 5 different categories to decide the working life stablished for the building, see Table 4-15.

Design working life category	**Indicative design working life**	**Examples**
1	10 years	Temporary structures.
2	10 to 25 years	Replaceable structural parts, e.g. gantry girders, bearings.
3	15 to 30 years	Agricultural and similar structures.
4	50 years	Building structures and other common structures.
5	100 years	Monumental buildings structures, bridges, and other civil engineering structures.
(1) Structures or parts of structures that can be dismantled with a view to being re-used should not be considered temporary.		

Table 4-15 Indicative design work life (CEN, 2009)

One of the parameters that will be keep constant for all buildings will be its usage, making this building category 4. The next step it is necessary to define the consequence class from annex B of this same norm, see Table 2-2 of this project. This building will be considered a consequence class type CC2 which is medium, consequence for loss of human life, economic, social or environmental consequence are considerable.

Reliability Class	Minimum values for β	
	1-year reference period	50 years reference period
RC3	5.2	4.3
RC2	4.7	3.8
RC1	4.2	3.3

Table 4-16 Recommended value for reliability index β (Ultimate Limit State) (CEN, 2009)

For the reliability class, for building with a 50-year working life the minimum allowed by the norm is a $\beta = 3.8$, making this building class RC2. After establishing the characterization of the building, the project is ready to be designed.

4.3.3. Composite slab

The floor system used is a two-way composite pavement with a steel sheet, that is supported by secondary steel beam, where the slab unloads the action to, with composite characteristics and that they are separated by 2 meters. The steel sheet was selected from company's catalogs and were verified for Ultimate Limit States and Serviceability Limited State. Its main geometrical characteristics are shown in Figure 4-6. Which represent "haircol S59" steel sheet from the company ArcelorMittal.

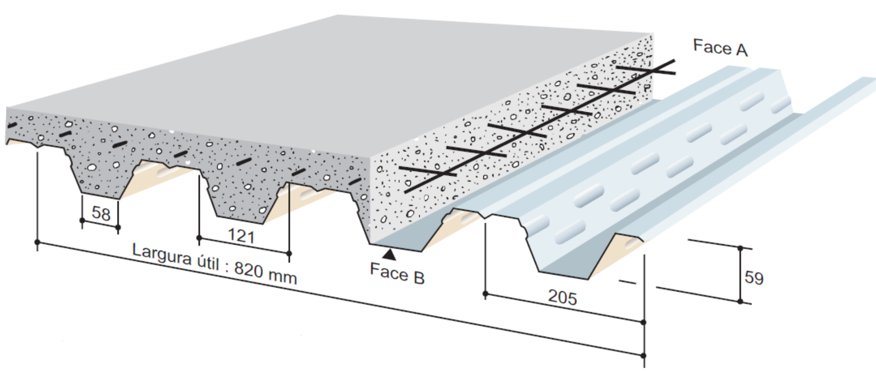


Figure 4-6 Steel sheeting geometrical characteristics

Using a steel sheet from this catalog ensures that all the local verifications are satisfied. Also, it gives us the moment that can be applied according to the set up stipulated for the floor. For all the buildings, it was considered a steel sheet with a thickness of 0.75 mm and it was considered a continuous beam with four symmetrical spans for the global analysis. Also, the concrete total height considered was 120 mm. As a result, the maximum load combination allowed is $13.26\ KN\ /\ m^2$. Using the characteristic combination for buildings the following load where used, see Table 4-17.

Loads	$KN\ /\ m^2$
Steel sheet self-weight	0.0851
Concrete slab self-weight	2.17
Other elements self-weight	2.92
Utilization load	3.00

Table 4-17 Load applied to concrete slab

$$\sum_{j>1} \gamma_{G,j} G_{k_1 j} \text{"} + \text{"} \gamma_p P \text{"} + \text{"} \gamma_{Q,1} Q_{k,1} \text{"} + \text{"} \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \tag{4.8}$$

Where,

$"+"$ *Implies "to be combine with".*

Σ *Implies "to be combine with".*

G_{kj} *The characteristic values of the permanent actions.*

$\gamma_{G,j}$ *Partial factor for permanent action j.*

P *Relevant representative value of a prestressing action.*

γ_p *Partial factor for prestressing actions.*

$Q_{k,1}$ *The characteristic value of the leading variable actions.*

$\gamma_{Q,1}$ *Partial value factor for the leading variable action.*

$Q_{k,i}$ *The characteristic values of the accompanying variable actions i.*

$\gamma_{Q,i}$ *Partial value factor for the accompanying variable actions i.*

$\psi_{o,i}$ *The factor for combination for the accompanying variable action i.*

Using this equation, it can be verified, in equation 4.2, that the load applied is lower than the resistance of the slab.

$$1.35(2.09 + 2.92) + 1.5(3) = 11.26 kN\ /\ m^2 < 13.26\ KN\ /\ m^2 \tag{4.9}$$

For the cracking of the concrete, EN 1994-2 in section 7, gives the minimum area admissible for reinforcement bar to control the cracks. This area can be found using different methods, one controlling the spacing between the reinforcement bar and another one controlling the size of the bars, the second approach was used. In this project a reinforcement welded mesh was used having a longitudinal wire of 8 mm spaced 100 mm and cross wire of 8 mm separated 200 mm.

$$A_s = \frac{k_s k_c k f_{ct,eff} A_{ct}}{\sigma_s} \tag{4.10}$$

k_s Coefficient that allows to take into consideration the effect of the reduction of the normal force of the concrete slab due to initial cracking and local slip of shear connection, which may be taken as 0.9

k_c Coefficient which considers of the stress distribution within the section immediately prior to cracking and is given by:

$$k_c = \frac{1}{1 + \frac{h_c}{2z_0}} + 0.3 \leq 1.0 \tag{4.11}$$

h_c The thickness of the concrete flange.

Z_0 The vertical distance between the centroids of the un-cracked concrete flange and the un-cracked composite section, calculated using the modular ration n_0 for short term loading.

k Coefficient which allows for the effect of non-uniform self-equilibrating stress which may be taken as 0.8.

$F_{ct,eff}$ The mean value of the tensile strength of the concrete effective at the time when cracks may first be expected to occur. This value may be taken as f_{ctm} from table 3.1 of NP EN 1992-1-1.

A_{ct} The area of the tensile zone immediately prior of the cracking of the cross section

σ_s The maximum stress permitted in the reinforcement immediately after cracking.

Where, σ_s was considered $360\ N / mm^2$, from the table 7.1 in EN 1994-2 here represented in Table 4-18, due to the size of the wire.

Steel stress σ_s (N/mm^2)	Maximum bar diameter ϕ^* (mm) for design crack width w_k		
	w_k=0.4mm	w_k=0.3mm	w_k=0.2mm
160	40	32	25
200	32	25	16
240	20	16	12
280	16	12	8
320	12	10	6
360	10	8	5
400	8	6	4
450	6	5	-

Table 4-18 Maximum bar diameter for high bond bars (CEN, 2005)

$$k_c = \frac{1}{1+\left(\frac{120}{2(19.4)}\right)} + 0.3 = 0.69 \tag{4.12}$$

$$A_{s,l} = \frac{(0.9)(0.8)(0.69)(2.9)(86{,}758.5)}{360} = 346.68\, mm^2/m \tag{4.13}$$

$$A_{s,t} = \frac{(0.9)(0.8)(0.69)(2.9)(61{,}000.0)}{360} = 243.75\, mm^2/m \tag{4.14}$$

Making the welded mesh selected enough to control the crack as it can be seen in Table 4-19.

	$A_{s,Ed}\ mm^2/m$	$A_{s,Rd}\ mm^2/m$
$A_{s,l}$	346.68	503
$A_{s,t}$	243.75	252

Table 4-19 Longitudinal reinforcement for cracking control

4.3.4. Secondary beam

The secondary beam was designed as a composite structure and the connection beam-to-beam was considered as perfectly pin for calculation purpose, as it was mention before. The ribs of the steel sheet where considered perpendicular to the beam. The layout for the beams can be seen in Figure 4-7and Figure 4-8, where the secondary beams are separated by 2 meters.

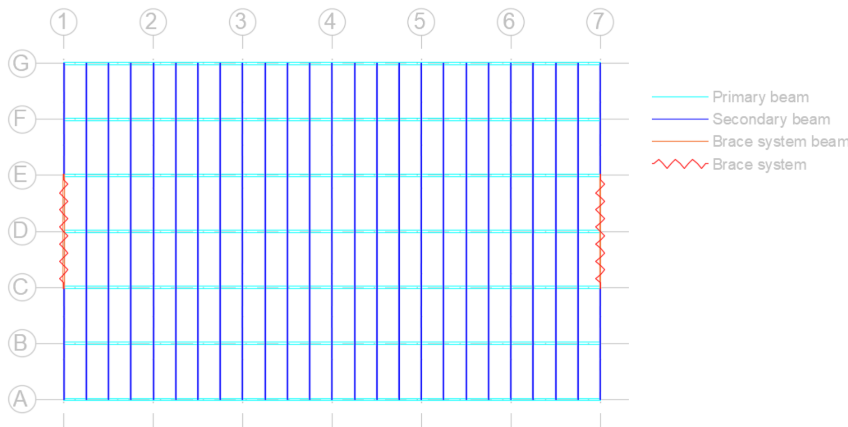


Figure 4-7 Primary and secondary beam layout building 1

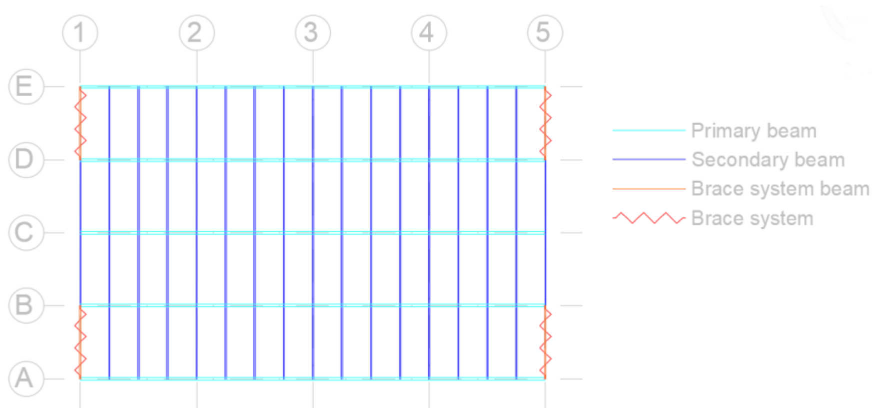


Figure 4-8 Primary and secondary beam layout building 2

For the design, the program known as ArcelorMittal Beam Calculation (ABC), which is a software created by AcerlorMittal, was used to help with the structural verification for composite beams. Since the same layout for the secondary beams were used for all buildings the verification was done only once. Using the same loads applied for the slab verification, see Table 4-17, the composite beam was design.

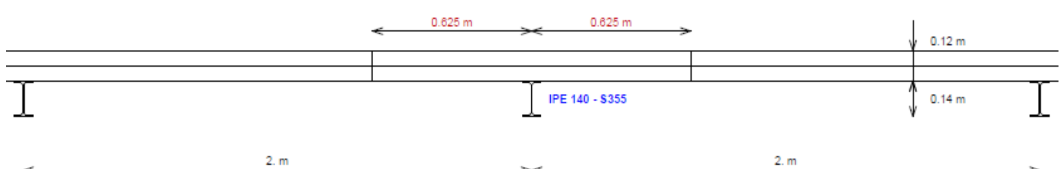


Figure 4-9 Composite beam design set up (ARCELORMITTAL)

For the verification, the section used was an IPE 140, see mechanical properties in Table 4-20. This was selected since it was the beam capable of resisting the moment applied, in both under construction and at the ultimate limited states stages, without the need of propping, as it was said before. This section was critical for the resistance of the maximum moment applied for the construction stage.

IPE 140		
A =	16.43	cm^2
A_v =	7.64	cm^2
I_y =	541.22	cm^4
I_z =	44.92	cm^4
I_t =	2.45	cm^4
I_w =	1,981.36	cm^6
$W_{el,y}$ =	77.32	cm^3
$W_{pl,y}$ =	88.34	cm^3

Table 4-20 Mechanical characteristics for the secondary beam (ARCELORMITTAL)

Using the verification for the Ultimate Limit State in EN 1993 part 1-1, for plastic shear and moment, the beam was validated. First for the construction phase then as a composite member. Also, this member does not have to be verified for the shear buckling of the web according to the norm.

$$V_{Pl,Rd} = \frac{A_v\left(f_y \,/\, \sqrt{3}\right)}{\gamma_{M_0}} = 156.64\ KN \tag{4.15}$$

$$M_{Pl,Rd} = \frac{W_{pl} f_y}{\gamma_{M_0}} = 31.36\ KNm \tag{4.16}$$

Where,

$M_{Pl,Rd}$ *The plastic resistance moment of the section.*

$V_{Pl,Rd}$ *The plastic shear resistance of the section.*

A_V *The shear area of the section.*

f_y *The yield strength of the structural steel.*

In the construction phase, the moment and shear applied using the characteristic combination for loads using a construction load of 1.5 $kN\ /\ m^2$ was calculated to be 8.28 KN/m, as it can be seen in equation 4.16. Using this value, and applying the statics equation for equilibrium, the forces acting on the beam were calculated. For this load the maximum deflection was $42.5mm$, that is below the limits of the Eurocode.

$$w = 1.35(0.13 + 4.34) + 1.5(1.5) = 8.28\ KN/m \tag{4.17}$$

$$V_{Ed} = \frac{wL}{2} = 20.70kN \tag{4.18}$$

$$M_{Ed,max} = \frac{wL^2}{8} = 25.87\ KNm \tag{4.19}$$

Where,

V_{Ed} *The shear stress applied to the beam.*

$M_{Ed,max}$ *The maximum moment applied to the beam.*

w *The combination load applied to the beam.*

L *The length of the beam.*

To calculate the plastic resistance of the composite beam, the shear lag for the concrete flange was considered following EN 1994 part 1-1 section 5.4.1.2. Where b_{ei} is the effective flange of concrete in each side of the web of the section and it's considered to be $L_e\ /\ 8$ or the actual length of the flange, whichever is smaller. For the ABC software, $L_e = 5\ m$ for simplification, when should be calculated as seen in Figure 4-10, making it $L_e = 0.85(5.00) = 4.25\ m$.

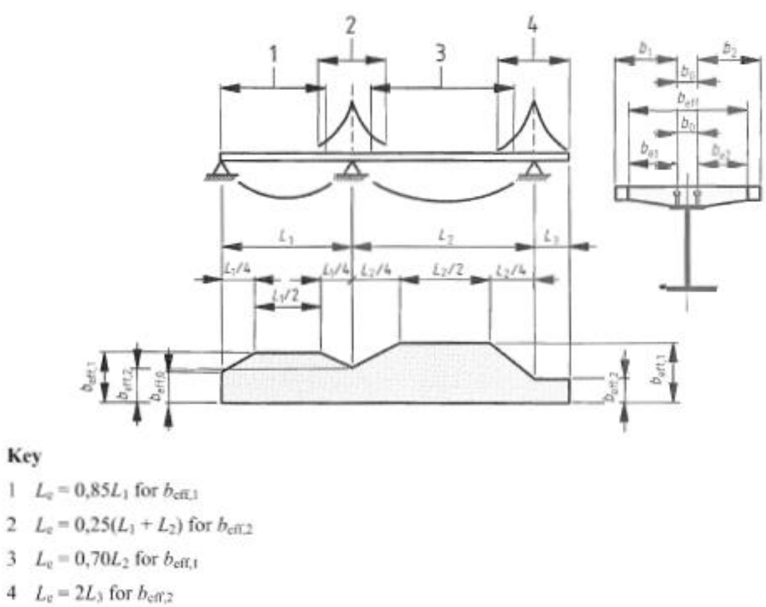


Figure 4-10 Equivalent span, for effective width of concrete flange (CEN, 2010)

$$b_{eff} = b_0 + \Sigma b_{ei} = 1.25\ m \tag{4.20}$$

Equation 4.13 is used for the effective flange in mid span, where,

b_0 *The distance between connectors.*

b_{ei} *The effective width of the concrete flange.*

$$b_{eff} = b_0 + \Sigma \beta_i b_{ei} = 0.938\ m \tag{4.21}$$

Equation 4.14 is used for external support, where,

$$\beta_i = (0.55 + 0.025 L_e / b_{ei}) = 0.75 \tag{4.22}$$

For the connection concrete to structural steel shear stud where used. It was selected from the catalogue used by the software; its mechanical characteristics are presented in Table 4-21. Using these values, the nominal resistance of the shear stud was calculated using equations 3.11, 3.12 and 3.13.

TRW Nelson KB 5/8" - 100		
d =	16.00	mm
h =	100.00	mm
f_y =	350.00	N/mm^2
f_u =	450.00	N/mm^2

Table 4-21 Mechanical properties for shear stud (ARCELORMITTAL)

$$P_{Rd,1} = 57.91\ kN \tag{4.23}$$

$$P_{Rd,2} = 58.19\ kN \tag{4.24}$$

$$P_{Rd} = 57.91 kN \tag{4.25}$$

Since the steel sheet is perpendicular to the beam the reduction factor was calculated, see equation 3.17.

$$k_t = 0.738\ KN \tag{4.26}$$

As a result, the resistance value for the shear stud will be:

$$P_{Rd} = P_{Rd,1} k_l = 42.73\ KN \tag{4.27}$$

One row of connectors on each rib of the steel sheet was used allowing a partial connection between the materials. This made the beam to have to resist some compression on the upper part of the section on the ultimate limit state. As a result, using the software, the ultimate resistance for the bending moment of the composite beam as well as the short and long-term mechanical properties, see Table 4-22.

$M_{pl,Rd}$ =	$96.11\ KNm$
$V_{Pl,Rd}$ =	$156.64\ KN$
$I_{m,short-term}$ =	$4573\ cm^4$
$I_{m,long-term}$ =	$3681\ cm^4$

Table 4-22 Composite beam mechanical characteristics (ARCELORMITTAL)

4.3.5. Primary beam

The primary beams were considered continuous along the X axis of the global coordinate of the building. They were not considered as composite structure in order to have enough resistance in both construction and utilization phase. This will allow to have a faster constructive process not having to use propping, like it was done for the secondary beam. For optimization purposes, the beams for the intermedium floors where design separately from the ones on the roof. The verification for the ultimate limit state of these beams where done through the software ROBOT Structural Analysis, se Annex, only for the utilization phase.

For the primary beams, it was considered that they do not have any connection to the concrete. For analysis purposes, the beams were restricted by the secondary beam and that they applied the load. As it was stated before, the slab is a two-way slab that unloads to the secondary beam. For the serviceability limit state, it was considered the beam with the worst conditions to do a deformation check.

$$\delta_{max} = L\ /\ 250 = 8{,}000mm\ /\ 250 = 40mm \qquad (4.28)$$

Where,

δ_{max} *Maximum deformation allowed*

L *Beam length*

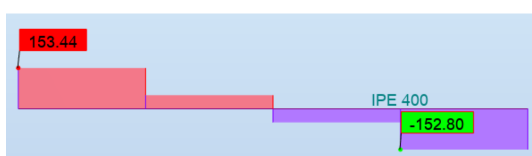


Figure 4-11 Shear stress diagram for primary beam in KN

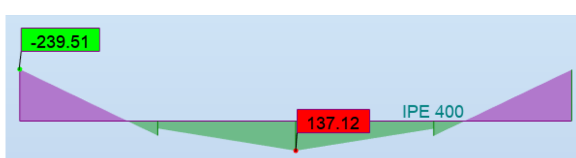


Figure 4-12 Bending moment diagram for primary beam in KNm

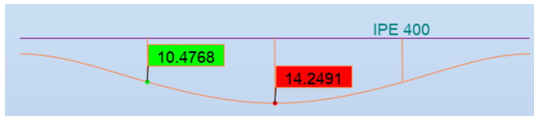


Figure 4-13 Maximum deformation for primary beam in mm

4.3.6. Columns

The columns were separated into two groups, according to its position, in order to try to optimize as much possible the structure. The first group is the external column, which usually must resist less stress and the internal column. For building 2, due to its height, each group was divided into three sub section to enhance the design even more, see

Figure 4-15 and Figure 4-17. As it was mention before, in both cases, for the global analysis, the column base where considered fixed allowing rotation on the x direction for the global coordinate.

The columns do not have any restraining in any direction making it mandatory to verify the resistance to lateral buckling on both directions. They were also checked using the software Robot Structural Analysis, in the same way it was done for the beams, following Eurocode 3 part 1-1, for Ultimate Limited State, see Annex.

The serviceability state the building must be checked for local and global deformation. This was done manually for each building following EN 1990. To do this verification, the set of columns with the worst conditions were selected for each building and verify.

Column A4	UY (mm)	u (mm)	Column G4	UY (mm)	u (mm)
100373	15.4732	1.4046	100379	15.4251	1.277
100320	14.0686	3.3653	100326	14.1481	3.3726
100267	10.7033	4.4969	100273	10.7755	4.4644
100059	6.2064	6.2064	100061	6.3111	6.3111
100058	0		100060	0	

Table 4-23 Critical deformation for building 1

Column A4	UY (mm)	u (mm)	Column G4	UY (mm)	u (mm)
100009	77.23	8.41	106009	77.31	8.54
90009	68.82	9.11	96009	68.77	9.09
80009	59.71	9.31	86009	59.68	9.32
70009	50.40	9.36	76009	50.36	9.34
60009	41.03	9.22	66009	41.01	9.24
50009	31.81	8.66	56009	31.77	8.66
40009	23.15	7.62	46009	23.11	7.59
30009	15.53	6.60	36009	15.52	6.63
20009	8.93	5.19	26009	8.89	5.20
10009	3.73	3.73	16009	3.69	3.69
3	0.00		6003	0.00	

Table 4-24 Critical deformation for building 2

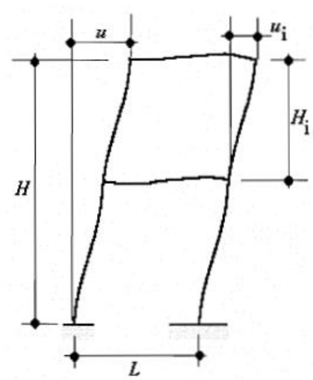


Figure 4-14 Definition of horizontal displacement (CEN, 2009)

$$u_i \leq H_i \;/\; 150 \tag{4.29}$$

$$u \leq H \;/\; 500 \tag{4.30}$$

Where,

u_i *Overall horizontal displacement over the building height* H

u *Horizontal displacement over a story height* H_i

In Figure 4-16 and Figure 4-18 we can see the reaction to compression for all elements in the structure. In both cases, it represents the result for the worst-case scenario for the building in ULS. In these images, we can have a sense on what it was said before. At the same time, the exterior columns had to resist to some moment as well.

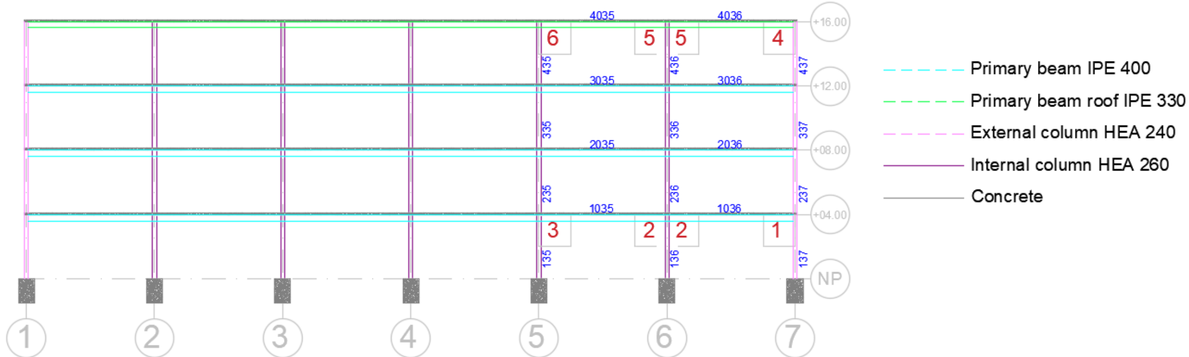


Figure 4-15 Connection layout building 1

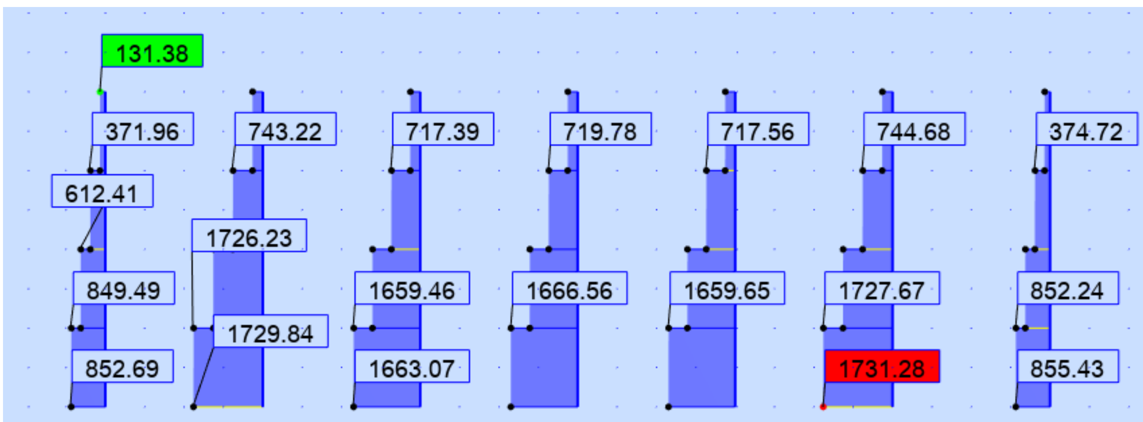


Figure 4-16 Compression diagram applied to each column for building 1

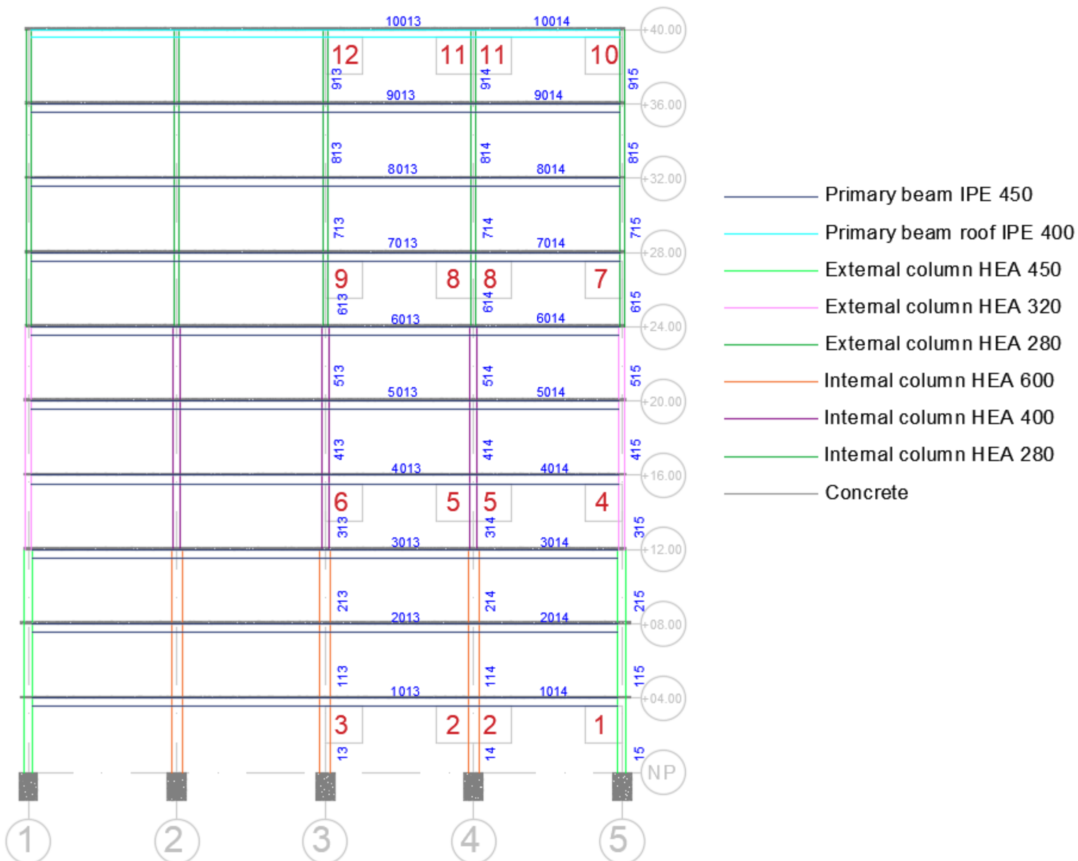


Figure 4-17 Connection layout building 2

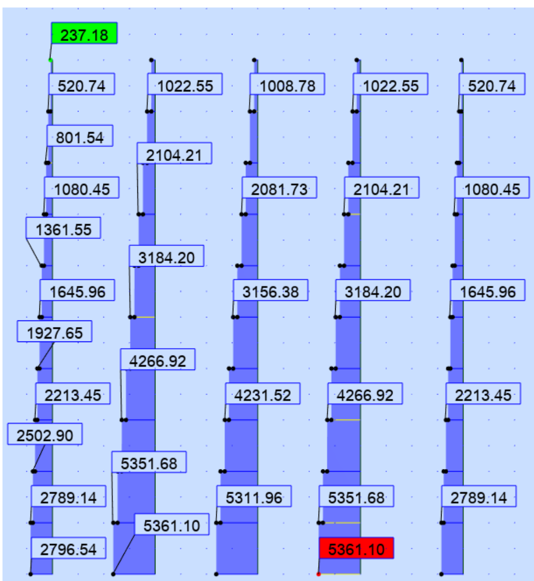


Figure 4-18 Compression diagram applied to each column for building 1

4.3.7. Beam-to-column connection

The design of the connection was done using the global analysis of the structure. Like the column and the beam, the software Robot Structural Analysis was used following Eurocode 3 part 1-8. In the global model all connections where considered rigid, as a result, all the connection where designed to fulfill this characteristic. For the building 1, six critical connection in B frame where selected, they were all verified, ensuring to have a rigid connection, and that they satisfied the requirements of the norms, in

Figure 4-15 it can be seen the layout of the connections considered.

Since connection two and three connect the same of elements, this connection where design equal. The same approach was considered for connection five and six and, as it will see later, for building 2 the same situation was presented.

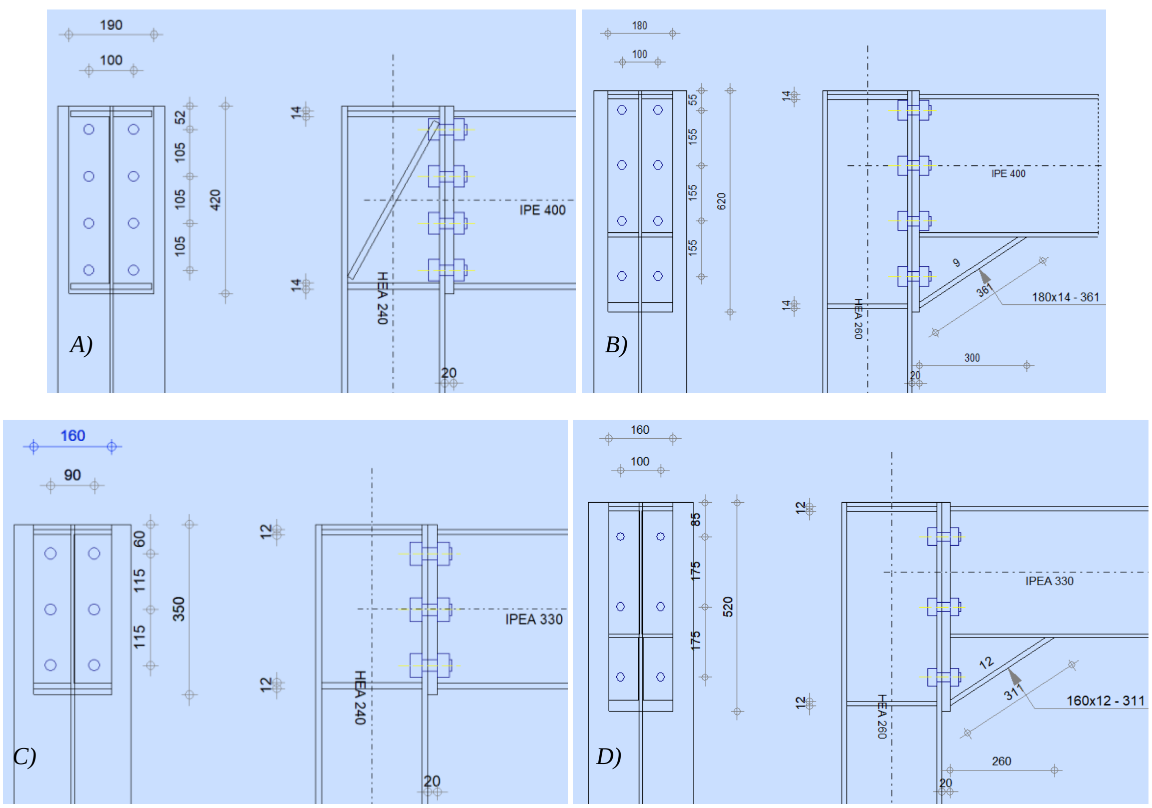
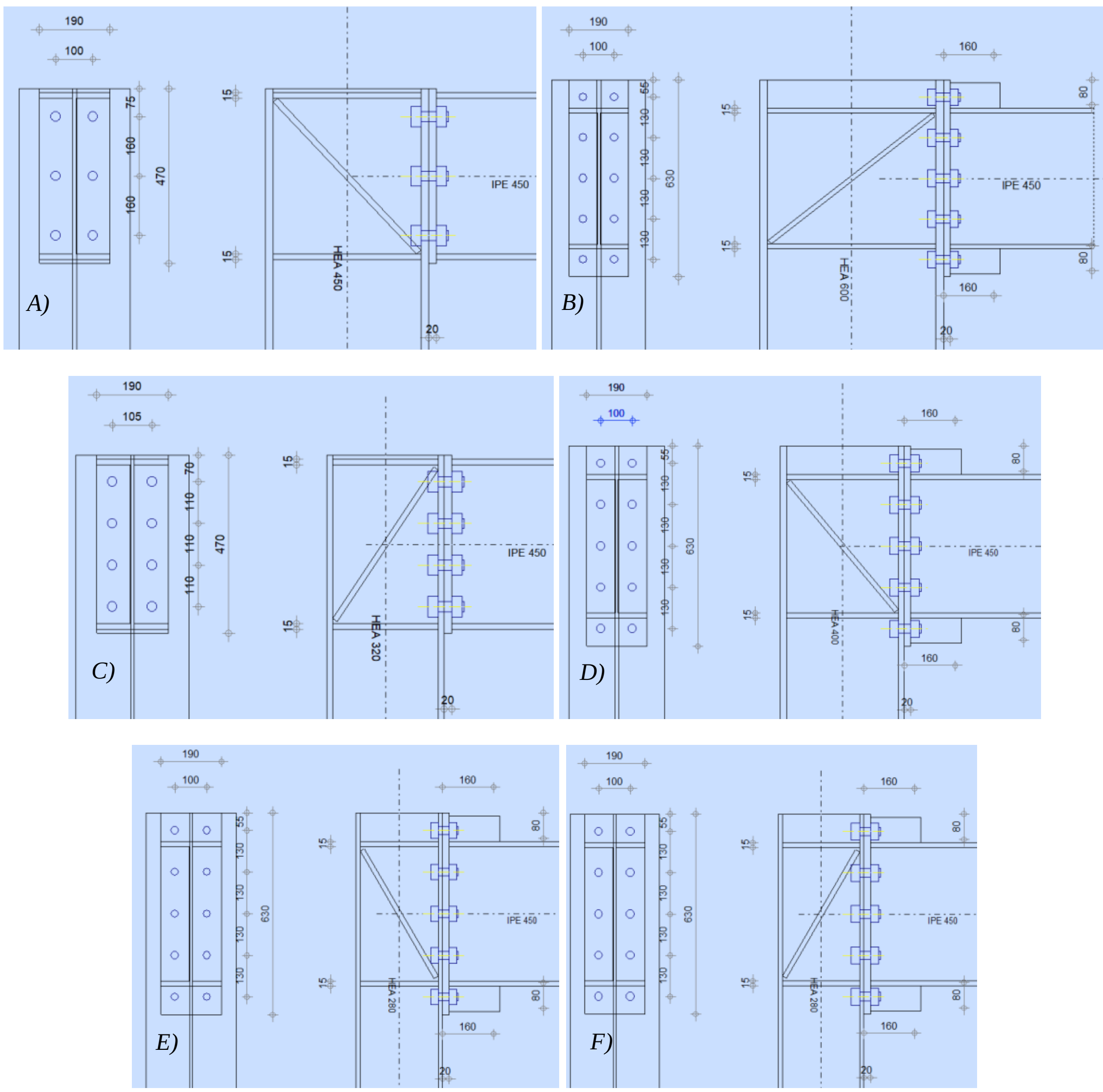


Figure 4-19 Connection drawing building 1 - A) Connection 1 - B) Connection 2 – C) Connection 4 – D) Connection 5

	Bolt	$S_{j,rig}$	$S_{j,pin}$	$S_{j,ini}$	S_j
Connection 1	M24	48,569.64	3,035.60	82,299.81	27,940.52
Connection 2	M27	48,569.64	3,035.60	170,811.84	57,406.96
Connection 3	M27	48,569.64	3,035.60	199,398.28	97,942.56
Connection 4	M24	21,484.68	1,342.79	21,925.29	13,768.92
Connection 5	M22	21,484.68	1,342.79	92,021.11	35,064.82
Connection 6	M22	21,484.68	1,342.79	98,056.68	46,974.64

Table 4-25 Connection rotational stiffness building 1-1 (kNm)



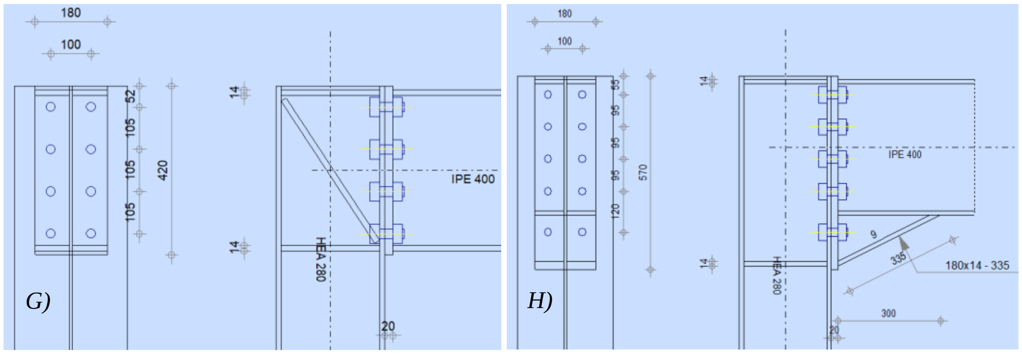


Figure 4-20 Connection drawing building 2 - A) Connection 1 - B) Connection 2 – C) Connection 4 – D) Connection 5 – E) Connection 7 – F) Connection 8 – G) Connection 10 – H) Connection 11

	Bolt	$S_{j,rig}$	$S_{j,pin}$	$S_{j,ini}$	S_j
Connection 1	M27	70,860.09	4,428.76	81,992.66	29,129.22
Connection 2	M27	70,860.09	4,428.76	60,917.45	28,618.48
Connection 3	M27	70,860.09	4,428.76	60,917.45	29,785.97
Connection 4	M27	70,860.09	4,428.76	106,629.51	37,346.29
Connection 5	M27	70,860.09	4,428.76	74,901.53	46,117.86
Connection 6	M27	70,860.09	4,428.76	75,976.43	75,976.43
Connection 7	M24	70,860.09	4,428.76	74,564.92	45,493.89
Connection 8	M27	70,860.09	4,428.76	74,564.92	29,657.89
Connection 9	M27	70,860.09	4,428.76	78,312.54	33,006.51
Connection 10	M24	48,569.64	3,035.60	77,988.74	31,091.51
Connection 11	M24	48,569.64	3,035.60	187,229.93	81,408.48
Connection 12	M24	48,569.64	3,035.60	180,997.80	73,471.38

Table 4-26 Connection rotational stiffness building 2-1 (kNm)

4.3.8. Braces system

The braces system was used to control the building deformation due to the wind pressure. These elements will only receive axial forces of tension or compression and will be considered pinned in both extremities. They were used on the y axis of the global coordinate of the structure as it can be seen in Figure 4-7 and Figure 4-8. The beam where the braces connect to the building was not considered as composite for simplification purpose.

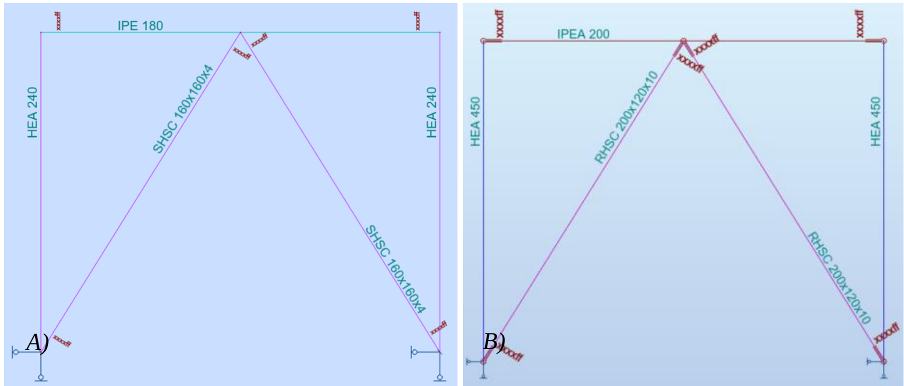


Figure 4-21 A) Brace system building 1 – B) Brace system building 2

4.3.9. Final solution

With these considerations made for the global and local analysis, the final solution for building 1 and 2 are presented in Table 4-27 and Table 4-28.

Element	Section
Internal Column	HEA 260
External Column	HEA 240
Primary Beam	IPE 400
Secondary Beam	IPE 140
Primary Beam Roof	IPEA 330
Brace	SHSC 160x160x4
Brace Beam	IPE 180

Table 4-27 Final solution building 1

Element	Section
Internal	
Column 1	HEA 600
Column 2	HEA 400
Column 3	HEA 280
External	
Column 1	HEA 450
Column 2	HEA 320
Column 3	HEA 280
Primary Beam	IPE 450

Secondary Beam	IPE 140
Primary Beam Roof	IPE 400
Brace	RHSC 200x120x10
Brace Beam	IPEA 200

Table 4-28 Final solution building 2

The parameters used for the structural analysis of these buildings will be kept the same. The only modification will be the removal of the column according to the Eurocodes to apply robustness.

5. Evaluation of robustness on the building

After both buildings were designed following the norm, we proceeded to apply EN 1991 part 1-7. The structural considerations for all elements will be the same as before explained, making sure they can sustain the stresses of an accidental combination by removing a column while applying robustness, and having the ultimate limited state and serviceability limited state for the building under normal conditions verified.

5.1. Classification of the buildings

The first step, as it was seen in chapter 2, is to stablish the categorization for consequences classes. This will help to minimize the cost for the required solution and it is based on what is was done in section 4.3.2, which stated that both buildings are consequences class CC2, with a medium risk of failure, following EN 1990. Using table A.1 from the EN 1991 part 1-7, represented in this text in Table 2-3, we stablished the category for both buildings.

For building 1, we concluded that it belongs to the Lower Risk Group 2a, since is an office building with four floors or less. For this building, it is only required to provide an effective horizontal tie, or effective anchorage for suspended floors to walls. However, this building will be designed for the notional removal of columns in order to maintain a pattern to make comparison between solutions.

For building 2, we got to the conclusion that this belongs to the Upper Risk Group 2b, since is an office building with more than four floors, but less than fifteen. For this group, the building must be checked for the notional removal of each support, along with providing an alternative path for the load to travel with horizontal and vertical ties.

5.2. Notional removal of supports

Since both buildings are bi-symmetrical, it reduced the number of columns that were needed to be eliminated. We proceeded to remove the columns on by one and re-design the building, identifying the two critical elements for both structures. For building 1, it was columns B-6 and B-7; and for building 2, it was columns C-4 and C-5. This was done only for the first floor since it is the segment that is withstanding greater stress.

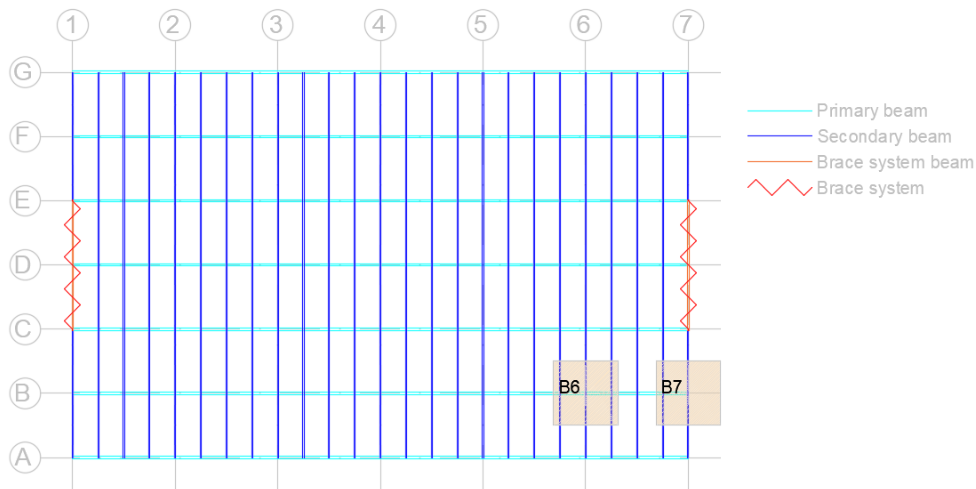


Figure 5-1 Critical column building 1

For building 1, the brace system was considered in the two-middle span of the y direction. This set up represented the most efficient way to control de deformation of the building under normal conditions for serviceability limit state. Also, it helped to alleviate the stresses on the columns around it.

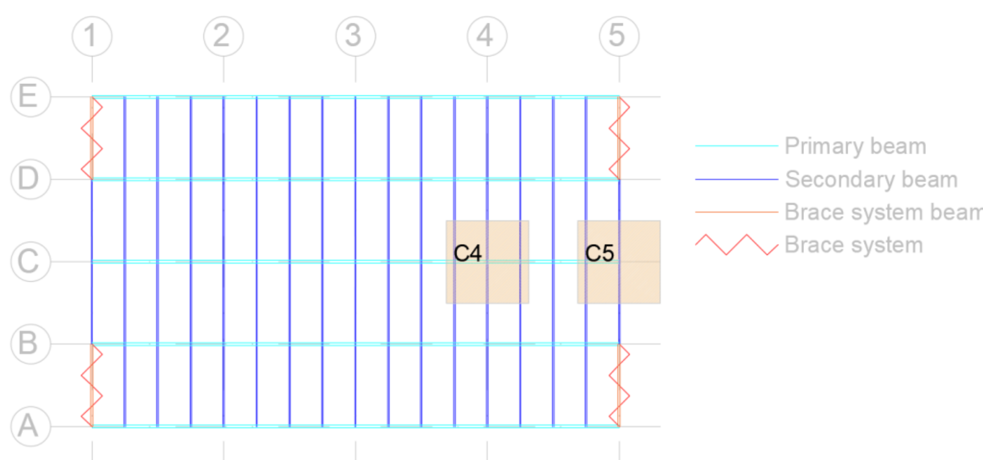


Figure 5-2 Critical column building 2

For building 2, the braces were considered on the outer spans in the y direction. As a result, the most critical elements were in the middle frame. Once again, the braces help to ease the load that are transmitted to the nearby columns. Making them as an alternative path for the load to be redistributed throughout the building.

5.3. Actions

To apply robustness in the structure, the accidental load combination will be used as it was explained before. However, all the loads will be applied only modifying the ψ factor.

5.3.1. Load combination

According to the EN 1991 part 1-7 and how it was explained before, robustness is considered an accidental load. As a result, the load combination that is needed to be used is the accidental combinations, see equation 5.1, to verify the structure at Ultimate Limited State for structural stability. However, since the buildings will be fully constructed, an ULS and SLS verification, as it was performed in chapter 4, most be performed.

$$\sum_{j\geq 1} G_{kj} \text{"} + \text{"} A_d \text{"} + \text{"} \psi_{1,1} Q_{k,1} \text{"} + \text{"} \sum_{i\geq 1} \psi_{2,i} Q_{k,i} \quad (5.1)$$

This combination will minorize the imposed load by a factor of ψ_2. In Table 5-1 we present the various combinations used to asses robustness. The Eurocode works with probabilities and, it assumes, that that the structure will not have to resist the loss of the column and the imposed load at its peak at the same time.

		PP		PP_LAJE		RCP		PP_FACHAD		PP_COBERT		Dt SUMMER		Dt WINTER		SNOW		WIND X FAV		WIND X DES		WIND Y FAV		WIND Y DES		CONSTRUC		UTILIZACAO		COBERTURA	
Name		Case	Coeff.	Case	Coeff.	Case	Coeff.	Case	Coeff.	Case	Coeff.	Case	Coeff.	Case	Coeff.	Case	Coeff.	Case	Coeff.	Case	Coeff.	Case	Coeff.	Case	Coeff.	Case	Coeff.	Case	Coeff.	Case	Coeff.
ACCIDENTAL_Qsv_Wx_FAV	ULS	1	1	2	1	3	1	4	1	5	1	6	0	7	0	8	0.2	9	0.2	10	0	11	0	12	0	13	0	14	0.7	15	0
ACCIDENTAL_Qsv_Wx_DES	ULS	1	1	2	1	3	1	4	1	5	1	6	0	7	0	8	0.2	9	0	10	0.2	11	0	12	0	13	0	14	0.7	15	0
ACCIDENTAL_Qsv_Wy_FAV	ULS	1	1	2	1	3	1	4	1	5	1	6	0	7	0	8	0.2	9	0	10	0	11	0.2	12	0	13	0	14	0.7	15	0
ACCIDENTAL_Qsv_Wy_DES	ULS	1	1	2	1	3	1	4	1	5	1	6	0	7	0	8	0.2	9	0	10	0	11	0	12	0.2	13	0	14	0.7	15	0
ACCIDENTAL_Wx_FAV	ULS	1	1	2	1	3	1	4	1	5	1	6	0	7	0	8	0.2	9	0.6	10	0	11	0	12	0	13	0	14	0.5	15	0
ACCIDENTAL_Wx_DES	ULS	1	1	2	1	3	1	4	1	5	1	6	0	7	0	8	0.2	9	0	10	0.6	11	0	12	0	13	0	14	0.5	15	0
ACCIDENTAL_Wy_FAV	ULS	1	1	2	1	3	1	4	1	5	1	6	0	7	0	8	0.2	9	0	10	0	11	0.6	12	0	13	0	14	0.5	15	0
ACCIDENTAL_Wy_DES	ULS	1	1	2	1	3	1	4	1	5	1	6	0	7	0	8	0.2	9	0	10	0	11	0	12	0.6	13	0	14	0.5	15	0
ACCIDENTAL_SNOW_Wx_FAV	ULS	1	1	2	1	3	1	4	1	5	1	6	0	7	0	8	0.5	9	0.2	10	0	11	0	12	0	13	0	14	0.5	15	0
ACCIDENTAL_SNOW_Wx_DES	ULS	1	1	2	1	3	1	4	1	5	1	6	0	7	0	8	0.5	9	0	10	0.2	11	0	12	0	13	0	14	0.5	15	0
ACCIDENTAL_SNOW_Wy_FAV	ULS	1	1	2	1	3	1	4	1	5	1	6	0	7	0	8	0.5	9	0	10	0	11	0.2	12	0	13	0	14	0.5	15	0
ACCIDENTAL_SNOW_Wy_DES	ULS	1	1	2	1	3	1	4	1	5	1	6	0	7	0	8	0.5	9	0	10	0	11	0	12	0.2	13	0	14	0.5	15	0
ACCIDENTAL_WINTER_Wx_FAV	ULS	1	1	2	1	3	1	4	1	5	1	6	0	7	0.6	8	0.2	9	0.2	10	0	11	0	12	0	13	0	14	0.5	15	0
ACCIDENTAL_WINTER_Wx_DES	ULS	1	1	2	1	3	1	4	1	5	1	6	0	7	0.6	8	0.2	9	0	10	0.2	11	0	12	0	13	0	14	0.5	15	0
ACCIDENTAL_WINTER_Wy_FAV	ULS	1	1	2	1	3	1	4	1	5	1	6	0	7	0.6	8	0.2	9	0	10	0	11	0.2	12	0	13	0	14	0.5	15	0
ACCIDENTAL_WINTER_Wy_DES	ULS	1	1	2	1	3	1	4	1	5	1	6	0	7	0.6	8	0.2	9	0	10	0	11	0	12	0.2	13	0	14	0.5	15	0
ACCIDENTAL_SUMMER_Wx_FAV	ULS	1	1	2	1	3	1	4	1	5	1	6	0.6	7	0	8	0.2	9	0.2	10	0	11	0	12	0	13	0	14	0.5	15	0
ACCIDENTAL_SUMMER_Wx_DES	ULS	1	1	2	1	3	1	4	1	5	1	6	0.6	7	0	8	0.2	9	0	10	0.2	11	0	12	0	13	0	14	0.5	15	0
ACCIDENTAL_SUMMER_Wy_FAV	ULS	1	1	2	1	3	1	4	1	5	1	6	0.6	7	0	8	0.2	9	0	10	0	11	0.2	12	0	13	0	14	0.5	15	0
ACCIDENTAL_SUMMER_Wy_DES	ULS	1	1	2	1	3	1	4	1	5	1	6	0.6	7	0	8	0.2	9	0	10	0	11	0	12	0.2	13	0	14	0.5	15	0

Table 5-1 Accidental load combinations

5.4. Design assumptions

The same assumptions made in chapter 4 will be used when evaluating the structure for robustness. In this case the buildings will be designed taking one of the critical columns off the building and designing it to have structural stability to the accidental load. This will be done having two approaches: the first one will be making the structure robust enough so that it is capable of sustaining the stress from the notional removal of the column by itself. In the other hand, the building will be designed with a supper-truss structure to help distribute the load around the building.

5.4.1. First approach: Design without truss superstructure

The buildings were designed by the removal of the notional columns and, as it was stated before, for building 1 column B7 and B6 conditioned the elements of the structure. By removing column B7, the internal columns and the roof primary beam were the most critical elements; and by removing column B6, the external columns and the primary beam were the most critical elements. Next, we verified the building for serviceability limit state, concluding that it satisfied the requirements.

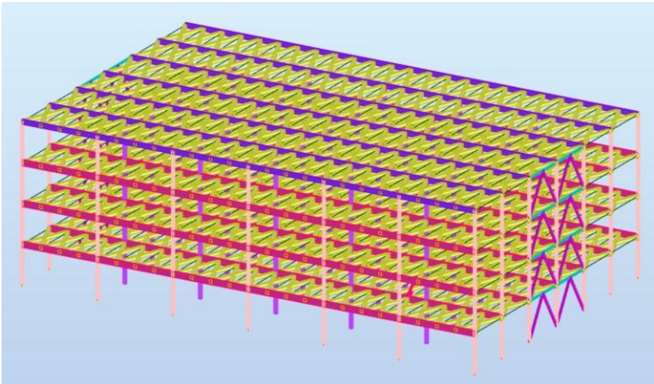


Figure 5-3 Building 1 global model overview for approach one

As it can be seen in Figure 5-4 how the load was re distributed along the structure when one of the elements is removed. The primary beams take all the loads to the two closes column. We used these over stresses to design the building once again. The same happen when the external column was removed. All the load was passed to the inside column. Something interesting is that the elements left from the second floor up did not presented any stress.

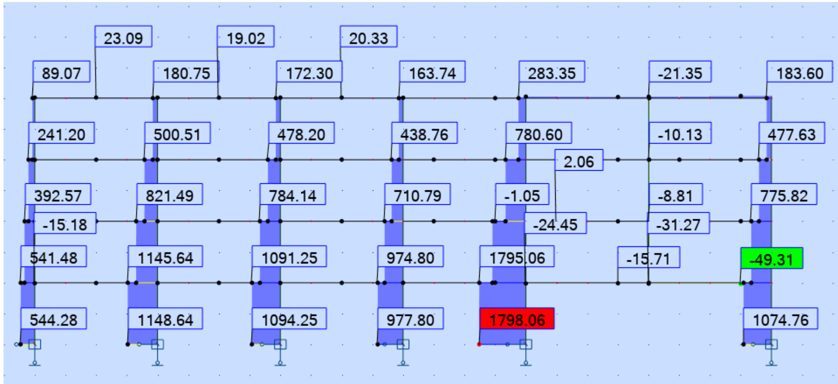


Figure 5-4 Compression diagram applied to each column for building 1

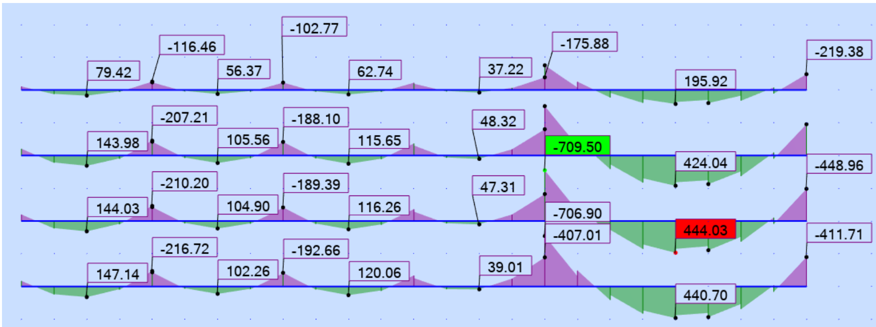


Figure 5-5 Moment diagram applied to each column for building 1

In building 2, by removing column C5, the internal columns were critical along with the primary beam. By removing the C4 column, the external and the roof's primary beams were critical. Then, we verify the building for serviceability state in normal conditions and the brace system needed to be reinforced.

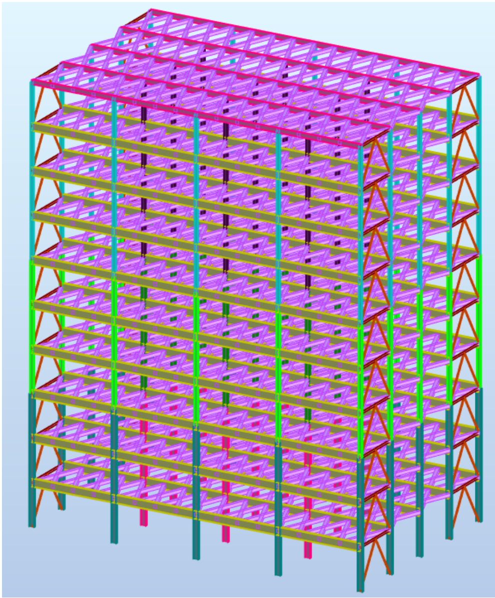


Figure 5-6 Building 2 global model overview for first approach

In the other hand, for building 2, due to the height, the elements from the second floor up did present some stresses, as it can be seen in Figure 5-7. Where we present the diagrams of compression for the worst-case scenario of the accidental combination.

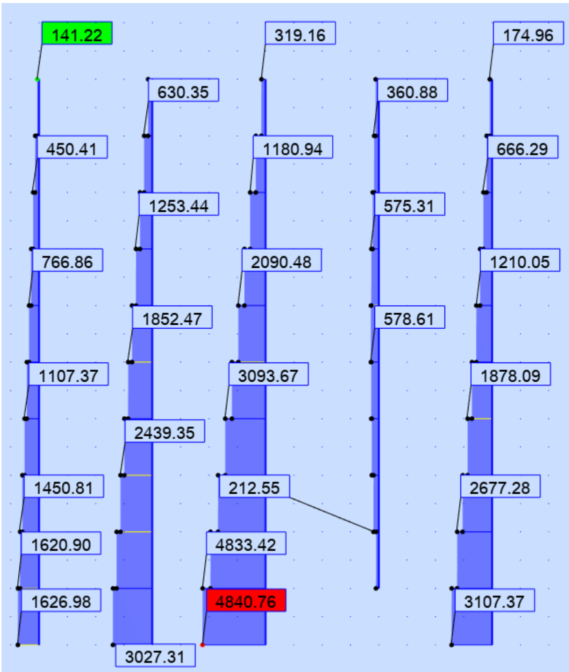


Figure 5-7 Compression results applied to each column for building 2

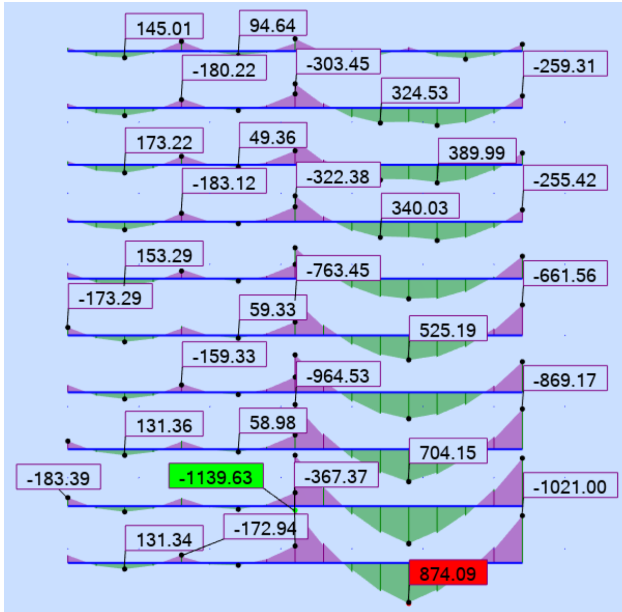


Figure 5-8 Moment results applied to each column for building 2

5.4.2. Second approach: Design with truss superstructure

The main idea in this instance was to have a truss superstructure capable of redistribution of the stresses, caused by the removal of the column, throughout the building. For building 1, column B6 was the one that created the worst condition for the truss superstructure elements. After designing the latter, we proceeded to verify the ultimate limit state and serviceability limit state, since it added over forty thousand kilograms, forcing us to reinforce the structure in order to verify this.

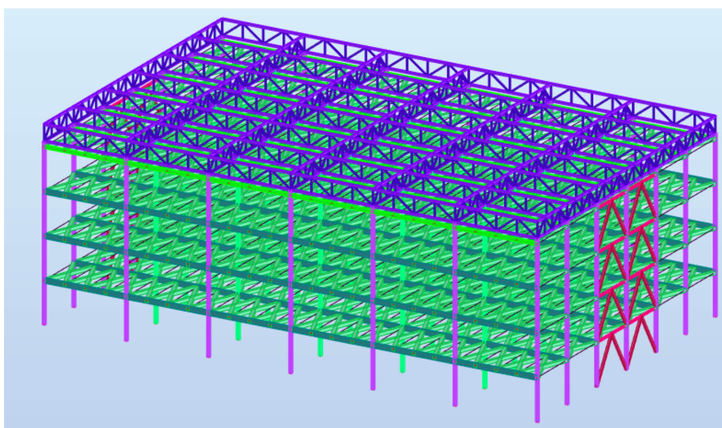


Figure 5-9 Building 1 global model overview for second approach

In Figure 5-10 we present the compression diagram for building 1 for the worst-case scenario for the accidental combination load. This time we notice how the truss superstructure the reaction of the model to re distribute the load. In this case the columns acted as if they were hanging on the truss superstructure and the columns instead of been under compression are under tension, where the steel structure has better behavior.

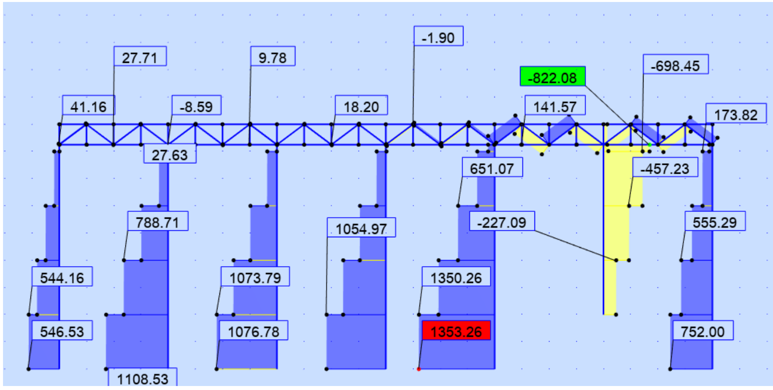


Figure 5-10 Compression diagram applied to each column for building 1

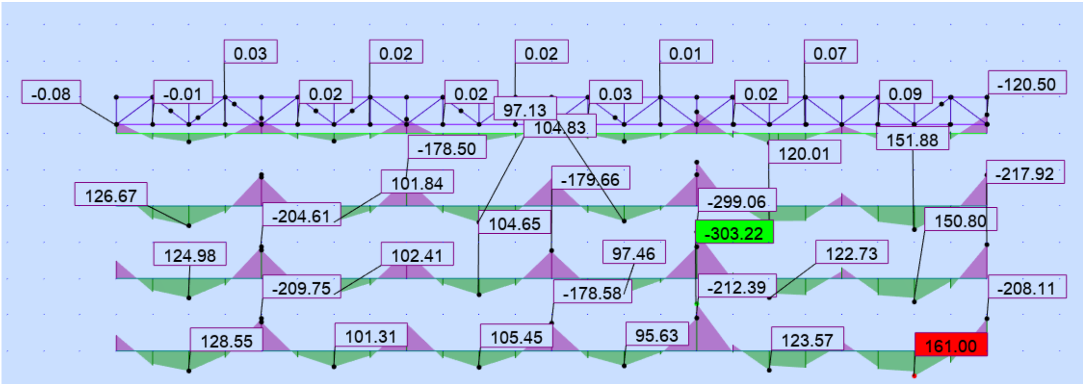


Figure 5-11 Moment diagram applied to each column for building 1

For building 2, column C4 was the one that created the worst condition for the truss superstructure elements. After designing the latter, we proceeded to verify the ultimate limit state and serviceability limit state, and the brace system needed to be reinforced.

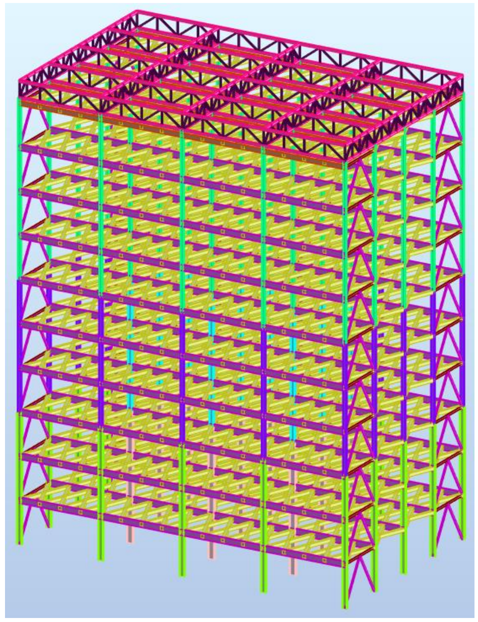


Figure 5-12 Building 2 global model overview for second approach

The same situation was presented for the truss superstructure of building 2, see Figure 5-13. Where we represent the compression diagram for the building. The same situation where the columns seem to be hanging from the truss superstructure making them to work on tensile stress.

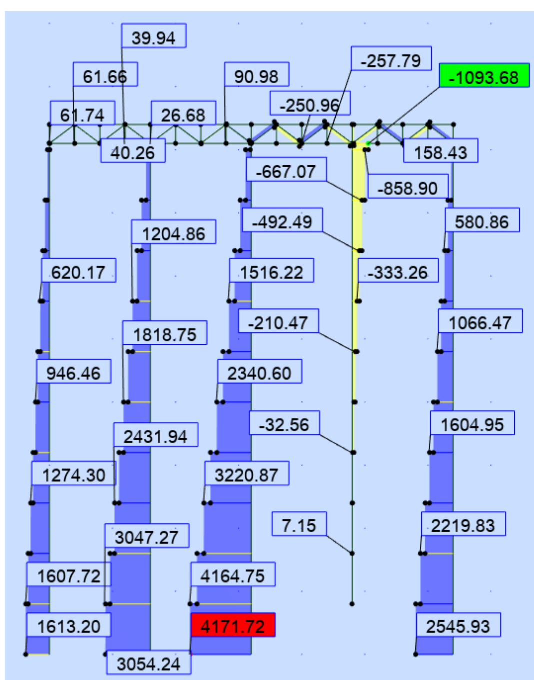


Figure 5-13 Compression results applied to each column for building 2

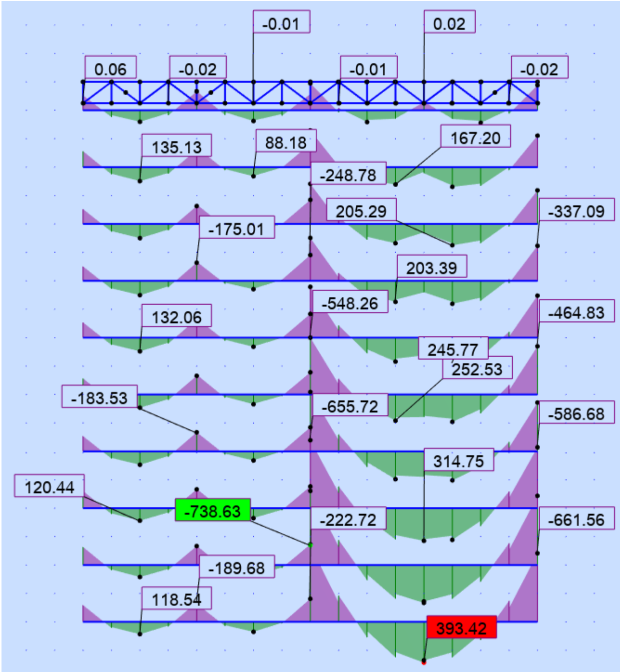


Figure 5-14 Moment results applied to each column for building 2

5.4.2.1. Trusses system

For the truss superstructure, it was selected a truss system on the top because is the most efficient solution, so that we could redistribute the load when removing an element. It can be seen on Figure 5-15 and Figure 5-17 for both buildings how the element works. To come up with the most accurate solution, the structure was modeled using different height for the truss. By doing so, the usage of the elements on the truss varied allowing us to choose the lightest result.

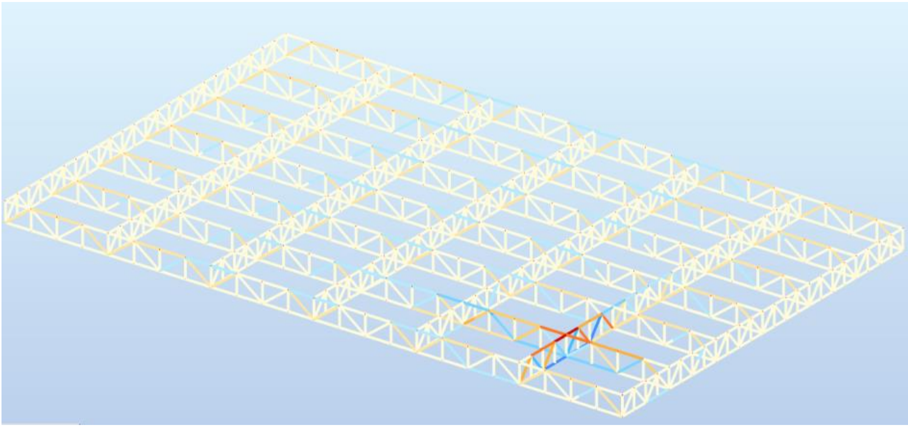


Figure 5-15 Overview of trusses system for building 1

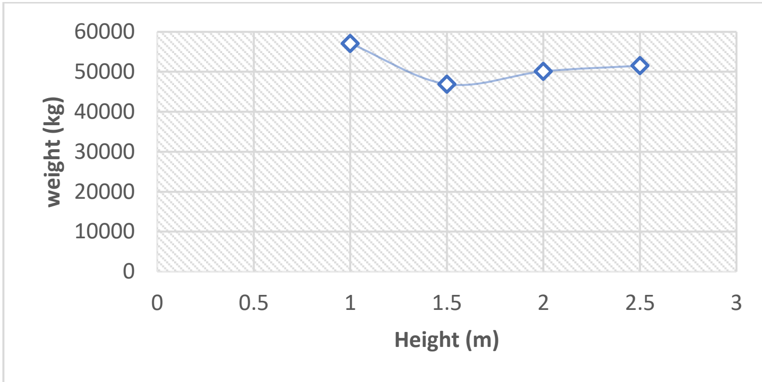


Figure 5-16 Weight vs. Height chart for building 1 trusses

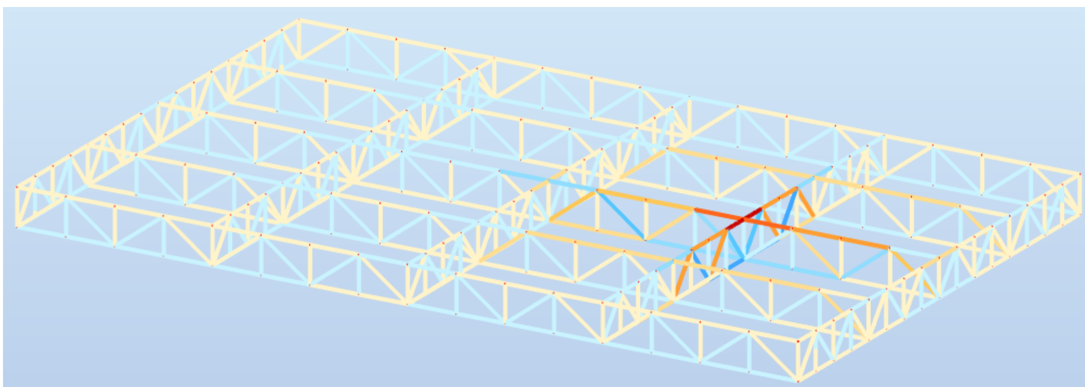


Figure 5-17 Overview of trusses system for building 2

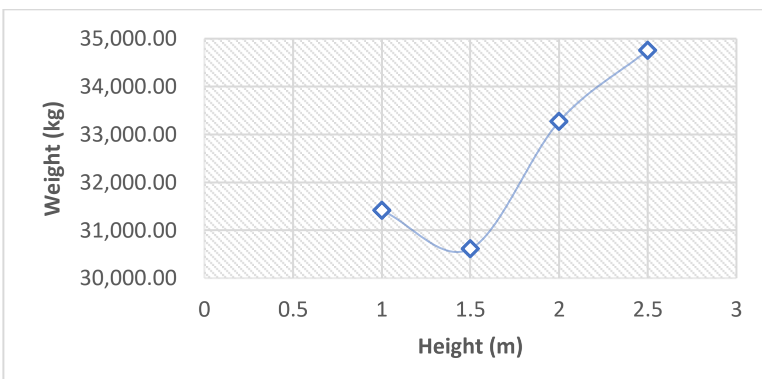


Figure 5-18 Weight vs. Height chart for building 2 trusses

5.5. Connections

The connections were verified, once again, to be able to resist the stress for the accidental combination, with the removal of the column, and the ULS in normal conditions. Also, they were designed to idolize a continuous connection making them rigid. In Figure 5-19 and Figure 5-20 we can see the final solution for the six characteristic connections that were selected to be design. In Table 5-2 and Table 5-3 we can see the final solution rotational stiffness. These connections were designed following EN 1993 part 1-8 and using the help of Robot AutoDesk Structural Analysis.

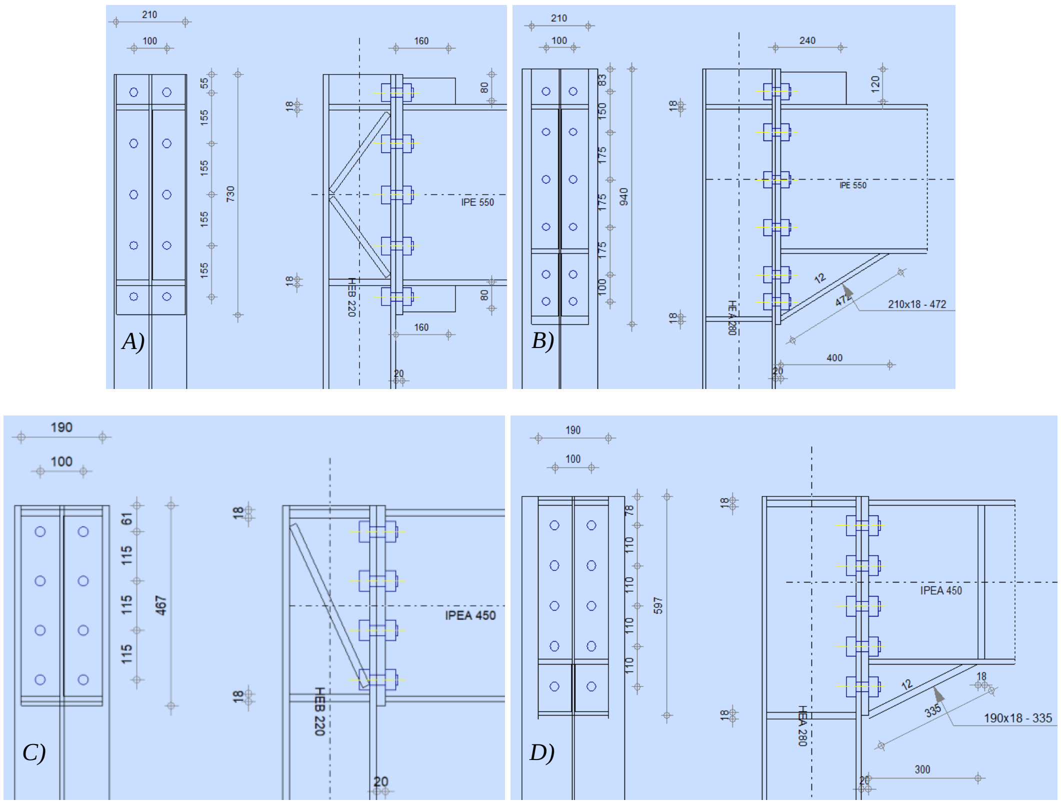
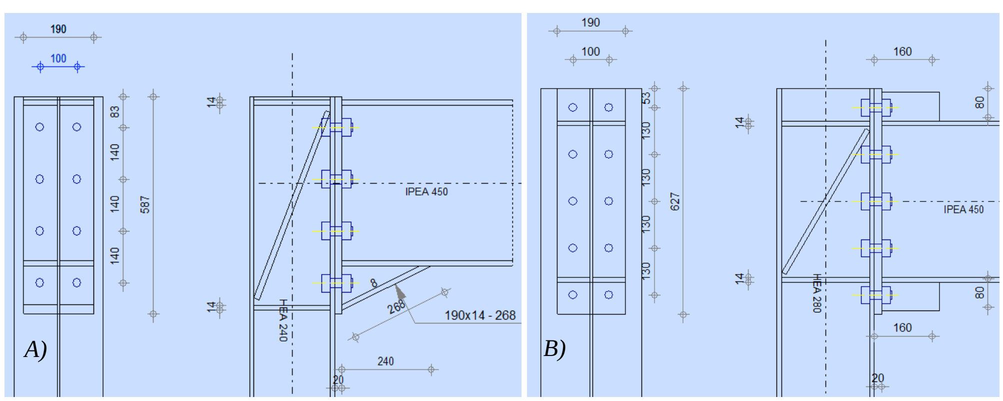


Figure 5-19 Connection drawing building 1-2 - A) Connection 1 - B) Connection 2 – C) Connection 4 – D) Connection 5



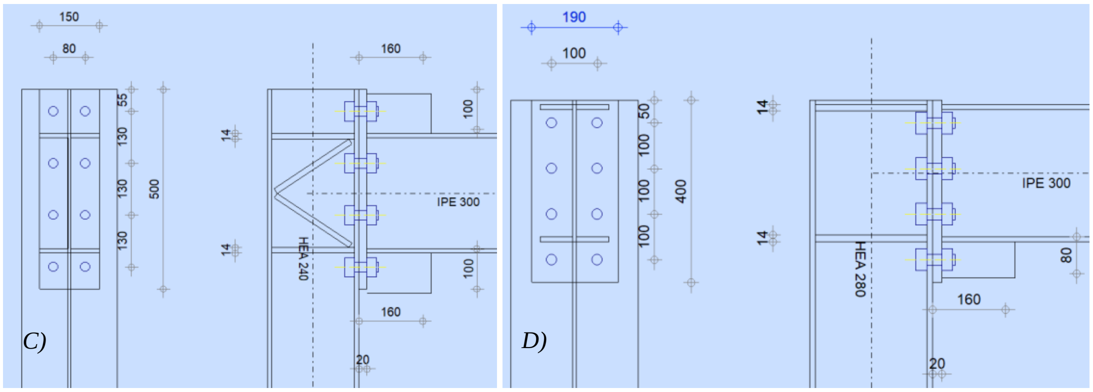


Figure 5-20 Connection drawing building 1-3 - A) Connection 1 - B) Connection 2 – C) Connection 4 – D) Connection 5

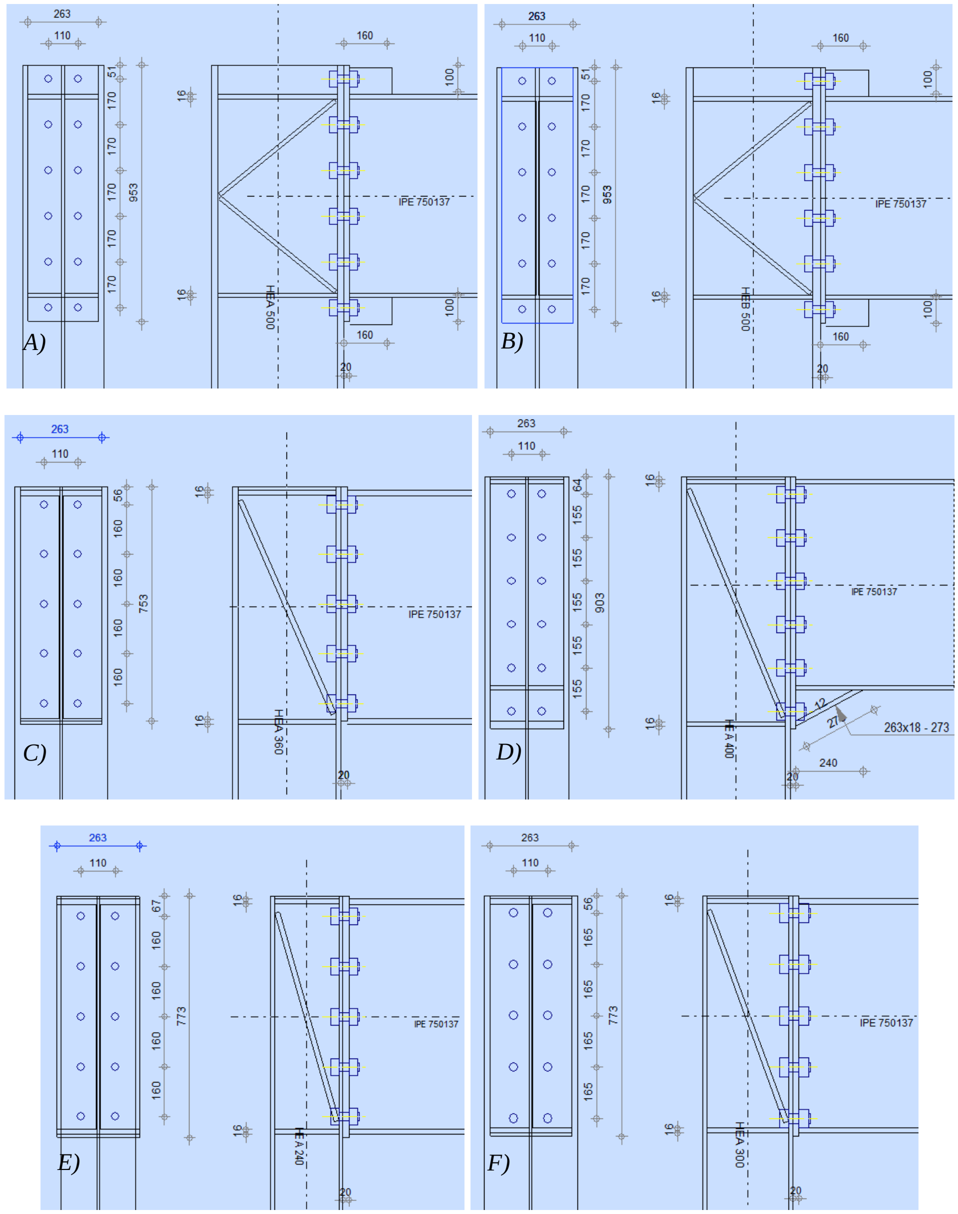
	Bolt	$S_{j,rig}$	$S_{j,pin}$	$S_{j,ini}$	S_j
Connection 1	M27	140,944.65	8,809.04	159,079.60	159,079.60
Connection 2	M30	140,944.65	8,809.04	282,478.47	282,478.47
Connection 3	M30	140,944.65	8,809.04	330,400.87	330,400.87
Connection 4	M24	62,493.48	3,905.84	136,456.80	136,456.80
Connection 5	M27	62,493.48	3,905.84	179,535.60	179,535.60
Connection 6	M27	62,493.48	3,905.84	199,286.69	199,286.69

Table 5-2 Connection rotational stiffness building 1-2 (kNm)

	Bolt	$S_{j,rig}$	$S_{j,pin}$	$S_{j,ini}$	S_j
Connection 1	M24	62,493.48	3,905.84	166,775.86	157,353.02
Connection 2	M24	62,493.48	3,905.84	73,154.28	33,481.41
Connection 3	M24	62,493.48	3,905.84	73,154.28	31,954.08
Connection 4	M24	17,547.83	1,096.74	23,779.72	23,779.72
Connection 5	M24	17,547.83	1,096.74	35,092.47	11,935.99
Connection 6	M24	17,547.83	1,096.74	73,671.73	73,671.73

Table 5-3 Connection rotational stiffness building 1-3 (kNm)

For the second building, we did the same procedure. The solutions can be seen in Table 5-4 and Table 5-5. These values will be used to make comparisons in the conclusion.


A)
IPE 750137
HEA 500
B)
IPE 750137
HEB 500
C)
IPE 750137
HEA 360
D)
IPE 750137
HE A 400
263x18 - 273
E)
IPE 750137
HE A 240
F)
IPE 750137
HEA 300

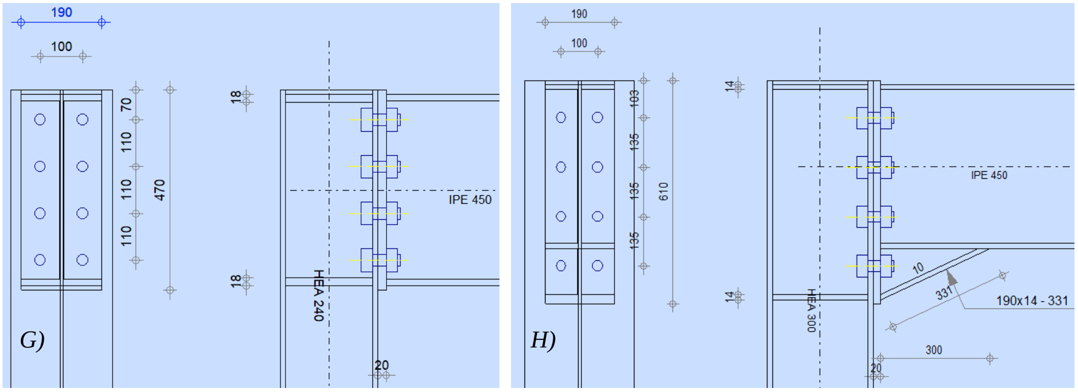
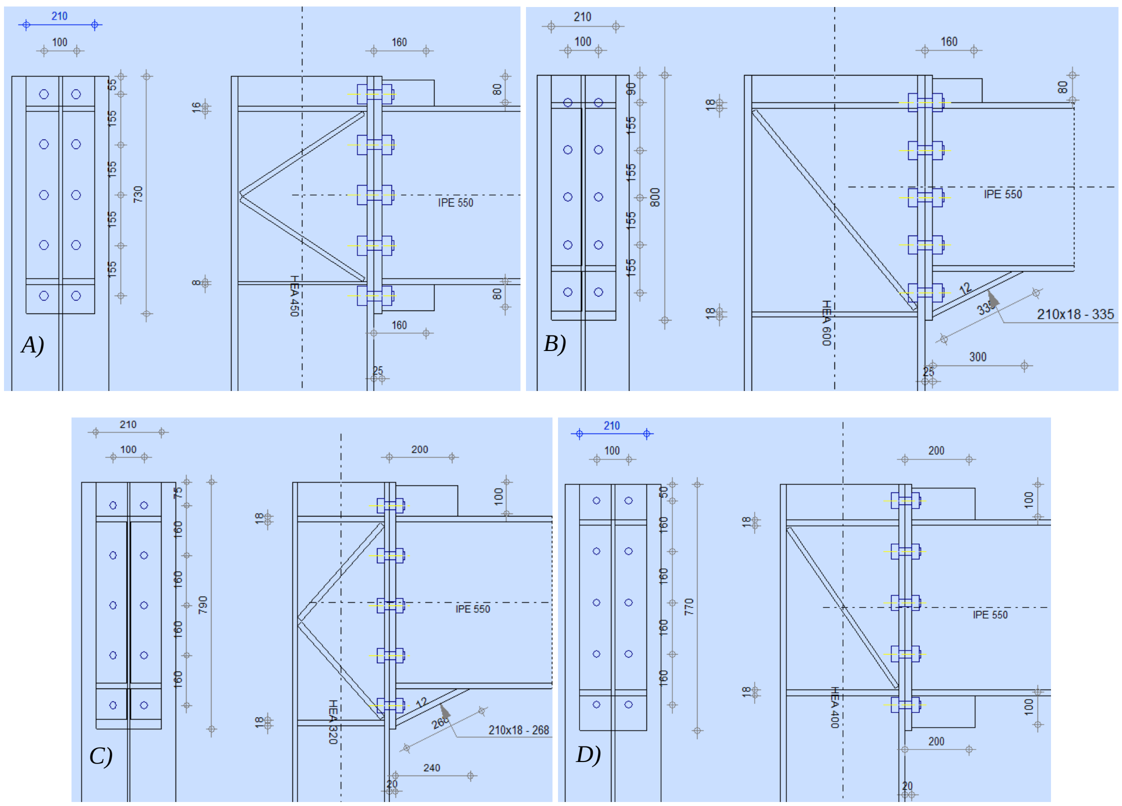


Figure 5-21 Connection drawing building 2-2 - A) Connection 1 - B) Connection 2 – C) Connection 4 – D) Connection 5 – E) Connection 7 – F) Connection 8 – G) Connection 10 – H) Connection 11



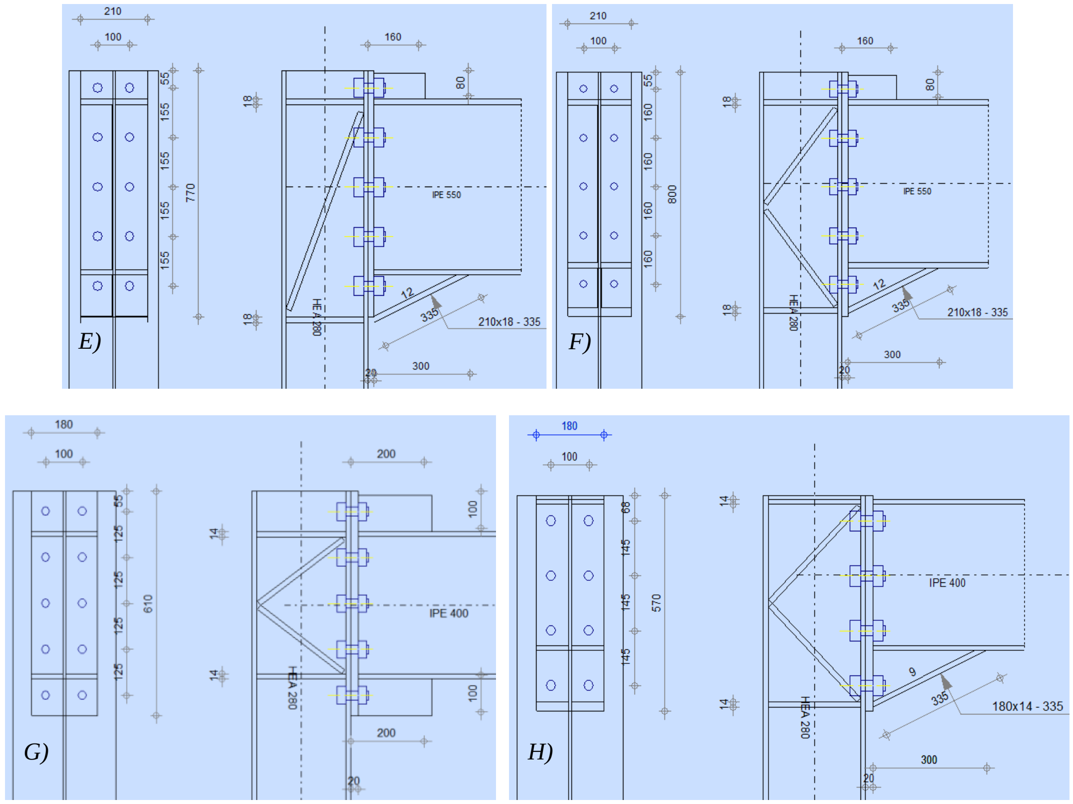


Figure 5-22 Connection drawing building 2-3 - A) Connection 1 - B) Connection 2 – C) Connection 4 – D) Connection 5 – E) Connection 7 – F) Connection 8 – G) Connection 10 – H) Connection 11

	Bolt	$S_{j,rig}$	$S_{j,pin}$	$S_{j,ini}$	S_j
Connection 1	M30	335,741.70	20,983.86	337,425.61	337,425.61
Connection 2	M30	335,741.70	20,983.86	372,792.26	372,792.26
Connection 3	M30	335,741.70	20,983.86	372,792.26	372,792.26
Connection 4	M27	335,741.70	20,983.86	475,297.82	475,297.82
Connection 5	M30	335,741.70	20,983.86	761,955.24	761,955.24
Connection 6	M30	335,741.70	20,983.86	761,955.24	761,955.24
Connection 7	M27	20,983.86	335,741.70	412,201.57	412,201.57
Connection 8	M30	20,983.86	335,741.70	487,049.50	487,049.50
Connection 9	M30	20,983.86	335,741.70	468,852.61	468,852.61
Connection 10	M27	70,860.09	4,428.76	29,392.84	38,149.26
Connection 11	M30	70,860.09	4,428.76	175,425.53	82,881.76
Connection 12	M30	70,860.09	4,428.76	180,778.90	93,209.31

Table 5-4 Connection rotational stiffness building 2-2 (kNm)

	Bolt	$S_{j,rig}$	$S_{j,pin}$	$S_{j,ini}$	S_j
Connection 1	M30	140,944.65	8,809.04	117,071.89	117,071.89
Connection 2	M30	140,944.65	8,809.04	184,738.33	184,738.33
Connection 3	M30	140,944.65	8,809.04	171,293.51	171,293.51
Connection 4	M24	140,944.65	8,809.04	210,953.03	210,953.03
Connection 5	M24	140,944.65	8,809.04	124,151.28	83,819.67
Connection 6	M24	140,944.65	8,809.04	232,641.10	232,641.10
Connection 7	M30	140,944.65	8,809.04	273,636.21	273,636.21
Connection 8	M27	140,944.65	8,809.04	256,444.10	256,444.10
Connection 9	M27	140,944.65	8,809.04	256,444.10	256,444.10
Connection 10	M24	48,569.64	3,035.60	62,450.46	39,194.08
Connection 11	M27	48,569.64	3,035.60	174,574.82	65,515.40
Connection 12	M27	48,569.64	3,035.60	174,574.82	81,138.74

Table 5-5 Connection rotational stiffness building 2-3 (kNm)

5.6. Final solution

With these considerations made for the global and local analysis, the final solution for building 1 and 2 are presented in Table 5-6 and Table 5-7.

Element	**Section 1-1**	**Section 1-2**	**Section 1-3**
Internal Column	HEA 260	HEA 280	HEA 280
External Column	HEA 240	HEB 220	HEA 260
Primary Beam	IPE 400	IPE 550	IPEA 450
Secondary Beam	IPE 140	IPE 140	IPE 140
Primary Beam Roof	IPEA 330	IPEA 450	IPE 330
Brace	SHSC 160x160x4	SHSC 160x160x4	SHSC 160x160x4
Brace Beam	IPE 180	IPE 180	IPE 200
Trusses Beams	-	-	SHSC 180x180x6.3
Trusses Columns	-	-	RHSC 120x80x4

Table 5-6 Final solution for building 1

Element	Section 2-1	Section 2-2	Section 2-3
Internal			
Column 1	HEA 600	HEB 500	HEA 600
Column 2	HEA 400	HEA 400	HEA 400
Column 3	HEA 280	HEA 300	HEA 280
External			
Column 1	HEA 450	HEA 500	HEA 450
Column 2	HEA 320	HEA 340	HEA 320
Column 3	HEA 280	HEA 280	HEA 280
Primary Beam	IPE 450	IPE 750x137	IPE 550
Secondary Beam	IPE 140	IPE 140	IPE 140
Primary Beam Roof	IPE 400	IPE 450	IPE 400
Brace	RHSC 200x120x10	SHSC 120x120x8	RHSC 200x120x10
Brace Beam	IPEA 200	IPEA 200	IPEA 200
Trusses Beams	-	-	SHSC 220x220x6.4
Trusses Columns	-	-	SHSC 120x120x5

Table 5-7 Final solution for building 2

6. Conclusion

Throughout the course of this study, we reviewed and designed two buildings separately following EN 1993 part 1-1 and EN 1994 part 1-1. Each one of them was analyzed three times under three different situations: one in normal conditions; another one applying the accidental loads combination following EN 1991 part 1-7; and a third one applying the latter using a truss superstructure to redistribute the loads. The purpose was to compare sections regarding the structure weights, and compare the beam-to-column connections, as well as the rotational stiffness.

On building 1, regarding structural behavior, the first two conditions had nothing out of the ordinary. This was not the case for the third situation, where the truss superstructure that was placed in order to resist the accidental loads, verified for the accidental combination. However, when it was verified for the ultimate limit state at normal conditions, the structure had to be reinforced due to the added weight of the truss superstructure.

On the other hand, for building 2, the only hazard was to control its total vertical deformation. We believe that using a different arrangement could have given a more efficient structure. In the case of building 2-3, despite of adding weight due to the truss superstructure, it was not required its reinforcement in order to resist the loads.

Figure 6-1 shows a summary of the total weight of the buildings in all three approaches. It can be seen how the re-design of building 1-3 made its total solution the heavier one. For building 2, not having the superstructure represented a heavier result.

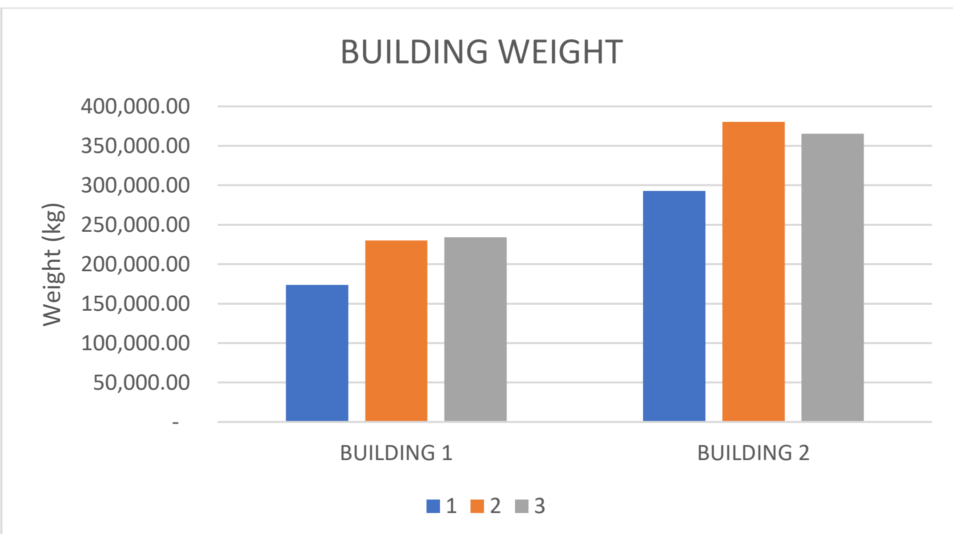


Figure 6-1 Weight comparison between buildings

Regarding the connections, due to the deformation that occurs when a column is withdrawn from building and the increase of the momentum happens, there needs to be a reinforcement in order to be capable of redistributing the load, as well as maintaining its rigidity. In this regard for building 1, as it can be seen in Figure 6-2, building 1-2 had to be the most reinforced one to withstand the loads. In the other hand, due the truss superstructure and the re design of the structure, building 1-3 was favored and the rigidity of the connection was lowered.

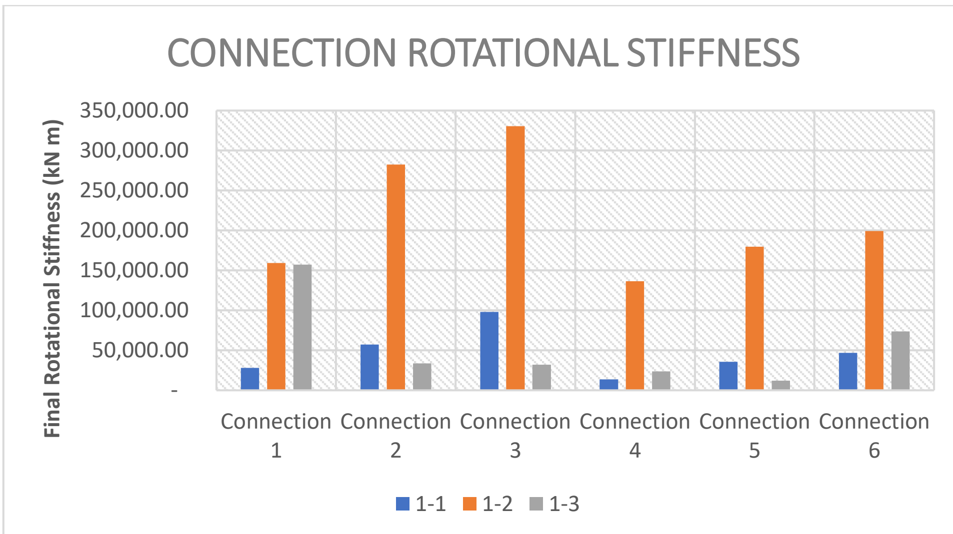


Figure 6-2 Rotational stiffness comparison for building 1

Figure 6-3 represents the third connection for all three approaches for building 1. This was the one that presented the bigger change through the process, which is a vivid example of the impact the removal of a column has on a connection. Making the connection more laborious and costly.

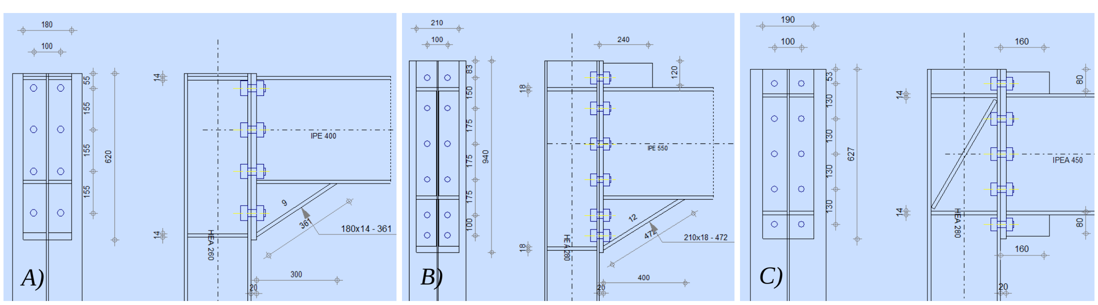


Figure 6-3 Connection 3 for building 1 - A) Building 1-1 B) Building 1-2 C) Building 1-3

For Figure 6-4, the same comparison for the connection rotational stiffness was made for building 2. Once again, the connections for the second approach were the one with the highest rotational stiffness. It had to be reinforced due to the deformation of the elements and the moment they had to resist.

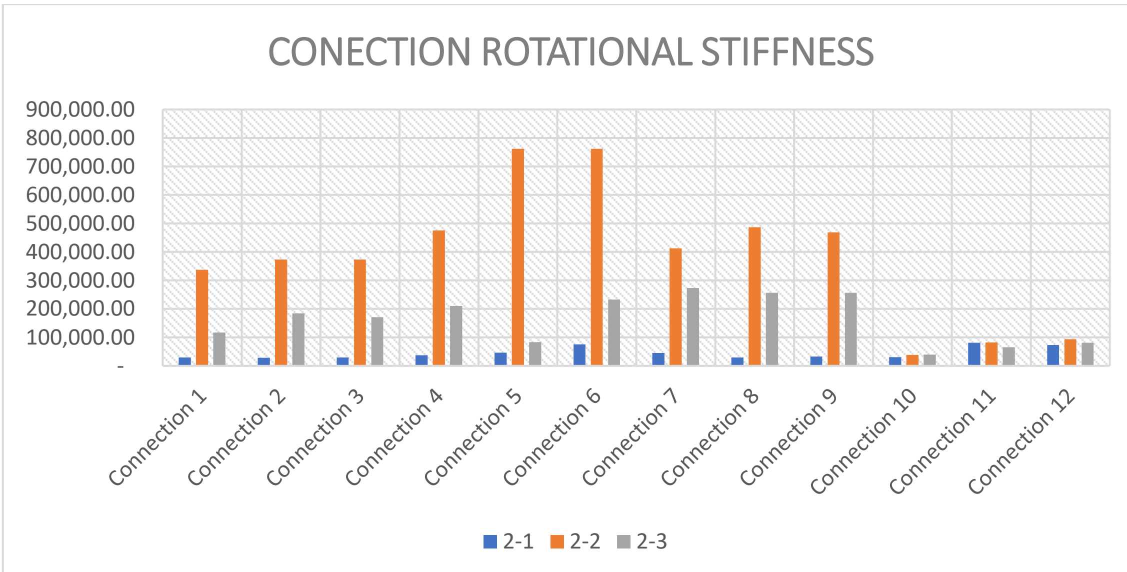


Figure 6-4 Rotational stiffness comparison for building 2

Having completed the analysis on this structural behavior, we recommend the expansion of the investigation in the following topics:

- Beam-to-column joints
- Steal-to-concrete connections
- Different superstructure
- Different brace system and different locations
- Develop other numerical structure with different categorization and height to compare solutions

Regarding the joints, from time to time it was needed to embed a bolt on the concrete, to increase the moment of the resistance. However, the composite behavior effect was not considered in this study and it is a topic that could be broadened. This could result in less robust connections.

As for the primary beam, it was not considered a composite structure for the ultimate limit state. This would have helped the analyses of robustness, due to the mechanical characteristics of the beam working on favor of the structure.

For this thesis only one type of truss superstructure was verified. This is the reason why we recommend evaluating other solutions that could lead to similar results, in order to evaluate which of them has higher efficiency. This other solution can include horizontal ties that can minimize or redistribute the loads in each floor. Even though the brace system did not have an important role when designing for robustness, they did work as an alternative path for the load when a column nearby was removed.

By increasing the number of studies or examples regarding this matter, it could bring up a pattern to be used in other structural designs, allowing the addition of new details to the current norms.

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8. Annex

Column calculation example

STEEL DESIGN

CODE: *EN 1993-1:2005/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 23 Column_23 **POINT:** 3 **COORDINATE:** x = 0.50 L = 2.00 m

---

LOADS:
Governing Load Case: 130 ACCIDENTAL_Qsv_Wy_DES (1+2+3+4+5)*1.00+(8+12)*0.20+14*0.70

MATERIAL:
S 355 (S 355) fy = 355.00 MPa

SECTION PARAMETERS: HEA 600

h=59.0 cm	gM0=1.00	gM1=1.00	
b=30.0 cm	Ay=163.28 cm2	Az=93.21 cm2	Ax=226.46 cm2
tw=1.3 cm	Iy=141208.00 cm4	Iz=11271.30 cm4	Ix=440.00 cm4
tf=2.5 cm	Wely=4786.71 cm3	Welz=751.42 cm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 4174.70 kN	My,Ed = -43.69 kN*m		
Nc,Rd = 8039.26 kN	My,Ed,max = -232.06 kN*m		
Nb,Rd = 6105.44 kN	My,c,Rd = 1699.28 kN*m		Vz,Ed = -94.19 kN
			Vz,c,Rd = 1910.39 kN
	Mb,Rd = 1634.13 kN*m		
			Class of section = 3

LATERAL BUCKLING PARAMETERS:

z = 0.00	Mcr = 12722.11 kN*m	Curve,LT - a	XLT = 0.96
Lcr,low=4.00 m	Lam_LT = 0.37	fi,LT = 0.58	

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 4.00 m	Lam_y = 0.21	Lz = 4.00 m	Lam_z = 0.74
Lcr,y = 4.00 m	Xy = 1.00	Lcr,z = 4.00 m	Xz = 0.76
Lamy = 16.02	kyy = 0.96	Lamz = 56.70	kzy = 0.96
Torsional buckling:		Flexural-torsional buckling	
Curve,T=b	alfa,T=0.34	Curve,TF=b	alfa,TF=0.34

Lt=4.00 m	fi,T=0.75	Ncr,y=182918.81 kN	fi,TF=0.52
Ncr,T=22583.68 kN	X,T=0.84	Ncr,TF=182918.81 kN	X,TF=1.00
Lam_T=0.60	Nb,T,Rd=6742.87 kN	Lam_TF=0.21	Nb,TF,Rd=8011.79 kN

VERIFICATION FORMULAS:
Section strength check:
My,Ed/My,c,Rd = 0.03 < 1.00 (6.2.5.(1))
N,Ed/Nc,Rd + My,Ed/My,c,Rd = 0.55 < 1.00 (6.2.1(7))
sqrt(Sig,x,Ed*^2 + 3*Tau,z,Ed^2)/(fy/gM0) = 0.55 < 1.00 (6.2.1.(5))
Vz,Ed/Vz,c,Rd = 0.05 < 1.00 (6.2.6.(1))
Global stability check of member:
Lambda,y = 16.02 < Lambda,max = 210.00 Lambda,z = 56.70 < Lambda,max = 210.00 STABLE
N,Ed/Min(Nb,Rd,Nb,T,Rd,Nb,TF,Rd) = 0.68 < 1.00 (6.3.1)
My,Ed,max/Mb,Rd = 0.14 < 1.00 (6.3.2.1.(1))
N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) = 0.66 < 1.00 (6.3.3.(4))
N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) = 0.82 < 1.00 (6.3.3.(4))

Section OK !!!

Column calculation example

STEEL DESIGN

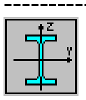
CODE: *EN 1993-1:2005/A1:2014, Eurocode 3: Design of steel structures.*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 6024 Beam_6024 **POINT:** 5 **COORDINATE:** x = 1.00 L = 8.00 m

---

LOADS:
Governing Load Case: 128 ACCIDENTAL_Qsv_Wx_DES (1+2+3+4+5)*1.00+(8+10)*0.20+14*0.70

MATERIAL:
S 355 (S 355) fy = 355.00 MPa


SECTION PARAMETERS: IPE 550

h=55.0 cm	gM0=1.00	gM1=1.00	
b=21.0 cm	Ay=82.51 cm2	Az=72.34 cm2	Ax=134.42 cm2
tw=1.1 cm	Iy=67116.50 cm4	Iz=2667.58 cm4	Ix=118.40 cm4
tf=1.7 cm	Wply=2787.21 cm3	Wplz=400.55 cm3	

INTERNAL FORCES AND CAPACITIES:

N,Ed = 32.24 kN	My,Ed = -405.13 kN*m	Mz,Ed = 0.00 kN*m	Vy,Ed = -0.01 kN
Nc,Rd = 4771.77 kN	My,Ed,max = -405.13 kN*m		Mz,Ed,max = -0.03 kN*m
	Vy,T,Rd = 1691.00 kN		
Nb,Rd = 4023.81 kN	My,c,Rd = 989.46 kN*m	Mz,c,Rd = 142.19 kN*m	Vz,Ed = -198.71 kN

	MN,y,Rd = 989.46 kN*m	MN,z,Rd = 142.19 kN*m	Vz,T,Rd = 1482.61 kN
	Mb,Rd = 781.83 kN*m		Tt,Ed = 0.00 kN*m
			Class of section = 1

LATERAL BUCKLING PARAMETERS:

z = 1.00	Mcr = 1468.89 kN*m	Curve,LT - c	XLT = 0.75
Lcr,low=2.00 m	Lam_LT = 0.82	fi,LT = 0.86	XLT,mod = 0.79

BUCKLING PARAMETERS:

About y axis:		About z axis:	
Ly = 8.00 m	Lam_y = 0.47	Lz = 2.00 m	Lam_z = 0.59
Lcr,y = 8.00 m	Xy = 0.93	Lcr,z = 2.00 m	Xz = 0.84
Lamy = 35.80	kzy = 1.00	Lamz = 44.89	kzz = 0.90

Torsional buckling:		Flexural-torsional buckling	
Curve,T=b	alfa,T=0.34	Curve,TF=b	alfa,TF=0.34
Lt=2.00 m	fi,T=0.66	Ncr,y=21735.44 kN	fi,TF=0.66
Ncr,T=20736.69 kN	X,T=0.89	Ncr,TF=21735.44 kN	X,TF=0.90
Lam_T=0.48	Nb,T,Rd=4261.66 kN	Lam_TF=0.47	Nb,TF,Rd=4284.51 kN

VERIFICATION FORMULAS:

Section strength check:

N,Ed/Nc,Rd = 0.01 < 1.00 (6.2.4.(1))

(My,Ed/MN,y,Rd)^ 2.00 + (Mz,Ed/MN,z,Rd)^1.00 = 0.17 < 1.00 (6.2.9.1.(6))

Vy,Ed/Vy,T,Rd = 0.00 < 1.00 (6.2.6-7)

Vz,Ed/Vz,T,Rd = 0.13 < 1.00 (6.2.6-7)

Tau,ty,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Tau,tz,Ed/(fy/(sqrt(3)*gM0)) = 0.00 < 1.00 (6.2.6)

Global stability check of member:

Lambda,y = 35.80 < Lambda,max = 210.00 Lambda,z = 44.89 < Lambda,max = 210.00 STABLE

N,Ed/Min(Nb,Rd,Nb,T,Rd,Nb,TF,Rd) = 0.01 < 1.00 (6.3.1)

My,Ed,max/Mb,Rd = 0.52 < 1.00 (6.3.2.1.(1))

N,Ed/(Xy*N,Rk/gM1) + kyy*My,Ed,max/(XLT*My,Rk/gM1) + kyz*Mz,Ed,max/(Mz,Rk/gM1) = 0.47 < 1.00 (6.3.3.(4))

N,Ed/(Xz*N,Rk/gM1) + kzy*My,Ed,max/(XLT*My,Rk/gM1) + kzz*Mz,Ed,max/(Mz,Rk/gM1) = 0.53 < 1.00 (6.3.3.(4))

Section OK !!!

Connection calculation example

	Autodesk Robot Structural Analysis Professional 2019 **Design of fixed beam-to-column connection** EN 1993-1-8:2005/AC:2009	Ratio **0.95**

General

Connection no.: 1

Connection name: Frame knee

Structure node: 13017

Structure bars: 25, 1024

Geometry

Column

Section:	HEA 450		
Bar no.:	25		
a =	-90.0	[Deg]	Inclination angle
h_c =	440	[mm]	Height of column section
b_{fc} =	300	[mm]	Width of column section
t_{wc} =	12	[mm]	Thickness of the web of column section
t_{fc} =	21	[mm]	Thickness of the flange of column section
r_c =	27	[mm]	Radius of column section fillet
A_c =	178.03	[cm^2]	Cross-sectional area of a column
I_{xc} =	63721.60	[cm^4]	Moment of inertia of the column section
Material:	S 355		
f_{yc} =	355.00	[MPa]	Resistance

Beam

Section:	IPE 550		
Bar no.:	1024		
a =	-0.0	[Deg]	Inclination angle
h_b =	550	[mm]	Height of beam section
b_f =	210	[mm]	Width of beam section
t_{wb} =	11	[mm]	Thickness of the web of beam section
t_{fb} =	17	[mm]	Thickness of the flange of beam section
r_b =	24	[mm]	Radius of beam section fillet
r_b =	24	[mm]	Radius of beam section fillet

a =	-0.0	[Deg]	Inclination angle
A_b =	134.42	[cm^2]	Cross-sectional area of a beam
I_{xb} =	67116.50	[cm^4]	Moment of inertia of the beam section
Material:	S 355		
f_{yb} =	355.00	[MPa]	Resistance

Bolts

The shear plane passes through the UNTHREADED portion of the bolt.

d =	30	[mm]	Bolt diameter
Class =	8.8		Bolt class
F_{tRd} =	323.14	[kN]	Tensile resistance of a bolt
n_h =	2		Number of bolt columns
n_v =	5		Number of bolt rows
h_1 =	55	[mm]	Distance between first bolt and upper edge of front plate

Horizontal spacing e_i = 100 [mm]

Vertical spacing p_i = 155;155;155;155 [mm]

Plate

h_p =	730	[mm]	Plate height
b_p =	210	[mm]	Plate width
t_p =	25	[mm]	Plate thickness
Material:	S 355		
f_{yp} =	355.00	[MPa]	Resistance

Upper stiffener

h_u = 80 [mm] Stiffener height

t_{wu} = 12 [mm] Thickness of vertical stiffener

l_u = 160 [mm] Length of vertical stiffener

Material: S 355

f_{yu} = 355.00 [MPa] Resistance

Lower stiffener

h_d = 80 [mm] Stiffener height

t_{wd} = 12 [mm] Thickness of vertical stiffener

l_d = 160 [mm] Length of vertical stiffener

Material: S 355

f_{ybu} = 355.00 [MPa] Resistance

Column stiffener

Upper

h_{su} = 398 [mm] Stiffener height

b_{su} = 144 [mm] Stiffener width

t_{hu} = 16 [mm] Stiffener thickness

Material: S 355

f_{ysu} = 355.00 [MPa] Resistance

Lower

h_{sd} = 398 [mm] Stiffener height

b_{sd} = 144 [mm] Stiffener width

h_{sd} = 398 [mm] Stiffener height

t_{hd} = 8 [mm] Stiffener thickness

Material: S 355

f_{ysu} = 355.00 [MPa] Resistance

Diagonal stiffener

Typ: Double

w_a = 144 [mm] Width of diagonal stiffener

t_a = 16 [mm] Thickness of diagonal stiffener

Material: S 355

f_{ya} = 355.00 [MPa] Resistance

Fillet welds

a_w = 8 [mm] Web weld

a_f = 13 [mm] Flange weld

a_s = 8 [mm] Stiffener weld

Material factors

g_{M0} = 1.00 Partial safety factor [2.2]

g_{M1} = 1.00 Partial safety factor [2.2]

g_{M2} = 1.25 Partial safety factor [2.2]

g_{M3} = 1.25 Partial safety factor [2.2]

Loads

Ultimate limit state

Case: 128: ACCIDENTAL_Qsv_Wx_DES (1+2+3+4+5)*1.00+(8+10)*0.20+14*0.70

$M_{b1,Ed}$ =	661.58	[kN*m]	Bending moment in the right beam
$V_{b1,Ed}$ =	265.35	[kN]	Shear force in the right beam
$N_{b1,Ed}$ =	55.72	[kN]	Axial force in the right beam
$M_{c1,Ed}$ =	-284.62	[kN*m]	Bending moment in the lower column
$V_{c1,Ed}$ =	-107.21	[kN]	Shear force in the lower column
$N_{c1,Ed}$ =	-2540.59	[kN]	Axial force in the lower column
$M_{c2,Ed}$ =	368.25	[kN*m]	Bending moment in the upper column
$V_{c2,Ed}$ =	175.00	[kN]	Shear force in the upper column
$N_{c2,Ed}$ =	-2225.46	[kN]	Axial force in the upper column

Results

Beam resistances

TENSION

A_b =	134.42	[cm^2]	Area	EN1993-1-1:[6.2.3]

$N_{tb,Rd} = A_b f_{yb} / g_{M0}$

$N_{tb,Rd}$ =	4771.77	[kN]	Design tensile resistance of the section	EN1993-1-1:[6.2.3]

SHEAR

A_{vb} =	91.54	[cm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]

$V_{cb,Rd} = A_{vb} (f_{yb}$ / Ö3) / g_{M0}

$V_{cb,Rd}$ =	1876.22	[kN]	Design sectional resistance for shear	EN1993-1-1:[6.2.6.(2)]

$V_{b1,Ed} / V_{cb,Rd} \leq 1,0$	0.14 < 1.00	verified	(0.14)

BENDING - PLASTIC MOMENT (WITHOUT BRACKETS)

W_{plb} =	2787.21	[cm^3]	Plastic section modulus	EN1993-1-1:[6.2.5.(2)]

$M_{b,pl,Rd} = W_{plb}\, f_{yb} / g_{M0}$

$M_{b,pl,Rd}$ =	989.46	[kN*m]	Plastic resistance of the section for bending (without stiffeners)	EN1993-1-1:[6.2.5.(2)]

BENDING ON THE CONTACT SURFACE WITH PLATE OR CONNECTED ELEMENT

W_{pl} =	2787.21	[cm^3]	Plastic section modulus	EN1993-1-1:[6.2.5]

$M_{cb,Rd} = W_{pl}\, f_{yb} / g_{M0}$

$M_{cb,Rd}$ =	989.46	[kN*m]	Design resistance of the section for bending	EN1993-1-1:[6.2.5]

FLANGE AND WEB - COMPRESSION

$M_{cb,Rd}$ =	989.46	[kN*m]	Design resistance of the section for bending	EN1993-1-1:[6.2.5]
h_f =	533	[mm]	Distance between the centroids of flanges	[6.2.6.7.(1)]

$F_{c,fb,Rd} = M_{cb,Rd} / h_f$

$F_{c,fb,Rd}$ =	1857.09	[kN]	Resistance of the compressed flange and web	[6.2.6.7.(1)]

Column resistances

WEB PANEL - SHEAR

$M_{b1,Ed}$ =	661.58	[kN*m]	Bending moment (right beam)	[5.3.(3)]
$M_{b2,Ed}$ =	0.00	[kN*m]	Bending moment (left beam)	[5.3.(3)]
$V_{c1,Ed}$ =	-107.21	[kN]	Shear force (lower column)	[5.3.(3)]

WEB PANEL - SHEAR

$M_{b1,Ed}$ =	661.58	[kN*m]	Bending moment (right beam)	[5.3.(3)]
$V_{c2,Ed}$ =	175.00	[kN]	Shear force (upper column)	[5.3.(3)]
z =	499	[mm]	Lever arm	[6.2.5]

$V_{wp,Ed} = (M_{b1,Ed} - M_{b2,Ed}) / z - (V_{c1,Ed} - V_{c2,Ed}) / 2$

$V_{wp,Ed}$ =	1467.18	[kN]	Shear force acting on the web panel	[5.3.(3)]
A_{vs} =	65.78	[cm^2]	Shear area of the column web	EN1993-1-1:[6.2.6.(3)]
A_{vd} =	39.14	[cm^2]	Area of the diagonal stiffener subjected to shear	EN1993-1-1:[6.2.6.(3)]
A_{vc} =	104.93	[cm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]
d_s =	538	[mm]	Distance between the centroids of stiffeners	[6.2.6.1.(4)]
$M_{pl,fc,Rd}$ =	11.74	[kN*m]	Plastic resistance of the column flange for bending	[6.2.6.1.(4)]
$M_{pl,stu,Rd}$ =	6.82	[kN*m]	Plastic resistance of the upper transverse stiffener for bending	[6.2.6.1.(4)]
$M_{pl,stl,Rd}$ =	1.70	[kN*m]	Plastic resistance of the lower transverse stiffener for bending	[6.2.6.1.(4)]

$V_{wp,Rd} = 0.9\ (A_{vs}*f_{y,wc}+A_{vd}*f_{ya}) / (Ö3\ g_{M0}) + Min(4\ M_{pl,fc,Rd} / d_s\ , (2\ M_{pl,fc,Rd} + M_{pl,stu,Rd} + M_{pl,stl,Rd}) / d_s)$

$V_{wp,Rd}$ =	1995.00	[kN]	Resistance of the column web panel for shear	[6.2.6.1]

$V_{wp,Ed} / V_{wp,Rd} \leq 1,0$	0.74 < 1.00	verified	(0.74)

WEB - TRANSVERSE COMPRESSION - LEVEL OF THE BEAM BOTTOM FLANGE

Bearing:

t_{wc} =	12	[mm]	Effective thickness of the column web	[6.2.6.2.(6)]
$b_{eff,c,wc}$ =	344	[mm]	Effective width of the web for compression	[6.2.6.2.(1)]
A_{vc} =	65.78	[cm^2]	Shear area	EN1993-1-1:[6.2.6.(3)]
w =	0.82		Reduction factor for interaction with shear	[6.2.6.2.(1)]
$s_{com,Ed}$ =	224.41	[MPa]	Maximum compressive stress in web	[6.2.6.2.(2)]
k_{wc} =	1.00		Reduction factor conditioned by compressive stresses	[6.2.6.2.(2)]
A_s =	17.31	[cm^2]	Area of the web stiffener	EN1993-1-1:[6.2.4]
a =	32.0	[Deg]	Inclination angle of a diagonal stiffener	
A_{sd} =	46.16	[cm^2]	Diagonal stiffener area	EN1993-1-1:[6.2.4]

$F_{c,wc,Rd1} = w\ k_{wc}\ b_{eff,c,wc}\ t_{wc}\ f_{yc} / g_{M0} + A_s\ f_{ys} / g_{M0} + A_{sd} \cos(a)\ f_{ya} / g_{M0}$

$F_{c,wc,Rd1}$ =	3162.44	[kN]	Column web resistance	[6.2.6.2.(1)]

Buckling:

d_{wc} =	344	[mm]	Height of compressed web	[6.2.6.2.(1)]
l_p =	1.15		Plate slenderness of an element	[6.2.6.2.(1)]
r =	0.72		Reduction factor for element buckling	[6.2.6.2.(1)]
l_s =	3.90		Stiffener slenderness	EN1993-1-1:[6.3.1.2]
c_s =	1.00		Buckling coefficient of the stiffener	EN1993-1-1:[6.3.1.2]
l_{sd} =	4.59		Diagonal stiffener slenderness	EN1993-1-1:[6.3.1.2]
c_{sd} =	1.00		Buckling coefficient of a diagonal stiffener	EN1993-1-1:[6.3.1.2]

$F_{c,wc,Rd2} = w\ k_{wc}\ r\ b_{eff,c,wc}\ t_{wc}\ f_{yc} / g_{M1} + A_s\ c_s\ f_{ys} / g_{M1} + A_{sd}\ c_{sd} \cos(a)\ f_{ya} / g_{M1}$

$F_{c,wc,Rd2}$ = 2838.39 [kN] Column web resistance [6.2.6.2.(1)]

Final resistance:

$F_{c,wc,Rd,low} = \mathrm{Min}\ (F_{c,wc,Rd1}\ , F_{c,wc,Rd2})$

$F_{c,wc,Rd}$ = 2838.39 [kN] Column web resistance [6.2.6.2.(1)]

Geometrical parameters of a connection

EFFECTIVE LENGTHS AND PARAMETERS - COLUMN FLANGE

Nr	**m**	**m_x**	**e**	**e_x**	**p**	**$l_{eff,cp}$**	**$l_{eff,nc}$**	**$l_{eff,1}$**	**$l_{eff,2}$**	**$l_{eff,cp,g}$**	**$l_{eff,nc,g}$**	**$l_{eff,1,g}$**	**$l_{eff,2,g}$**
1	23	-	100	-	87	142	128	128	128	0	0	0	0
2	23	-	100	-	155	142	181	142	181	226	151	151	151
3	23	-	100	-	155	142	216	142	216	310	155	155	155
4	23	-	100	-	155	142	181	142	181	226	151	151	151
5	23	-	100	-	87	142	181	142	181	158	117	117	117

EFFECTIVE LENGTHS AND PARAMETERS - FRONT PLATE

Nr	**m**	**m_x**	**e**	**e_x**	**p**	**$l_{eff,cp}$**	**$l_{eff,nc}$**	**$l_{eff,1}$**	**$l_{eff,2}$**	**$l_{eff,cp,g}$**	**$l_{eff,nc,g}$**	**$l_{eff,1,g}$**	**$l_{eff,2,g}$**
1	35	-	55	-	87	220	275	220	275	197	214	197	214
2	35	-	55	-	155	222	214	214	214	266	186	186	186
3	35	-	55	-	155	222	210	210	210	310	155	155	155
4	35	-	55	-	155	222	210	210	210	266	183	183	183
5	35	-	55	-	87	220	275	220	275	197	214	197	214

m – Bolt distance from the web

m_x – Bolt distance from the beam flange

e – Bolt distance from the outer edge

e_x – Bolt distance from the horizontal outer edge

p – Distance between bolts

$l_{eff,cp}$ – Effective length for a single bolt in the circular failure mode

$l_{eff,nc}$ – Effective length for a single bolt in the non-circular failure mode

$l_{eff,1}$ – Effective length for a single bolt for mode 1

$l_{eff,2}$ – Effective length for a single bolt for mode 2

$l_{eff,cp,g}$ – Effective length for a group of bolts in the circular failure mode

$l_{eff,nc,g}$ – Effective length for a group of bolts in the non-circular failure mode

$l_{eff,1,g}$ – Effective length for a group of bolts for mode 1

$l_{eff,2,g}$ – Effective length for a group of bolts for mode 2

Connection resistance for tension

$F_{t,Rd}$ =	323.14	[kN]	Bolt resistance for tension	[Table 3.4]
$B_{p,Rd}$ =	669.76	[kN]	Punching shear resistance of a bolt	[Table 3.4]

$N_{j,Rd} = \text{Min}\ (N_{tb,Rd}\ ,\ n_v\ n_h\ F_{t,Rd}\ ,\ n_v\ n_h\ B_{p,Rd})$

$N_{j,Rd}$ =	3231.36	[kN]	Connection resistance for tension	[6.2]

$N_{b1,Ed}\ /\ N_{j,Rd} \leq 1{,}0$ 0.02 < 1.00 verified (0.02)

Connection resistance for bending

$F_{t,Rd}$ =	323.14	[kN]	Bolt resistance for tension	[Table 3.4]
$B_{p,Rd}$ =	669.76	[kN]	Punching shear resistance of a bolt	[Table 3.4]

$F_{t,fc,Rd}$ – column flange resistance due to bending

$F_{t,wc,Rd}$ – column web resistance due to tension

$F_{t,ep,Rd}$ – resistance of the front plate due to bending

$F_{t,wb,Rd}$ – resistance of the web in tension

$F_{t,fc,Rd} = \text{Min}\ (F_{T,1,fc,Rd}\ ,\ F_{T,2,fc,Rd}\ ,\ F_{T,3,fc,Rd})$ [6.2.6.4] , [Tab.6.2]

$F_{t,wc,Rd} = w\ b_{eff,t,wc}\ t_{wc}\ f_{yc}\ /\ g_{M0}$ [6.2.6.3.(1)]

$F_{t,ep,Rd} = \text{Min}\ (F_{T,1,ep,Rd}\ ,\ F_{T,2,ep,Rd}\ ,\ F_{T,3,ep,Rd})$ [6.2.6.5] , [Tab.6.2]

$F_{t,wb,Rd} = b_{eff,t,wb}\ t_{wb}\ f_{yb}\ /\ g_{M0}$ [6.2.6.8.(1)]

RESISTANCE OF THE BOLT ROW NO. 1

$F_{t1,Rd,comp}$ - Formula	**$F_{t1,Rd,comp}$**	**Component**
$F_{t1,Rd} = \text{Min}\ (F_{t1,Rd,comp})$	507.83	Bolt row resistance
$F_{t,fc,Rd(1)} = 556.26$	556.26	Column flange - tension
$F_{t,wc,Rd(1)} = 507.83$	507.83	Column web - tension
$F_{t,ep,Rd(1)} = 646.27$	646.27	Front plate - tension
$B_{p,Rd} = 1339.52$	1339.52	Bolts due to shear punching
$V_{wp,Rd}/b = 1995.00$	1995.00	Web panel - shear
$F_{c,wc,Rd} = 2838.39$	2838.39	Column web - compression
$F_{c,fb,Rd} = 1857.09$	1857.09	Beam flange - compression

RESISTANCE OF THE BOLT ROW NO. 2

$F_{t2,Rd,comp}$ - Formula	**$F_{t2,Rd,comp}$**	**Component**
$F_{t2,Rd} = \text{Min}\ (F_{t2,Rd,comp})$	558.94	Bolt row resistance
$F_{t,fc,Rd(2)} = 637.36$	637.36	Column flange - tension

$F_{t2,Rd,comp}$ - Formula	$F_{t2,Rd,comp}$	Component
$F_{t,wc,Rd(2)} = 558.94$	558.94	Column web - tension
$F_{t,ep,Rd(2)} = 646.27$	646.27	Front plate - tension
$F_{t,wb,Rd(2)} = 843.51$	843.51	Beam web - tension
$B_{p,Rd} = 1339.52$	1339.52	Bolts due to shear punching
$V_{wp,Rd}/b - \sum_1^1 F_{ti,Rd} = 1995.00 - 507.83$	1487.17	Web panel - shear
$F_{c,wc,Rd} - \sum_1^1 F_{tj,Rd} = 2838.39 - 507.83$	2330.57	Column web - compression
$F_{c,fb,Rd} - \sum_1^1 F_{tj,Rd} = 1857.09 - 507.83$	1349.27	Beam flange - compression

RESISTANCE OF THE BOLT ROW NO. 3

$F_{t3,Rd,comp}$ - Formula	$F_{t3,Rd,comp}$	Component
$F_{t3,Rd} = \text{Min}(F_{t3,Rd,comp})$	507.32	Bolt row resistance
$F_{t,fc,Rd(3)} = 646.27$	646.27	Column flange - tension
$F_{t,wc,Rd(3)} = 558.94$	558.94	Column web - tension
$F_{t,ep,Rd(3)} = 646.27$	646.27	Front plate - tension
$F_{t,wb,Rd(3)} = 828.87$	828.87	Beam web - tension
$B_{p,Rd} = 1339.52$	1339.52	Bolts due to shear punching
$V_{wp,Rd}/b - \sum_1^2 F_{ti,Rd} = 1995.00 - 1066.77$	928.23	Web panel - shear
$F_{c,wc,Rd} - \sum_1^2 F_{tj,Rd} = 2838.39 - 1066.77$	1771.62	Column web - compression
$F_{c,fb,Rd} - \sum_1^2 F_{tj,Rd} = 1857.09 - 1066.77$	790.32	Beam flange - compression
$F_{t,fc,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 1187.94 - 558.94$	628.99	Column flange - tension - group
$F_{t,wc,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 1066.27 - 558.94$	507.32	Column web - tension - group
$F_{t,fc,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 1187.94 - 558.94$	628.99	Column flange - tension - group

$F_{t3,Rd,comp}$ - Formula	$F_{t3,Rd,comp}$	Component
$F_{t,wc,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 1066.27 - 558.94$	507.32	Column web - tension - group
$F_{t,ep,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 1193.58 - 558.94$	634.64	Front plate - tension - group
$F_{t,wb,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 1345.24 - 558.94$	786.30	Beam web - tension - group
$F_{t,ep,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 1193.58 - 558.94$	634.64	Front plate - tension - group
$F_{t,wb,Rd(3+2)} - \sum_2^2 F_{tj,Rd} = 1345.24 - 558.94$	786.30	Beam web - tension - group

RESISTANCE OF THE BOLT ROW NO. 4

$F_{t4,Rd,comp}$ - Formula	$F_{t4,Rd,comp}$	Component
$F_{t4,Rd} = \mathrm{Min}\ (F_{t4,Rd,comp})$	283.00	Bolt row resistance
$F_{t,fc,Rd(4)} = 637.36$	637.36	Column flange - tension
$F_{t,wc,Rd(4)} = 558.94$	558.94	Column web - tension
$F_{t,ep,Rd(4)} = 646.27$	646.27	Front plate - tension
$F_{t,wb,Rd(4)} = 828.87$	828.87	Beam web - tension
$B_{p,Rd} = 1339.52$	1339.52	Bolts due to shear punching
$V_{wp,Rd}/b - \sum_1^3 F_{ti,Rd} = 1995.00 - 1574.09$	420.91	Web panel - shear
$F_{c,wc,Rd} - \sum_1^3 F_{tj,Rd} = 2838.39 - 1574.09$	1264.30	Column web - compression
$F_{c,fb,Rd} - \sum_1^3 F_{tj,Rd} = 1857.09 - 1574.09$	283.00	Beam flange - compression
$F_{t,fc,Rd(4+3)} - \sum_3^3 F_{tj,Rd} = 1187.94 - 507.32$	680.62	Column flange - tension - group
$F_{t,wc,Rd(4+3)} - \sum_3^3 F_{tj,Rd} = 1066.27 - 507.32$	558.94	Column web - tension - group
$F_{t,fc,Rd(4+3+2)} - \sum_3^2 F_{tj,Rd} = 1778.76 - 1066.27$	712.49	Column flange - tension - group
$F_{t,wc,Rd(4+3+2)} - \sum_3^2 F_{tj,Rd} = 1378.93 - 1066.27$	312.67	Column web - tension - group

$F_{t4,Rd,comp}$ - Formula	$F_{t4,Rd,comp}$	Component
$F_{t,ep,Rd(4+3)} - \sum_3^3 F_{tj,Rd} = 1188.41 - 507.32$	681.09	Front plate - tension - group
$F_{t,wb,Rd(4+3)} - \sum_3^3 F_{tj,Rd} = 1330.60 - 507.32$	823.28	Beam web - tension - group
$F_{t,ep,Rd(4+3+2)} - \sum_3^2 F_{tj,Rd} = 1807.06 - 1066.27$	740.79	Front plate - tension - group
$F_{t,wb,Rd(4+3+2)} - \sum_3^2 F_{tj,Rd} = 2065.07 - 1066.27$	998.80	Beam web - tension - group

The remaining bolts are inactive (they do not carry loads) because resistance of one of the connection components has been used up or these bolts are positioned below the center of rotation.

SUMMARY TABLE OF FORCES

Nr	h_j	$F_{tj,Rd}$	$F_{t,fc,Rd}$	$F_{t,wc,Rd}$	$F_{t,ep,Rd}$	$F_{t,wb,Rd}$	$F_{t,Rd}$	$B_{p,Rd}$
1	576	507.83	556.26	507.83	646.27	-	646.27	1339.52
2	421	558.94	637.36	558.94	646.27	843.51	646.27	1339.52
3	266	507.32	646.27	558.94	646.27	828.87	646.27	1339.52
4	111	283.00	637.36	558.94	646.27	828.87	646.27	1339.52
5	-44	-	637.36	558.94	646.27	-	646.27	1339.52

CONNECTION RESISTANCE FOR BENDING $M_{j,Rd}$

$M_{j,Rd} = \sum h_j F_{tj,Rd}$

$M_{j,Rd} =$ 694.93 [kN*m] Connection resistance for bending [6.2]

$M_{b1,Ed} / M_{j,Rd} \leq 1,0$ 0.95 < 1.00 verified (0.95)

Connection resistance for shear

a_v =	0.60		Coefficient for calculation of $F_{v,Rd}$	[Table 3.4]
b_{Lf} =	0.97		Reduction factor for long connections	[3.8]
$F_{v,Rd}$ =	263.74	[kN]	Shear resistance of a single bolt	[Table 3.4]
$F_{t,Rd,max}$ =	323.14	[kN]	Tensile resistance of a single bolt	[Table 3.4]
$F_{b,Rd,int}$ =	592.20	[kN]	Bearing resistance of an intermediate bolt	[Table 3.4]
$F_{b,Rd,ext}$ =	329.00	[kN]	Bearing resistance of an outermost bolt	[Table 3.4]

Nr	$\mathbf{F_{tj,Rd,N}}$	$\mathbf{F_{tj,Ed,N}}$	$\mathbf{F_{tj,Rd,M}}$	$\mathbf{F_{tj,Ed,M}}$	$\mathbf{F_{tj,Ed}}$	$\mathbf{F_{vj,Rd}}$
1	646.27	11.14	507.83	483.46	494.60	239.13
2	646.27	11.14	558.94	532.12	543.27	210.76
3	646.27	11.14	507.32	482.98	494.12	239.41
4	646.27	11.14	283.00	269.42	280.57	363.92
5	646.27	11.14	0.00	0.00	11.14	520.99

$F_{tj,Rd,N}$ – Bolt row resistance for simple tension

$F_{tj,Ed,N}$ – Force due to axial force in a bolt row

$F_{tj,Rd,M}$ – Bolt row resistance for simple bending

$F_{tj,Ed,M}$ – Force due to moment in a bolt row

$F_{tj,Ed}$ – Maximum tensile force in a bolt row

$F_{vj,Rd}$ – Reduced bolt row resistance

$F_{tj,Ed,N} = N_{j,Ed}\, F_{tj,Rd,N} / N_{j,Rd}$

$F_{tj,Ed,M} = M_{j,Ed}\, F_{tj,Rd,M} / M_{j,Rd}$

$F_{tj,Ed} = F_{tj,Ed,N} + F_{tj,Ed,M}$

$F_{tj,Ed,N} = N_{j,Ed}\ F_{tj,Rd,N} / N_{j,Rd}$

$F_{vj,Rd} = \mathrm{Min}\ (n_h\ F_{v,Ed}\ (1 - F_{tj,Ed} / (1.4\ n_h\ F_{t,Rd,max}),\ n_h\ F_{v,Rd}\ ,\ n_h\ F_{b,Rd}))$

$V_{j,Rd} = n_h \sum_1^n F_{vj,Rd}$ [Table 3.4]

$V_{j,Rd}$ =	1574.21	[kN]	Connection resistance for shear	[Table 3.4]

$V_{b1,Ed} / V_{j,Rd} \leq 1{,}0$	0.17 < 1.00	verified	(0.17)

Weld resistance

A_w =	186.97	[cm^2]	Area of all welds	[4.5.3.2(2)]
A_{wy} =	86.55	[cm^2]	Area of horizontal welds	[4.5.3.2(2)]
A_{wz} =	100.42	[cm^2]	Area of vertical welds	[4.5.3.2(2)]
I_{wy} =	101456.77	[cm^4]	Moment of inertia of the weld arrangement with respect to the hor. axis	[4.5.3.2(5)]
$s_{^max}=t_{^max}$ =	165.79	[MPa]	Normal stress in a weld	[4.5.3.2(6)]
$s_{^}=t_{^}$ =	165.79	[MPa]	Stress in a vertical weld	[4.5.3.2(5)]
t_{II} =	26.43	[MPa]	Tangent stress	[4.5.3.2(5)]
b_w =	0.90		Correlation coefficient	[4.5.3.2(7)]

$Ö[s_{^max}^2 + 3*(t_{^max}^2)] \leq f_u/(b_w*g_{M2})$	331.59 < 417.78	verified	(0.79)
$Ö[s_{^}^2 + 3*(t_{^}^2+t_{II}^2)] \leq f_u/(b_w*g_{M2})$	334.73 < 417.78	verified	(0.80)
$s_{^} \leq 0.9*f_u/g_{M2}$	165.79 < 338.40	verified	(0.49)

Connection stiffness

t_{wash} =	6	[mm]	Washer thickness	[6.2.6.3.(2)]
h_{head} =	21	[mm]	Bolt head height	[6.2.6.3.(2)]
h_{nut} =	30	[mm]	Bolt nut height	[6.2.6.3.(2)]
L_b =	84	[mm]	Bolt length	[6.2.6.3.(2)]
k_{10} =	11	[mm]	Stiffness coefficient of bolts	[6.3.2.(1)]

STIFFNESSES OF BOLT ROWS

Nr	hj	k_3	k_4	k_5	$k_{eff,j}$	$k_{eff,j}\ h_j$	$k_{eff,j}\ h_j^2$
					Sum	17.06	557.49
1	576	0	0	65	0	0.00	0.00
2	421	3	102	59	2	9.02	380.13
3	266	3	102	49	2	5.66	150.81
4	111	3	102	58	2	2.38	26.55

$k_{eff,j} = 1 / (\sum_3^5 (1 / k_{i,j}))$ [6.3.3.1.(2)]

$z_{eq} = \sum_j k_{eff,j}\ h_j^2 / \sum_j k_{eff,j}\ h_j$

z_{eq} = 327 [mm] Equivalent force arm [6.3.3.1.(3)]

$k_{eq} = \sum_j k_{eff,j}\ h_j / z_{eq}$

k_{eq} = 5 [mm] Equivalent stiffness coefficient of a bolt arrangement [6.3.3.1.(1)]

k_1 = ¥ Stiffness coefficient of the column web panel subjected to shear [6.3.2.(1)]

k_2 = ¥ Stiffness coefficient of the compressed column web [6.3.2.(1)]

$S_{j,ini} = E\ z_{eq}^2 / \sum_i (1 / k_1 + 1 / k_2 + 1 / k_{eq})$ [6.3.1.(4)]

$S_{j,ini}$ = 117071.89 [kN*m] Initial rotational stiffness [6.3.1.(4)]

m = 2.62 Stiffness coefficient of a connection [6.3.1.(6)]

$S_j = S_{j,ini} / m$ [6.3.1.(4)]

S_j = 44737.03 [kN*m] Final rotational stiffness [6.3.1.(4)]

Connection classification due to stiffness.

$S_{j,rig}$ = 140944.65 [kN*m] Stiffness of a rigid connection [5.2.2.5]

$S_{j,pin}$ = 8809.04 [kN*m] Stiffness of a pinned connection [5.2.2.5]

$S_{j,pin} \leq S_{j,ini} < S_{j,rig}$ SEMI-RIGID

Weakest component:

BEAM FLANGE AND WEB - COMPRESSION

Connection conforms to the code Ratio 0.95