VILNIUS GEDIMINAS TECHNICAL UNIVERSITY

FACULTY OF CIVIL ENGINEERING

DEPARTMENT OF STEEL AND TIMBER STRUCTURES

Analysis of I-shape beam to CHS column connection with and without filled of concrete

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Contents

ABSTRACT

It is thought that the earliest practice of civil engineering may have commenced in ancient Egypt (4000 and 2000 BC) when the humans started to abandon a nomadic live to stay on the same place for live, that means that civil engineering is a science that has been studied more than 5000 years. Even so, nowadays there are many fields to investigate and study. Not everything is known.

This thesis was thought to study a topic that nowadays is not reflected on the regulations. This theme is the connection between steel beams and composite columns.

This thesis starts with theoretical concepts, the review and analysis of the composite structures and steel and composite joints. This first part was made with the basis of Eurocode and Design Guide. After this first theoretical part the practical part starts; this part is based on the comparison of two types of connections, two connections that are equal CHS column joint with an I-Beam, but one of the connection the CHS is filled of concrete. The analysis of the connections was done by hand, all the calculation than could made with Eurocode, and following with a finite elements study, with the support of the ANSYS program.

To sum up, the thesis was finished with the comparison of both types of joints to solve questions like: which are the benefits of infill concrete on behaviour of joint? How much load can support? ...

PREAMBLE

Keywords

Bucking; CHS (Circular Hollow Section); Concrete C25/30; Concrete-filled steel tubular; Fin plate; I-Beam; Simple shear connection; Steel S275.

Major symbols

a	Effective throat of weld	*I*	Inertia
Aa	Steel area	*Ia*	Steel inertia
Ac	Concrete area	*Ic*	Concrete inertia
Ant	Area subjected to tension	*Is*	Reinforcement inertia
Anv	Area subjected to shear	*Lcri*	Critical length of Euler
As	Reinforcement area	*M*	Bending moment
Av	Shear area of element	*My,Sd*	Bending moment action on y axis
d	Diameter	*My,Sd*	Bending moment resistance on y axis
d0	Hole diameter	*Mpl,y,Rd*	Plastic bending moment resistance on y axis
dc	Diameter column	*Mpl,Rd*	Plastic bending moment resistance
E	Elastic young modulus	*Mpl,z,Rd*	Plastic bending moment resistance on z axis
e	Eccentricity	*MRd*	Bending moment resistance
e1	End distance	*Mz,Sd*	Bending moment resistance on z axis
e2	Edge distance	*n*	Total number of bolts
Ea	Elastic young modulus steel	*Ncr*	Critical axis load of Euler
Ɛc	Concrete strain	*NG,Sd*	Permanent axis load
Ɛc1	Maximum concrete elastic strain schematic curve	*Npl,Rd*	Plastic axis resistance
Ɛc2	Maximum concrete elastic strain parabola-rectangle curve	*NRd*	Axis resistance
Ɛc3	Maximum concrete elastic strain bi-linear curve	*NSd*	Acting design normal force
Ecm	Elastic young modulus concrete	*p1*	Distance between holes on load direction
Ɛcu1	Maximum concrete plastic strain schematic curve	*p2*	Distance between holes perpendicular load direction
Ɛcu2	Maximum concrete plastic strain parabola-rectangle curve	*Pcri*	Critical load of Euler
Ɛcu3	Maximum concrete plastic strain bi-linear curve	*Rd*	Resistance
Es	Elastic young modulus refoircement	*Sd*	Action
Et	Elastic young modulus total	*t*	Thickness
F	Tying force or resistance	*tc*	Thickness column
Fb,Rd	Bearing resistance	*V*	Shear
fcd	Concrete strength with security factor	*β*	Correlation factor
fck	Concrete strength	*βw*	Correlation factor for fillet welds
fcm	Compression concrete strength	*ν*	Poisson
fy	Yield strength of steel	*ρs*	Longitudinal reinforcement ration
fyb	Bolt yield strength	*σeq*	Equivalent Von Mises stress
fyd	Yield strength with security factor	$\tau_{\perp}$	Shear stress perpendicular to axis
fsd	Refoircement yield strength with security factor	$\tau_{=}$	Shear stress parallel to the axis
fsk	Refoircement yield strength	*ϒa*	Steel safety factor
Ft,Rd	Tension resistance	*ϒc*	Concrete safety factor
fu	Ultimate tensile strength	*ϒM0*	Partial safety factor relative to plasticization of the material
fub	Ultimate tensile strength of the bolt	*ϒM1*	Partial security coefficient relating to instability phenomena
fur	Ultimate tensile strength of the rivet	*ϒM2*	Partial safety coefficient relative to the ultimate strength of the material or section, and the strength of the bonding means
Fv,Ed	Shear action	*ϒs*	Reinforcement safety factor
Fv,Rd	Shear resistance	*χ*	Reduction for the bucking curve

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1 INTRODUCTION

Research object

The case that is present on this thesis is a joint between a CHS (circular hollow section) (219,1 mm) and an I-beam (IPE A 330). The aim of the thesis is the reaction between the steel and the concrete on the column because of that, the beam is decided with the material S350, harder than the column or the fin plate.

Because of the complex on the column it was decided to choose a simple joint, the joint that was decided to study is a: simple shear connection, one example that you can see is on Figure 1: Simple shear joint.

The elements that have been decided for the practical analysis are:

Column:	Beam:	Fin plate:
Circular Hollow Section (CHS) Diameter 219,1 mm Thickness 8 mm Steel: S275	IPE A 330 Steel: S350	Thickness: 10 mm Steel: S275 Base: 150 mm Height: 228 mm

Table 1: Components of the specific case of the thesis

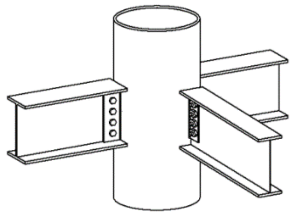


Figure 1: Simple shear joint.

The joint is more specific defined on the section: 3.1 Design of the joint.

Research of objectives

The aim of this thesis is investigate and study how reacts a composite column on a joint with an I-beam.

To find out the goal first we studied the norms that nowadays we can found about: steel, composite and joints. After that, we can study a steel joint and then compare with the same joint but with a composite column.

To determinate the resistance of the composite joint it was compared with the steel one. To determine how the composite joint reacts it was evaluated altering different parameters.

Tasks research

A key part of this thesis is the methodology, it was done by two methods: analytical and numerical. That means that there are some computer analysis and was compare with some hand calculation too.

Research methods

For the numerical analysis it was used a FEM program (Finite Elements Method program), the program was the ANSYS workbench. To do the simulation of the joint was used a static structural analysis.

For analytical calculation it was used the Eurocode. Principal Eurocode used was:

Eurocode 2	Design of concrete structures (EN 1992)
Eurocode 3	Design of steel structures (EN 1993)
Eurocode 4	Design of composite steel and concrete structures (EN 1994)

2. THEORETICAL BASIS OF COMPOSITE COLUMNS

Composite member is defined by (Eurocode 4 (EC4 1996) as: "a structural member with components of concrete and of structural or cold-formed steel, interconnected by shear connection so as to limit the longitudinal slip between concrete and steel and the separation of one component from the other."*[1]*

To start knowing about composite columns we need to do some reference to the history of that ones, we must know why they been created, how, when…

- Evolution of composite columns*[2]*

We can divide the evolution of composites columns in four historical periods:

1st period: Initial research. The first contact and started research in composite columns was on the beginning of the 20s century. The combining of steel and concrete has been started with the motivation of the protection in front of the fire.

2nd period: first climax. Fritz von Emperger presented more than 1500 tests about composite structures on the IABSE congress in Paris (1932). It was him who complained the lack of design rules for composite columns in Europe and signalize the American "Standard Specifications for Concrete and Reinforce Concrete" of 1924 where it is found specific formulas and concepts about the topic. Emperger was not the first pioneer in concrete construction, but is consider one of the most influential scientific on this area.

3rd period: a period of oblivion

4th period: A revival of research and application. On 1950s the composite constructions started again looking for a better characteristic that cannot be found in steel or concrete individually. In 1957 was publish in German by Klöppel and Goder three tests where was examined in detail, with stresses and strains, the concrete-filled steel tubes and in both, stress and strain, the steel and concrete had an incremental values of concentric load. Although they defined the formula for the design of CFTs, the equation was based on the separation of the tangent modulus of steel and concrete.

In 1970s was developed the Eurocode 4 by Roik, Bergmann, Bode and Wagenknecht. The aim of this code is the simplified design method for composite columns.

- Advantages and disadvantages *[3]*

Composite structures collect all advantages of steel and concrete structures and even create new positive properties, even if, the design is a correct design. It is necessary good design of elements and of connectors.

Steel	Concrete
Effective in tension	Effective in compression
More plasticity	Prevent from buckling
Light material	Protect from corrosion
	More protection in front of fire

Table 2: Advantages of steel and concrete

The most representative advantages and disadvantages about composite columns are:

Advantages	Disadvantages
High bearing capacity and full utilization of concrete and steel strength	Unknown prediction of the evolution
High fire resistance	Difficult solutions for connections with beams
Plastic behaviour at limit state	Difficulties in case of later strengthening of the column
Better resistance to corrosion, chemical attack, and outdoor weathering.	Case edge protection is necessary
Economical solution with regard to material cost	

Table 3: Advantages and disadvantages of composite column

Steel structures are used for big spans and concrete is economic and protects better in front of fire or extreme temperatures. For big spans, it is used to use composite floors. Those composite floors are the union of concrete and steel by shear connectors and composite slabs on special steel decks. Composite structures made for steel and concrete are used to: slabs, beams and columns. In this theoretical part of the thesis is focus on composite columns, concrete and steel columns.

- Examples

It is important to realize where and how is using this types of columns on the actuality. There are millions of examples that can be analyses; in this case, it is going to talk about few of them.

Peckham Library (London, England): Designed by Alsop and Störmer and winner of the Stirling Priza for Architecture. This design includes composite columns, CHS columns filled by concrete, with 18 meters long. Seven supporting columns are angled out of the vertical. This inclination of the columns provides additional stiffness against lateral loads and have more resistance in front of the bending moment. *[4]*

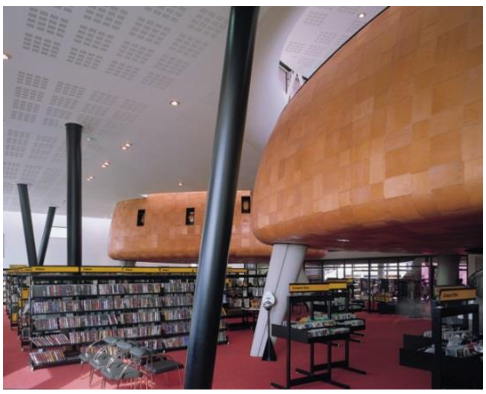


Figure 2: Peckham Library

Bank of China tower (Hong Kong): I. M. Pei & Partners, Sherman Kung & Associates Architects Ltd., Thomas Boada S.L. are the architectural companies that have done one of the most recognizable skyscrapers in Hong Kong. Made by triangular truss in composite steel and reinforce concrete. Its innovative composite structural system of steel and concrete resists high-velocity winds and involved significant savings in constructions time and materials.

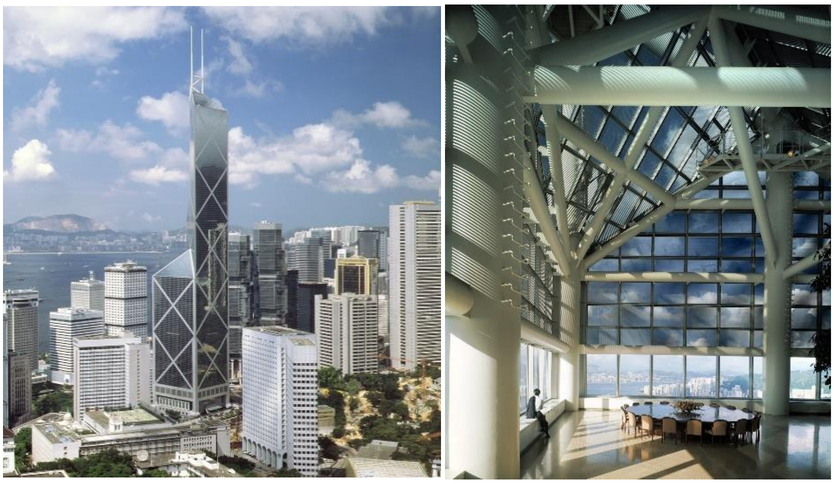


Figure 3: Bank of China tower (Hong Kong)

- Limitations

On the using of composite columns exists some limitations, the fact that there are two different materials makes the study much more complex. It is important to have in mind that steel and concrete have different stress-deformation curves and clear different properties, that is an advantage when you can control it, but is a clear disadvantage to anticipate futures behaviours because is not possible to define global properties like: inertia, yield stress and youth modulus.

The type of failature depends in a huge way on the measures of the structures. A composite structure can have a different performance if the diameter, length, thickness of steel tube and resistance of concrete and steel, and even the type of load applied are changed.

Nowadays are being studied facts like local buckling, temperature effect, load-sharing and even the connections between column and beam.

Another limitation that does not help on the study of the composite members is that is complex to extrapolate values found on the laboratory and studies. That fact is studied in country like Japan (uMorino et al., 1996).

2.1 Behaviour peculiarities of composite columns

The reason why composite structures are used can be expressed in one simple sentence: concrete is good in compression and steel is good in tension. Joining the materials in a correct way you can obtain a high efficient and light weigh design.

Most of books and papers classify composite structures in 3 types:

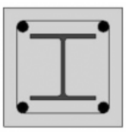
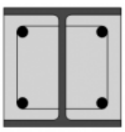
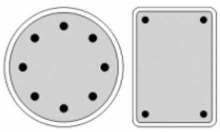
Concrete encased	Partially concrete encased	Concrete filled hollow
		

Table 4: Types of composite column [5]

Exists several design methods to calculate the resistance of a composite column, all with the same aim, look for the highest load that the column can stand.

Eurocode 4 explains the design of composite steel and concrete structures and CIDECT have done a research adopted by this Eurocode, (CIDECT design guide number 5), on that document can be found a detailed information for the static design of concrete filled columns.

Eurocode define the calculation for the ultimate limit state. The column must verify that have more resistance and stability on worst combination of actions. The global stability must be confirming.

The resistance capacity must include imperfections, the deformation influence on the balance (second order theory) and the loss of stiffness if it plasticizes. Because of that, the material properties are relevant. For the concrete is considered a parable-rectangular graph and for steel is correct to considerer a bilinear one.

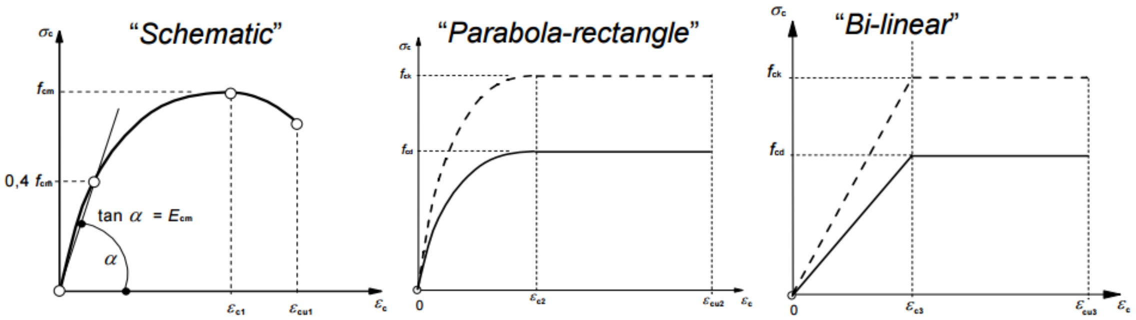


Figure 4: Different representations of stress-strain graph. Eurocode 2.[6]

If it is necessary an exact calculation it must be used a FEM program, but it is not always useful, because of that these types of analysis are considered as complemental simulations, but cannot be the bases of the study.

To know which is the highest load that a composite column can support the Eurocode 4 define the aim equation:

$$Sd \leq Rd \qquad \textit{eq. 1}$$

Where S_d is the actions combination and the R_d the resistance that the column has. This resistance depends on security coefficients for the loads Υ_F, and for each material too Υ_M. For composite elements that is the complex part, how this fact effects on the resistance of the column, which proportion, how many load supports the steel and how many the concrete, how is the stress distribution…

$$Rd = R(\frac{fy}{\Upsilon a};\frac{fck}{\Upsilon c};\frac{fsk}{\Upsilon s}) \qquad \textit{eq. 2}$$

Eurocode 4 use an addition security coefficient with the aim to covert equilibrium failures. For composite structures it must be only modify the steel security coefficient Υ_{Ma}. For buckling failure steel resistance must be divided for Υ_{Rd}, or Υ_a, depending on the case. The stability failure could be not studied if the column is compact, in other words, relative slenderness is not higher than 0,2, or axial load is not higher than $0{,}1\cdot N_{cr}$.

All the coefficients that Eurocode indicate are just recommendations and could be change depending on the national documents. *[6]*

Structural Steel	Concrete	Reinforcement
Υ_a = 1,1	Υ_c = 1,5	Υ_S= 1,15

Table 5: Safety factors

Actions security coefficients must be chosen (Υ_F) by Eurocode 1 or national codes. If any of the coefficients are modify must be included on the technical conditions of the structure.

Material properties: *[6]*

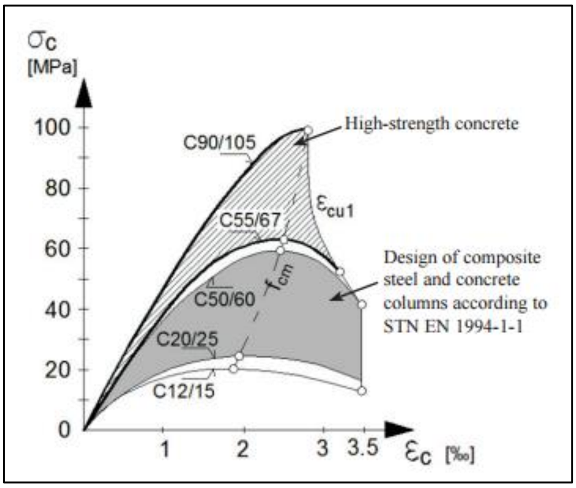


Figure 5: Stress-strain of different types of concrete. Eurocode 2.

On Eurocode 2 we can found the concrete that can be used for a composite column and on Eurocode 3 the different steels.

Different types of concrete $f_{ck,cyl}/f_{ck,cub}$	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60
Resistance f_{ck} [N/mm^2]	20	25	30	35	40	45	50
Elastic modulus E_{cm} [N/mm^2]	29000	30500	32000	33500	35000	36000	37000

Table 6: Resistance and elastic modulus for different concretes

Concrete resistance must be reduced on 0,85 for long term structures. For composite structures is not necessary reduce that value because the steel protect it, furthermore, concrete will increase its resistance. That fact is explain with detail on the next point 2.1.1 Axial compression on the effects section.

On Eurocode 2 it is include types steel and some properties of that ones:

Types of steel	S235	S275	S355	S460
Yield stress	235	275	355	460
Elastic modulus	210000			

Table 7: Yield stress and elastic modulus of different steels

On EN 1994-1-1 there are two methods to verify the resistance of the member for structural stability in a composite column:

- *General method:* can be apply any type of cross-section and any combination of materials
- *Simplified method:* Not all sections can be determinate with this method. The ones that can use the simplified method are sections that have double-symmetric cross-section, uniform cross-section over the member length, limited steel contribution factor δ, related slenderness smaller than 2, limited reinforcing ratio, limitation of b/t-values.

On simplified method is found two evaluations:

- Axial compression
- Combined compression and bending

The verification of axial compression and combined, compression and bending, is based on European buckling curves and on second order analysis with equivalent geometrical bow imperfections.

On the next sections is studied how to interpret and calculate the methods to obtain the highs load that a composite column can support.

2.1.1 Axial compression

As it is said before the general expression that must be verify is:

$$Sd \leq Rd \qquad \textit{eq. 3}$$

On a axial compression case we can defined that general expression in:

$$NSd \leq NRd \qquad \textit{eq. 4}$$

In addition, the most relevant part now is how to develop N_{Rd}, the axial resistance that the column can afford. On this case:

$$NRd = \mathcal{X} \cdot Npl, Rd \qquad \textit{eq. 5}$$

That means that N_{Rd} will be a fraction of the axial plasticity resistance.

$$NSd \leq \mathcal{X} \cdot Npl, Rd$$

As it is known, N_{Sd} is the design normal force, including of course load factors. The new factor that appears is the $\mathcal{X}$, that factor is a reduction for the buckling curve, in this case curve "a".

To calculate $N_{pl,Rd}$, resistance of the section, we have to consider that now there are two materials and is not as easy as it used to be. To consider more exact axial resistance the weighted average of the strengths of materials must be done.

$$Npl, Rd = Aa \cdot fyd + Ac \cdot fcd + As \cdot fsd \qquad \textit{eq. 6}$$

These areas are the transversal areas of the section. The sub index "a" means steel, "c" concrete and "s" reinforcement. With their design strengths f_{xd}.

As you can see the strengths has the sub index "d" too, so that means that must be divided about the factors ϒ that you can found in *Table 5: Safety factors* that are based in Eurocode 1.

Npl, Rd because of the confinement effect, Eurocode 4-1-1 indicates that $N_{pl,Rd}$ can be higher. That effect is explained on eq. 16.

The factor$\mathcal{X}$, as it is said before, is a reduction factor that it is found in Eurocode curves, this factor is determined for the relative slenderness λ' and can be determinate with

$$\bar{\lambda} = \sqrt{\frac{Npl, Rd}{Ncr}} \qquad \textit{eq. 7}$$

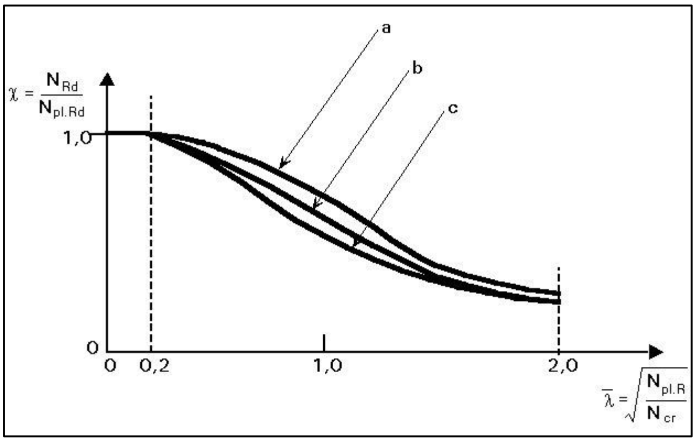
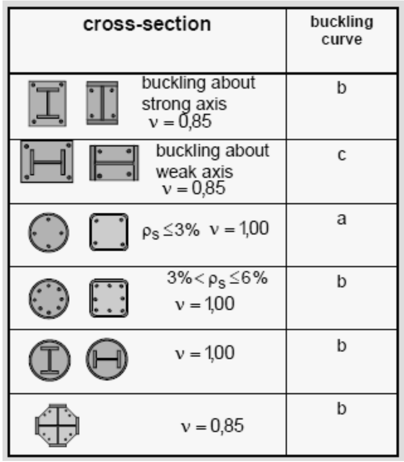


Figure 6: Bucking curves of Eurocode



cross-section	buckling curve
buckling about strong axis $\nu = 0,85$	b
buckling about weak axis $\nu = 0,85$	c
$\rho_S \leq 3\%$ $\nu = 1,00$	a
$3\% < \rho_S \leq 6\%$ $\nu = 1,00$	b
$\nu = 1,00$	b
$\nu = 0,85$	b

Figure 7: Classification of the different sections on the bucking curves

On Figure 7: Classification of the different sections on the bucking curves*,* appear the buckling curve that must be use depending on the ρs and υ. Those factors are the longitudinal reinforcement radio (ρs) and the type of composite column (υ):

$$0{,}3\% \leq \rho s \leq 6{,}0\% \qquad \rho s = \frac{As}{Ac} \tag{eq. 8}$$

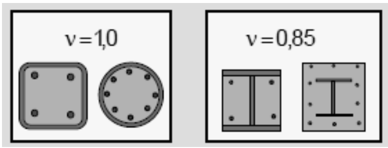


Figure 8: Value of υ for different types of composite column.

Npl,Rd is defined on *eq. 6* and N_{cr} is the Euler critical load, is the elastic buckling load of the member, in a theoretical calculation:

$$Ncr = \frac{(EI) \cdot \pi^2}{l^2} \tag{eq. 9}$$

Now the question is how to calculate EI. If there are two materials, steel and concrete, the most exact way to do it is with the equation:

$$EI = Ea \cdot Ia + 0{,}6 \cdot Ecm \cdot Ic + Es \cdot Is \tag{eq. 10}$$

That equation is obtaining on Eurocode 3, for columns in non-sway systems, as a safe approximation, the column length may be taken as the buckling length.

$0{,}6 * Ecm * Ic$ is the effective stiffness of the concrete section with E_{cm} being the modulus of elasticity of concrete.

- LIMITS

There are few limits like the area of the reinforcement and area of the section, ρ = 4%. Another proportion is δ:

$$\delta = \frac{Aa \cdot fyd}{Npl, Rd} \qquad \text{eq. 11}$$

Where; $\gamma_{Ma} = \gamma_a$. and must be: $0{,}2 \leq \delta \leq 0{,}9$.

That value is very relevant for the study of the composite structure. If the value is lower than 0,2 the column is going to be consider concrete and it is calculate based on Eurocode 2; otherwise, if the value is higher than 0,9 the column is considering steel and it is calculate following Eurocode 3.

For bending and compression load exists another limit to avoid local buckling:

- For RHS: being "h" the grater overall dimension of the section:

$$\frac{h}{t} \leq 52 \cdot \varepsilon \qquad \text{eq. 12}$$

- For CHS:

$$\frac{d}{t} \leq 90 \cdot \varepsilon^2 \qquad \text{eq. 13}$$

The factor ε accounts for different yield limits.

$$\varepsilon = \sqrt{\frac{235}{fy}} \qquad \text{eq. 14}$$

- EFFECTS
  - Long-term effect

On these section we saw that on composite columns the concrete does not use the long term factor (0,85), but the influence of the long-term is considered by a modification of the modulus Ec. The load bearing capacity of the columns may be reduced because of the creep and shrinkage. For loading that are permanently on the column the modulus of the concrete is half of the original value. For loads that are only partly permanent, an interpolation can be carried out:

$$Ec = 0{,}6 \cdot Ecm \cdot (1 - 0{,}5 \cdot \frac{NG, Sd}{NSd}) \qquad \text{eq. 15}$$

N_{Sd} is the acting design normal force

$N_{G,Sd}$ is the permanent part of it

That reduction of E_c do not have to be used always, for short columns or high eccentricities of loads, creep and shrinkage do not have to be considered. Furthermore, the influence of creep and shrinkage has to be only considered only for slender columns.

- Confinement effect

Another effect that must be in consider is the effect of confinement. On columns, the transversal deformation is blocked; because of that, they have more resistant in front of axial loads.

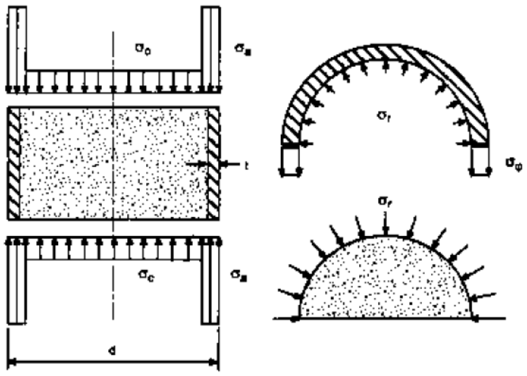


Figure 9: Confinement effect. Distribution of stress

That effect can be considering on the equation increasing the concrete resistance and decreasing the steel one, the steel is going to have less traction resistance.

$$Npl,Rd = \eta a \cdot fyd \cdot Aa + Ac \cdot fcd \cdot \left(1 + \eta c \cdot \frac{t}{d} \cdot \frac{fyk}{fck}\right) \qquad \text{eq. 16}$$

You can consider that plastic resistance if it can be considering that the column is stocky and the loads are on the center:

$$\eta ao = 0{,}25$$

$$\eta co = 4{,}9$$

If you cannot consider that values, you can calculate the influence of slenderness and load eccentricity with:

- Influence of slenderness for $\bar{\lambda} \leq 0{,}5$

$$\eta a,\lambda = \eta ao + 0{,}5 \cdot \bar{\lambda}\text{k} \leq 1{,}0$$
$$\eta c,\lambda = \eta co - 18{,}5 \cdot \bar{\lambda}\text{k}(1 - 0{,}92 \cdot \bar{\lambda}\text{k} \geq 0 \qquad \text{eq. 17}$$

- Influence of load eccentricity:

$$\eta a = \eta a,\lambda + 10(1 - \eta ao) \cdot \frac{e}{d}$$
$$\eta c = \eta c,\lambda \cdot (1 - 10 \cdot \frac{e}{d}) \qquad \text{eq. 18}$$
$$\frac{e}{d} \geq 0{,}1 \;\; ; \;\; \eta a = 1 \; and \; \eta c = 0$$

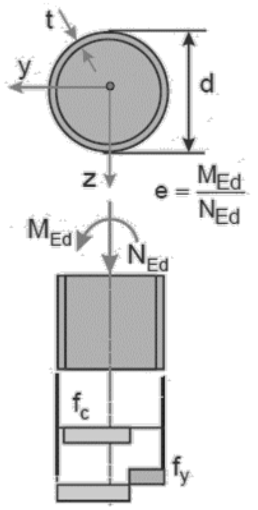


Figure 10: Representation of a compression with eccentricity on a CHS filled of concrete

2.1.2 Compression and bending

The interaction between compression and bending could be represented by an interaction curve where is represented the axis N_{Rd} and the flexural moment M_{Rd} Figure 11: Interaction curve. Different relevant points and cases of each one. Evolution of the curve with different parameters of δ.. This curve is calculating in an exhaustive way. The curve is obtaining studding the section with different positions of the neutral axis.

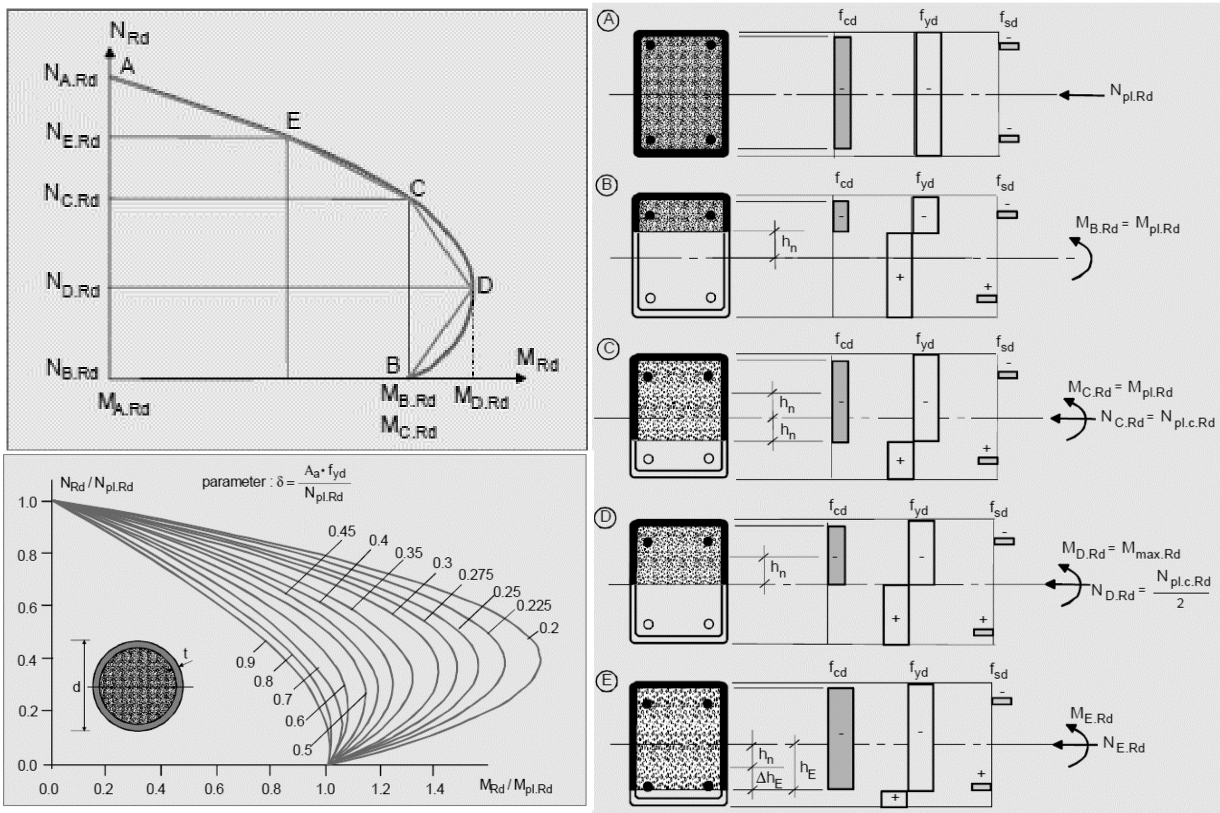


Figure 11: Interaction curve. Different relevant points and cases of each one. Evolution of the curve with different parameters of δ. [5]

The interaction curves depend on the type of section and on the type of load that are working on it, because of that, it is need to calculate the cross section parameter δ. The curves are evaluate without the refoircement, but that fact must be considered on the calculation of δ because is function of $N_{Pl,Rd}$.

$$\delta = \frac{\mathrm{Aa} \cdot \mathrm{fyd}}{\mathrm{Npl,Rd}} \qquad \textit{eq. 19}$$

Knowing how are the loads on the section, it can be known the neutral axis position and the distribution of stress. Depending if in the section there are: only axial, only moment or different combinations of axial and moment you can identify the position on the interaction curve.

For the calculation of the resistance of a member to bending and compression we can found two cases:

- Uniaxial bending and compression
- Biaxial bending and compression

2.1.2.1 Uniaxial bending and compression

On this same thesis it is show that for the calculation on the simple compression on composite structures appears a new confident χ. Now the column is going to resist less capacity of axial compression because with the same resistance the loads increase.

The curve that defines the uniaxial bending and compression it depends on different factors:

χ: the same one that it was show in 2.1.1

χ_d: resulting from the actual design normal force N_{sd}

χ_n: considers the influence of the imperfection on different bending moment distributions. This imperfection is only need to be considered for high normal forces (χ_n>0). Except on constant moments or lateral loads within the columns length or for sway frames, the imperfection has to consider χ_n=0. For end moments can be determinate by:

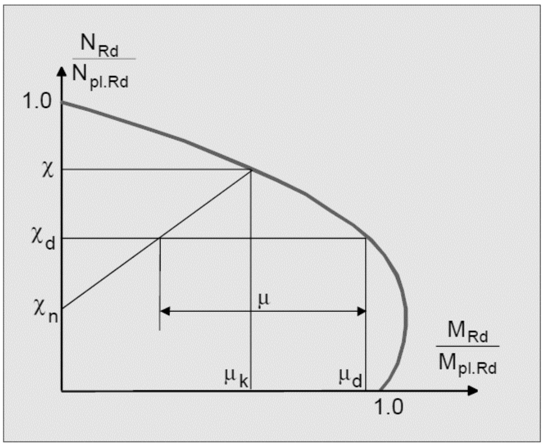


Figure 12: Interaction curve [5]

$$\chi n = \chi \frac{1 - r}{4} \qquad \text{eq. 20}$$

r: radios of the larger to the smaller end moment (-1≤r≤1)

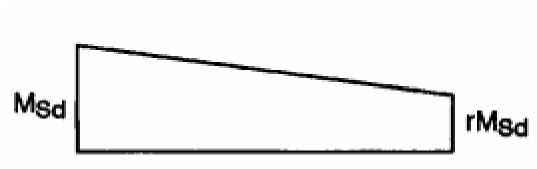


Figure 13: Definition of r. Example of distribution of flexural moment.[5]

μ_d: it is defined as the imperfection moment

μ:

$$\mu = \mu d - \mu k \cdot \frac{\chi d - \chi n}{\chi - \chi n} \qquad \text{eq. 21}$$

With uniaxial bending and compression, the calculation is guide by the expression:

$$Msd \leq 0.9 \cdot \mu \cdot Mpl, Rd \qquad \text{eq. 22}$$

M_{sd}: design bending moment of the column

0.9 is the factor given from the simplified design of the column and is a reduction because of:

- The interaction curve does not have in consideration the strain limitations.

- And because of the effective stiffness, the concrete cracking it is cover with this factor.

- Effects:

When the section is in compression could happens that the section is over pressed and that causes a higher resistance to bending. Because of that appears one effect that increase the value of bending capacity, higher than $M_{pl,Rd}$. This effect has just to take into account if it is ensured that bending moment and axial force are going to be acting at the same time and they are not going to act individually. If you cannot have ensured, you must take the value of μ a maximum value 1.

2.1.2.2 Biaxial bending and compression

When the section of the column it is affected by moments in different direction you cannot calculate the value of the maximum moment with the equations that we have seen.

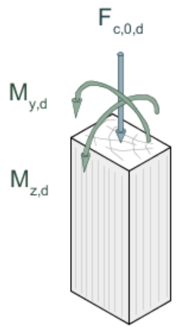


Figure 15: Biaxial bending and compression.

The section is going to need more resistance to tolerate the new actions. The interaction curve of the section and the moment factor have included also the additional moments. The imperfection needs to be taken into account and that reduce the resistance of the composite column.

Now there is not just one interactive curves, there are two, one for each bending and then the interaction of both have to be evaluated.

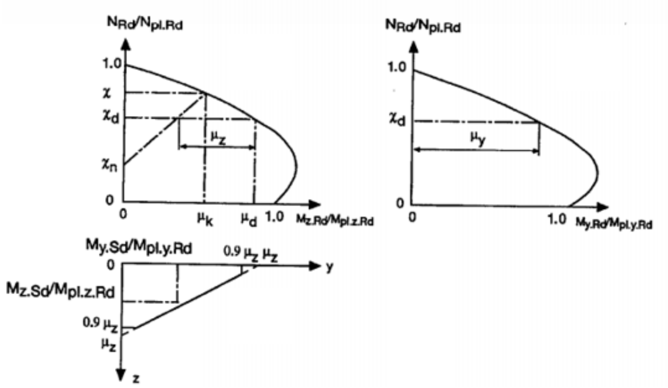


Figure 14: Representation of the interactive curve of a biaxial moment and the interaction between them. [5]

In that case the expression that have to be verify is:

$$\frac{My, Sd}{\mu y * Mpl, y, Rd} + \frac{Mz, Sd}{\mu z * Mpl, z, Rd} \leq 1 \qquad \textit{eq. 23}$$

In addition, it is limited:

$$\frac{My, Sd}{\mu y * Mpl, y, Rd} \leq 0{,}9 \qquad \textit{eq. 24}$$

$$\frac{Mz, Sd}{\mu z * Mpl, z, Rd} \leq 0{,}9$$

2.1.2.3 Simplified determination of bending moments

It is accepted to use a short calculation to know how to evaluate the bending moments.

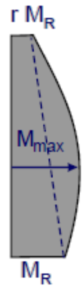


Figure 16: example of a bending distribution

$$k = \frac{Mmax}{MR} = \frac{\beta}{1 - \frac{NRd}{Ncr}} \qquad \text{eq. 25}$$

β=0,66+0,44r

β≥0,44

2.2 Component method for design of steel joint connections

The other main topic of this thesis is the joints. After composite columns research and knowing how to calculate the maximum load, now is the moment to study the different types of joints.

On this thesis we are on a particulate case of a CHS (circular hollow section) filled of concrete joint to a I-beam, so, if we want to study the joint we can study steels joints because there is no concrete on the beam or external of the column.

The design of steel structures can be found on Eurocode 3: Design of steel structures – Part 1-8: Design of joints (EN 1993-1-8). On that part of the Eurocode you can found the design methods for the design of joints subject to predominantly static loading using steel grades S235, S275, S355, S420, S450 and S460.

Joints can be classifying with different criteria:

- Rotational stiffness of the union:
  - Simple/Flexible: can allowed a rotation. Because of that the shear is transmitted but not the bending. Joints must be capable of transmitting internal forces without developing significant moments.
  - Rigid: joints have rotational stiffness and because of that can transmit the moment and the deformation can be considering negligible. Must be capable to transmit shear and bending.
  - Semi-rigid: the ones that cannot be consider rigid or flexible.

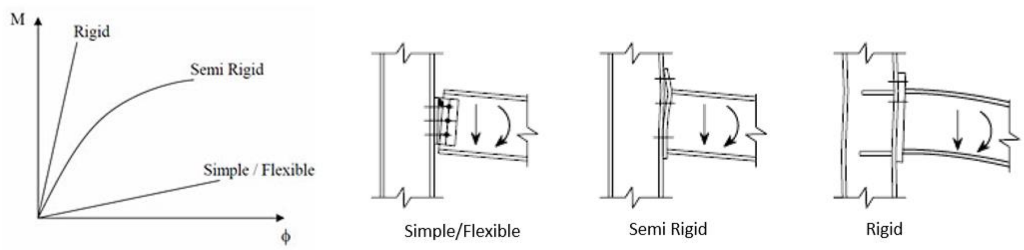


Figure 17: types of joints. Rotational stiffness of the union

- Elements on the joint:
  - Welded connections
  - Connections made with bolts, rivets or pins
  - Hollow sections joints
- Based on function:
  - Beam-to-Beam
  - Beam-to-Column
  - Column-to-Column

- Column Base Plates
- Pocket Beam
- Gusset plate (truss type, frame type, bracings, …)
- Splices (cover plates, …)

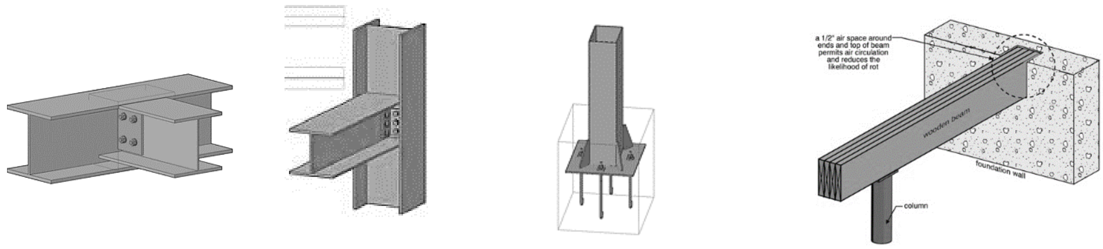


Figure 18: Different types of unions: Beam to beam, Beam to column, Column to base plate and pocket beam.

Depending on the type of union, the structure will have different properties of stiffness, resistance and stability.

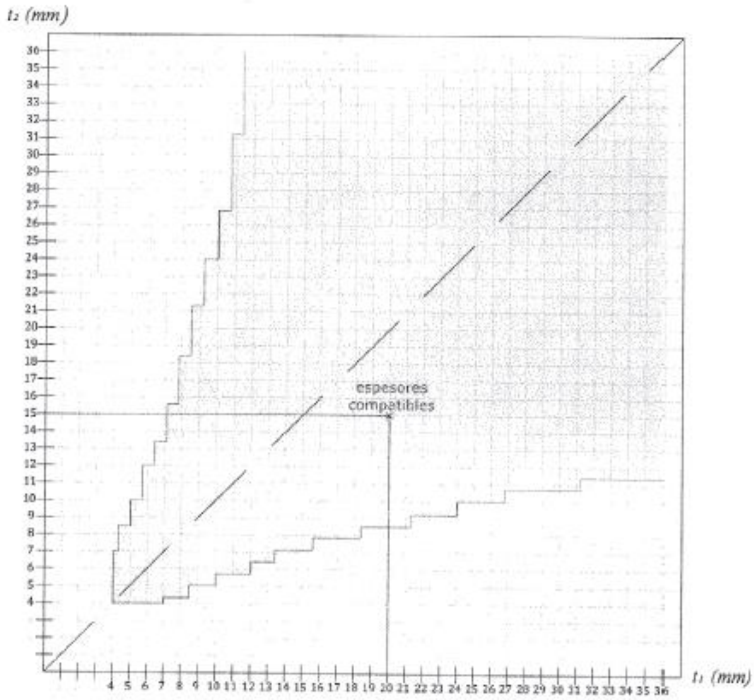
Joints requirements are resistance, stiffness and deformation capability.

2.2.1 Welded joints

The welded joint is the most economical one, but difficult to realize on the construction. Most of the joints that can be done on factory are welded.

Eurocode defines the directional method. That method consider that the efficiency plane of the weld is situated at 45°, and have the thickness is represented by the letter 'a'.

It is necessary define a thickness of the weld and there are limitations depending on the bulkiness of the elements that wants to joint. Depending on the country the minimum thickness could change. For example, in Spain the minimum is defined by NBE-EA-95:



Espesor pieza (mm)	Garganta a (mm) Máx.	Mín.
4,0-4,2	2,5	2,5
4,3-4,9	3,0	2,5
5,0-5,6	3,5	2,5
5,7-6,3	4,0	2,5
6,4-7,0	4,5	2,5
7,1-7,7	5,0	3,0
7,8-8,4	5,5	3,0
8,5-9,1	6,0	3,5
9,2-9,9	6,5	3,5
10,0-10,6	7,0	4,0
10,7-11,3	7,5	4,0
11,4-12,0	8,0	4,0
12,1-12,7	8,5	4,5
12,8-13,4	9,0	4,5
13,5-14,1	9,5	5,0
14,2-15,5	10,0	5,0
15,6-16,9	11,0	5,5
17,0-18,3	12,0	5,5
18,4-19,7	13,0	6,0
19,8-21,2	14,0	6,0
21,3-22,6	15,0	6,5
22,7-24,0	16,0	6,5
24,1-25,4	17,0	7,0
25,5-26,8	18,0	7,0
26,9-28,2	19,0	7,5
28,3-31,1	20,0	7,5
31,2-33,9	22,0	8,0
34,0-36,0	24,0	8,0

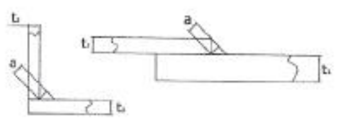


Figure 19: Different possible options of plates union because of the thickness [8]

In addition, the maximum, defined on the same regulation:

$$amax \leq 0{,}7 * tmin \qquad \text{eq. 26}$$

The directional method defines resistance of the filled weld applying von misses:

$$\sigma eq \leq \frac{fu}{\beta w * \gamma Mw} \qquad \text{eq. 27}$$

$$\sigma eq = \sqrt{\sigma_{\perp}^2 + 3 \cdot (\tau_{\perp}^2 + \tau_{\parallel}^2)} \qquad \text{eq. 28}$$

Because of that, the equation that must be verify is:

$$\sqrt{\sigma_{\perp}^2 + 3 \cdot (\tau_{\perp}^2 + \tau_{\parallel}^2)} \leq \frac{fu}{\beta w * \gamma Mw} \qquad \text{eq. 29}$$

In this method, forces transmitted by a unit length of weld are resolved into components parallel and transverse to the longitudinal axis of the weld and normal and transverse to the plan of its throat.

$\sigma_{\perp}$: Normal stress perpendicular to the throat.

$\sigma_{\parallel}$: Normal stress parallel to the axis of the weld.

$\tau_{\perp}$: Shear stress (in the plane of the throat) perpendicular to the axis of the weld.

$\tau_{\parallel}$: Shear stress (in the plane of the throat) parallel to the axis of the weld.

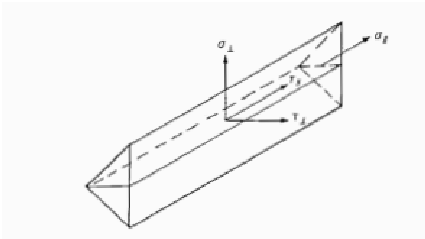


Figure 20: Representation of the weld

The normal stress $\sigma_{\parallel}$ could be not take into account when the design resistance is verified.

We can see that appears a new constant, the correlation factor βw, that elements give the property of the material. Depending on the yield stress of the elements that the weld is joining.

Standard and steel grade			Correlation factor β_w
EN 10025	**EN 10210**	**EN 10219**	
S 235 S 235 W	S 235 H	S 235 H	0,8
S 275 S 275 N/NL S 275 M/ML	S 275 H S 275 NH/NLH	S 275 H S 275 NH/NLH S 275 MH/MLH	0,85
S 355 S 355 N/NL S 355 M/ML S 355 W	S 355 H S 355 NH/NLH	S 355 H S 355 NH/NLH S 355 MH/MLH	0,9
S 420 N/NL S 420 M/ML		S 420 MH/MLH	1,0
S 460 N/NL S 460 M/ML S 460 Q/QL/QL1	S 460 NH/NLH	S 460 NH/NLH S 460 MH/MLH	1,0

Table 8: Correlation factor for fillet βw welds [8]

For large welds there are some limitations because of the stress distribution. On the beginning and ending there are more resistant to the stress and less on the middle. Because of that there is a reduction factor (βlw) that must be multiply by weld resistance if the length of the weld is longer than 150·a.

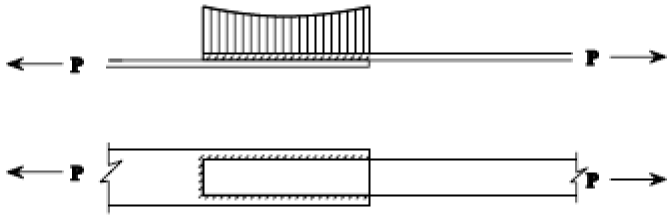


Figure 21: distribution of the stress on a weld

$$\beta lw = 1{,}2 - \frac{0{,}2Lj}{150a} \leq 1 \qquad \text{eq. 30}$$

Lj: weld length on the force direction.

2.2.2 Bolted joints

Bolds are distributed by classes depending on the material:

Bolt class	4.6	4.8	5.6	5.8	6.8	8.8	10.9
f_{yb} (N/mm^2)	240	320	300	400	480	640	900
f_{ub} (N/mm^2)	400	400	500	500	600	800	1000

Table 9: Resistance of the different classes of bolts [8]

The position of the holes for the bolts and rivets are limited to avoid easy failures, on Eurocode are defined on a summary table:

Distances and spacings, see Figure 3.1	Minimum	Maximum[1) 2) 3)]		
		Structures made from steels conforming to EN 10025 except steels conforming to EN 10025-5		Structures made from steels conforming to EN 10025-5
		Steel exposed to the weather or other corrosive influences	Steel not exposed to the weather or other corrosive influences	Steel used unprotected
End distance e_1	$1{,}2d_0$	$4t + 40$ mm		The larger of $8t$ or 125 mm
Edge distance e_2	$1{,}2d_0$	$4t + 40$ mm		The larger of $8t$ or 125 mm
Distance e_3 in slotted holes	$1{,}5d_0$ [4)]			
Distance e_4 in slotted holes	$1{,}5d_0$ [4)]			
Spacing p_1	$2{,}2d_0$	The smaller of $14t$ or 200 mm	The smaller of $14t$ or 200 mm	The smaller of $14t_{min}$ or 175 mm
Spacing $p_{1,0}$		The smaller of $14t$ or 200 mm		
Spacing $p_{1,i}$		The smaller of $28t$ or 400 mm		
Spacing p_2 [5)]	$2{,}4d_0$	The smaller of $14t$ or 200 mm	The smaller of $14t$ or 200 mm	The smaller of $14t_{min}$ or 175 mm

1) Maximum values for spacings, edge and end distances are unlimited, except in the following cases:

- for compression members in order to avoid local buckling and to prevent corrosion in [AC2) exposed members (the limiting values are given in the table) and; (AC2]
- for exposed tension members [AC2) to prevent corrosion (the limiting values are given in the table). (AC2]

2) The local buckling resistance of the plate in compression between the fasteners should be calculated according to EN 1993-1-1 using $0{,}6\ p_i$ as buckling length. Local buckling between the fasteners need not to be checked if p_1/t is smaller than $9\ \varepsilon$. The edge distance should not exceed the local buckling requirements for an outstand element in the compression members, see EN 1993-1-1. The end distance is not affected by this requirement.

3) t is the thickness of the thinner outer connected part.

4) The dimensional limits for slotted holes are given in 1.2.7 Reference Standards: Group 7.

5) For staggered rows of fasteners a minimum line spacing of $p_2 = 1{,}2d_0$ may be used, provided that the minimum distance, L, between any two fasteners is greater or equal than $2{,}4d_0$, see Figure 3.1b).

Figure 22: Minimum and maximum spacing, end and edge distance [8].

The nomenclatures of the distances are:

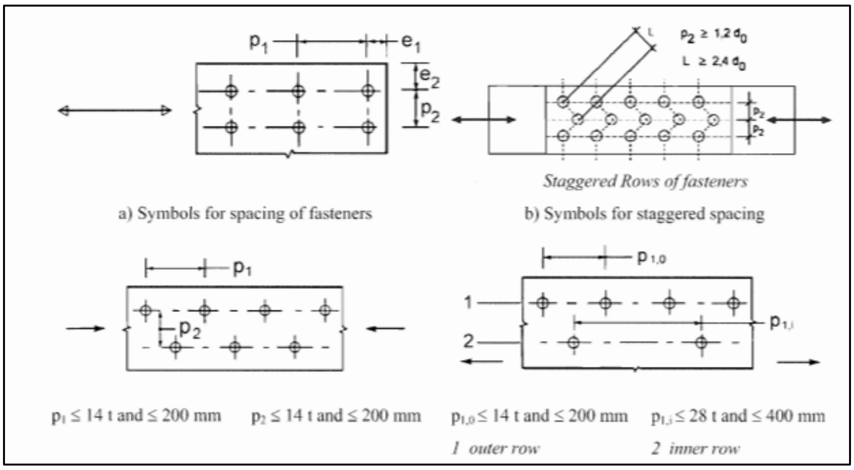


Figure 23: Representation of the distances [8]

There exist some minimum and maximum values for these distances:

a) Minimums:
  i) On the direction of the load:
   - $e_1 \geq 1{,}2d_0$ from center of the hole to edge of the piece
   - $p_1 \geq 2{,}2d_0$ between centers of holes
  ii) perpendicular to the load direction:
   - $e_2 \geq 1{,}5d_0$ from center of the hole to edge of the piece
   - $p_2 \geq 3d_0$ between centers of holes
b) maximum:
  i) to the edge of the piece:
   - For e_1 and e_2 $\begin{cases} \leq 40mm+4t \\ \leq 12t \text{ or } 150 \text{ mm} \end{cases}$
  ii) Between bolts:
   - On compression elements $p \leq 14t$ and $p \leq 200$ mm; being 't' the lowest thickness of the pieces that the connections are joint.
   - On tension:
     Exterior bolts $p_e \leq 14t$ and $p_e \leq 200$ mm;
     Interior bolts $p_i \leq 28t$ and $p_i \leq 400$ mm

There are categories because of the shear or tension connection:

- Category A: Bearing type, on this class have to use bolt classes from 4.6 to 10.9. This bolds are not preloading and special provisions for contact surface are required.
- Category B: Slip-resistant at serviceability limit state
- Category C: Slip-resistant at ultimate limit state
- Category D: non-preloaded
- Category E: preloaded

Depending on the categories, the criteria of verifications are different. The summary of that ones can be found on the Eurocode:

Category	Criteria	Remarks
	Shear connections	
A bearing type	$F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	No preloading required. Bolt classes from 4.6 to 10.9 may be used.
B slip-resistant at serviceability	$F_{v,Ed.ser} \leq F_{s,Rd,ser}$ $F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	Preloaded 8.8 or 10.9 bolts should be used. For slip resistance at serviceability see 3.9.
C slip-resistant at ultimate	$F_{v,Ed} \leq F_{s,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$ AC2 $\sum F_{v,Ed} \leq N_{net,Rd}$ AC2	Preloaded 8.8 or 10.9 bolts should be used. For slip resistance at ultimate see 3.9. $N_{net,Rd}$ see 3.4.1(1) c).
	Tension connections	
D non-preloaded	$F_{t,Ed} \leq F_{t,Rd}$ $F_{t,Ed} \leq B_{p,Rd}$	No preloading required. Bolt classes from 4.6 to 10.9 may be used. $B_{p,Rd}$ see Table 3.4.
E preloaded	$F_{t,Ed} \leq F_{t,Rd}$ $F_{t,Ed} \leq B_{p,Rd}$	Preloaded 8.8 or 10.9 bolts should be used. $B_{p,Rd}$ see Table 3.4.
The design tensile force $F_{t,Ed}$ should include any force due to prying action, see 3.11. Bolts subjected to both shear force and tensile force should also satisfy the criteria given in Table 3.4.		

NOTE: If preload is not explicitly used in the design calculations for slip resistances but is required for execution purposes or as a quality measure (e.g. for durability) then the level of preload can be specified in the National Annex.

Figure 24: Verifications depending on the joint category [8]

- $F_{v,Rd}$: shear resistance per shear plane

$$Fv, Rd = \frac{\alpha v \cdot fu \cdot As}{\gamma M2} \ (bolts)$$ eq. 31

$$Fv, Rd = \frac{0{,}6 \cdot fur \cdot A0}{\gamma M2} \ (rivets)$$

  - Where the shear plane passes through the threaded portion of the bolt (A is the tensile stress area of the bolt As):
    - For classes 4.6,5.6 and 8.8: αv=0,6
    - For classes 4.8,5.8,6.8 and 10.9: αv=0,5
  - Where the shear plane passes through the unthreaded portion of the bolt (A is the gross cross section of the bolt): αv=0.6.

- $F_{b,Rd}$: Bearing resistance 1) 2) 3)

$$Fb, u = \frac{k1 \cdot \propto b \cdot fu \cdot d \cdot t}{\gamma M2}$$ eq. 32

Where αb is the smallest of αd, $\frac{fub}{fu}$ or 1,0:

In the direction of load transfer:

  - For end bolts: $\alpha d = \frac{e1}{3d0}$; for inner bolts: $\alpha d = \frac{p1}{3d0} - \frac{1}{4}$

Perpendicular to the direction of load transfer:

  - For edge bolts: $k1$ is the smallest of $2{,}8\frac{e2}{d0} - 1{,}7$, $1{,}4\frac{p2}{d0} - 1{,}7$ and $2{,}5$
  - For inner bolts: $k1$ is the smalles of $1{,}4\frac{p2}{d0} - 1{,}7$ or 2,5

- Tension resistance 2)

$$Ft, Rd = \frac{k2 \cdot fub \cdot As}{\gamma M2} \ (bolts)$$ eq. 33

$$Ft, Rd = \frac{0{,}6 \cdot fur \cdot A0}{\gamma M2} \ (rivets)$$

Where k_2=0,63 for countersunk bolt,

Otherwise k_2=0,9

- Punching shear resistance

$$Bp, Rd = \frac{0{,}6 \cdot \pi \cdot dm \cdot tp}{\gamma M2} \ (bolts)$$ eq. 34

  Rivets are not necessary to be check.

- Combined shear and tension

$$\frac{Fv, Ed}{Fv, Rd} + \frac{Ft, Ed}{1{,}4\, Ft, Rd} \leq 1{,}0$$ eq. 35

[1)] The bearing resistance $F_{b,Rd}$ for bolts

- In oversized holes is 0,8 times the bearing resistance for bolts in normal holes
- In slotted holes, where the longitudinal axis of the slotted hole is perpendicular to the direction of the force transfer, is 0,6 times the bearing resistance for bolts in round, normal holes.

[2)] For countersunk bolt:

- The bearing resistance $F_{b,Rd}$ should be based on a plate thickness t equal to the thickness of the connected plate minus half the depth of the countersinking
- For the determination of the tension resistance $F_{t,Rd}$ the angle and depth of countersinking should conform with 1.2.4 Reference Standards: Group 4, otherwise the tension resistance $F_{t,Rd}$ should be adjusted accordingly.

[3)] When the load on a bolt is not parallel to the edge, the bearing resistance may be verified separately for the bolt load components parallel and normal to the end.

Another factor needs to take into account: Design for block tearing. That concept consist on the rupture because of the stress and the subjection of the bolts.

To calculate the resistance of the block tearing it is need to calculate the area that would be break and determine the resistance to the tension and shear of this area.

Could be two cases:

- For a symmetric bolt group to concentric loading:

$$Veff,1,Rd = \frac{fuv \cdot Ant}{\gamma M2} + \frac{\frac{fy}{\sqrt{3}} \cdot Anv}{\gamma M0} \qquad \textit{eq. 36}$$

A_{nt} is net area subjected to tensions; without the holes

A_{nv} is the net area subject to shear; without the holes too.

- For a bolt group subject to eccentric loading the design block shear tearing resistance Veff,2,Rd is given by:

$$Veff,2,Rd = 0,5\frac{fu \cdot Ant}{\gamma M2} + \frac{\frac{fy}{\sqrt{3}} \cdot Anv}{\gamma M0} \qquad \textit{eq. 37}$$

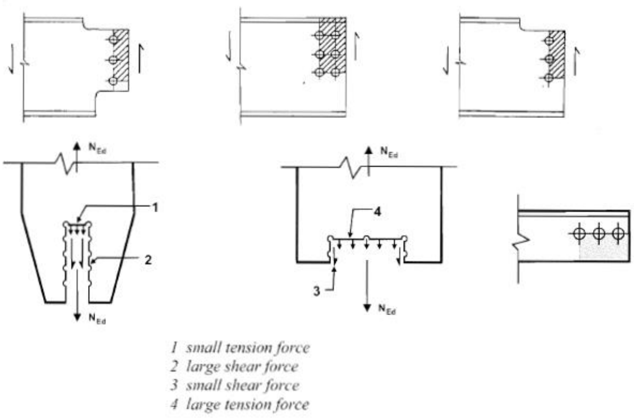


Figure 25: Some examples of the block tearing.

2.2.3 Hollow section joints

The hollow sections works a little bit different on the joints. These types of joints are close section that their general verification must be:

$$\frac{Ni,Ed}{Ni,Rd}+\frac{Mip,Ed}{Mip,Rd}+\frac{Mop,Ed}{Mop,Rd}\leq 1 \qquad \text{eq. 38}$$

Of course, that means that the resistance of the joint must be higher than the actions in it.

If the joint is a CHS member, formula can be substituted by:

$$\frac{Ni,Ed}{Ni,Rd}+\left(\frac{Mip,i,Ed}{Mip,i,Rd}\right)^2+\frac{|Mop,i,Ed|}{Mop,i,Rd}\leq 1{,}0 \qquad \text{eq. 39}$$

On Eurocode 3, EN 1993-1-8 define six types of failures that could happen in a hollow section joint.

- Chord face failure (yielding)
- Chord side wall failure (yielding and/or instability)
- Chord shear failure (yielding and/or instability)
- Chord punching shear
- Brace failure
- Local buckling failure (brace or chord)

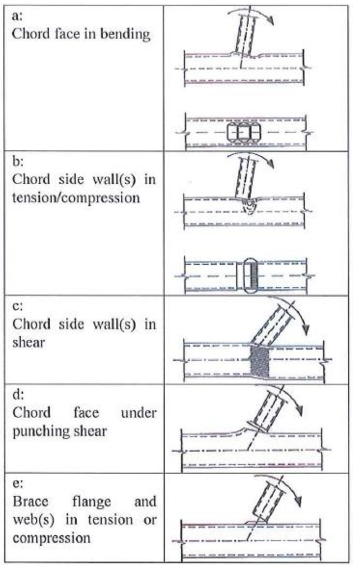


Figure 26: Types of failures. From Design Guide 9: For structural hollow section column connection.

Because of that, if it is necessary to verify the joint these six possible failures must be verified.

The calculation and definition about hollow section connections is extended. Because of that in that item this thesis have been focused on the example case: circular hollow section joint to a I-beam. As we defined at first time on *2.2 Component method for design of steel joint connections* exists different types of joints depending on the rotational stiffness of the union, this rotation define which and how the stress is distributed by the joint.

On circular hollow column connections we can defined X types of connections:

- Simple shear connection (flexible or non-rigid connection)

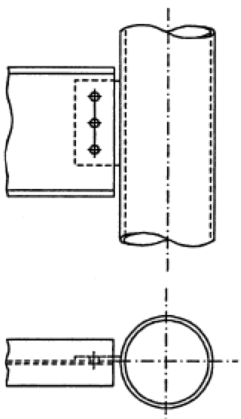


Figure 27: Simple shear connection between CHS and I-beam

This type of connections allowed the beam to rotate, because of that the bending moment is not transferred to the column, just the shear. The shear creates a flexural moment on the column that must be studied too.

Of course to analyses this type of connections the welded and the bold must be verify as it is explained on sections *2.2.1 Welded joints and 2.2.2 Bolted joints* but now the number of verification increases, the column must be analysed too.

New verifications for hollow section columns:

- Shear yield strength of the tube wall adjacent to a weld
- Punching shear through the tube wall
- Plasticization of the tube wall, using a yield line mechanism

Now the column have to internal loads: shear and the bending moment that creates the shear because of the eccentricity (form the point that it is considered the rotation to the centre of the column). There exist some restriction of the dimensions:

$$\frac{dc}{tc} \leq 0{,}114 \frac{E}{fc, y} \qquad \text{eq. 40}$$

That limitation makes sure the punching effect will not appears. It is need to do more verifications on the wall column, for example the column wall resistance. On the Eurocode, we can find the expression on the section of hollow holes, welded joints:

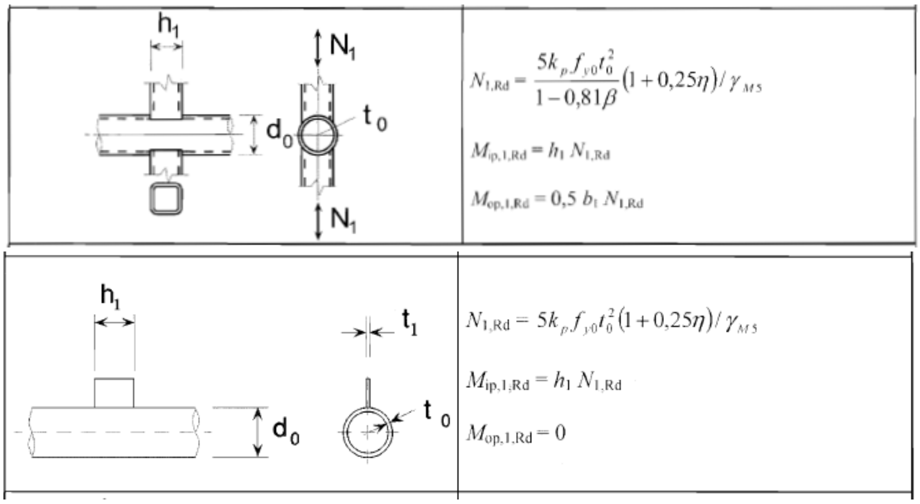


Figure 28: Example of the calculation of column wall resistance. Eurocode 3.

Therefore, we can define the verifications on this type of joints:

- Shear capacity of beam
- Shear plate thickness
- Bolts required
- Bearing resistance for beam web and shear plate
- Plate length
- Shear yield strength of tube wall adjacent to welds
- Net section fractures of shear plate
- Net section fractures of beam web
- Grass section yielding of shear plate
- Fillet welds.

Another option is design the connection with a through plate. The plate does not ended on the column wall; the plate is cross all the column section that means that the same plate can help to the column to resist the load that are distributed because of the beam. The disadvantage that could appears is that a torsional moment needs to be verified too.

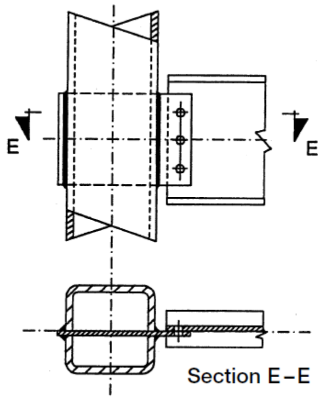


Figure 29: Through-plate connection

- Semi-rigid connection

Most of these semi-rigid connections are on the case that the beam is welded completely to the column. To calculate this type of connection the Design Guide 9 define different tables with verification and calculation of maximum flexural moment that the column can resist.

The limitation on these cases are: chord plasticization and punching shear. As we know, the general equation that must be verify is:

$$\frac{Ni,Ed}{Ni,Rd}+\frac{Mip,Ed}{Mip,Rd}+\frac{Mop,Ed}{Mop,Rd}\leq 1 \quad \textit{eq. 38}$$

$$\frac{Ni,Ed}{Ni,Rd}+\left(\frac{Mip,i,Ed}{Mip,i,Rd}\right)^2+\frac{|Mop,i,Ed|}{Mop,i,Rd}\leq 1{,}0 \quad \textit{eq. 39}$$

For CHS section.

On design guide not only it is defined the maximum load, it is defined limitations because of dimensions too.

One example of these tables that can be found on Eurocode or Design guide is:

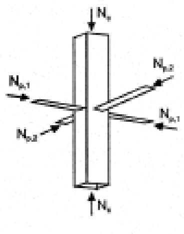
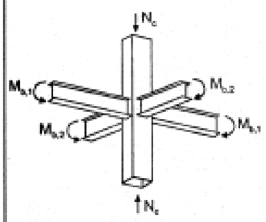
Type of connection	Factored connection resistance
 $J = N_{b,2}/N_{b,1} \le 1.0$	$N_p^* = (0.5 + 0.7\beta) \cdot \frac{4}{\sqrt{1-0.9\beta}} \cdot f_{c,y} \cdot t_c^2 \cdot f(n) \cdot f(J)$ $N_p^* = f_{p,y} \cdot t_p \cdot b_e$
 $J = M_{b,2}/M_{b,1} \le 1.0$	$M_b^* = (0.5 + 0.7\beta) \cdot \frac{4}{\sqrt{1-0.9\beta}} \cdot f_{c,y} \cdot t_c^2 \cdot (h_b - t_{b,f}) \cdot f(n) \cdot f(J)$ $M_b^* \le f_{b,y} \cdot t_{b,f} \cdot b_e \cdot (h_b - t_b)$
General	
Punching shear Chord side wall crippling	Do not govern within validity range given
Functions	
$n = \frac{N_c}{A_c . f_{c,y}} + \frac{M_c}{W_{c,el} \cdot f_{c,y}}$ $J = \frac{N_{p,2}}{N_{p,1}}$ or $\frac{M_{b,2}}{M_{b,1}}$	$f(n) = 1 + 1.48n(2\gamma)^{-0.33} - 0.46n^{1.5}(2\gamma)^{(-.33-0.11\beta^2)} \le 1.0$ $f(J) = 1 + 0.4 \cdot J$ (lower bound) ≤ 1.0 $b_e = \frac{10}{b_c / t_c} \cdot \frac{f_{c,y} \cdot t_c}{f_{b,y} \cdot t_{b,f}} \cdot b_{b,f}$ but $\le b_{b,f}$
Range of validity	
$0.2 \le \beta \le 0.8$, $1.5 \le 2\gamma \le 37.5$, $0.3 \le \eta \le 2.0$	

Figure 30: Semi-rigid connection between a RHS and I-beam[8]

on the case of CHS to I-beam connection on the semi-rigid connection we can found that on Design Guide 9 defines that most common use of that joint are stiffened with plates and that can be calculated with rigid connection equations.

- Rigid (full strength) connection:

A rigid joint is not recommended on seismic zones because the limitation of movement on the beam could cause a faster failure. On a rigid connection there is no energy dissipated on the joint. Although nowadays most of the connections beam to column are full strength connections, or have to be consider full strength to be secure. The only exception that can be classify as non-rigid is the simple shear joints done by a fin plate, that types of joint could be considered shear connection.

Most of the rigid connections must have an element that transfer the loads from the beam to the column, not just welding the beam, these elements can be called stiffeners and one of the most common stiffeners are the diaphragm, that stiffeners transfer the axial loads on the beam to the column. The diaphragms can be designed as an internal or external.

Some examples are:

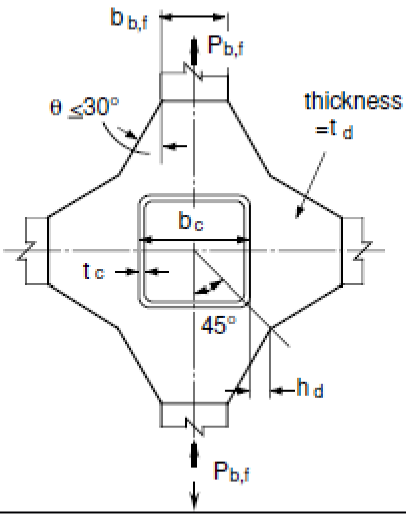
Shape of external diaphragm	Ultimate resistance equation
	$$P_{b,f}^* = 3.17\left(\frac{t_c}{b_c}\right)^{2/3}\left(\frac{t_d}{b_c}\right)^{2/3}\left(\frac{t_c+h_d}{b_c}\right)^{1/3} b_c^2 f_{d,u} \quad (2)$$ where $\frac{b_c/2+h_d}{t_d} \leq \frac{240}{\sqrt{f_{d,y}}}$ Symbols: $f_{d,y}$ = Yield strength of diaphragm material; $f_{d,u}$ = Ultimate tensile strength of diaphragm material; $P_{b,f}$ = Axil load in tension or compression flange
Range of validity	
$17 \leq \frac{b_c}{t_c} \leq 67$ $0.07 \leq \frac{h_d}{b_c} \leq 0.4$ $0.75 \leq \frac{t_d}{t_c} \leq 2.0$ $\theta \leq 30°$	

Figure 31: Example of external diaphragm calculation and limitation of dimensions [8]

We can see that as the other calculations of different types of joint the aim is search the maximum load. The Eurocode do not have just the equation to calculated, these documents includes limitations of dimensions that we must have in mind.

2.3 Component method for design of composite joint connections

The aim of this section is to define the different, on a theory bases, between the steel joints and the concrete and steel joints.

As we know exists 3 types of composites joints: concrete encased, partially concrete encased and concrete filled hollow. Table 4: Types of composite column:

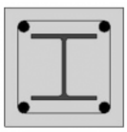
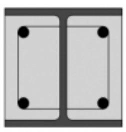
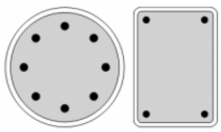
Concrete encased	Partially concrete encased	Concrete filled hollow
		

Table 10: Types of composite column

Because the topic is extend on this section of the thesis we will focus on the concrete filled hollow steel column, that are the specific case that we are interested. The mixture of a two different elements that have different type of behaviour in front of the load is difficult to preview or calculate the futures events that can cause.

On this section, we will see how this topic is controlled on the Eurocodes, Design Guide and other articles that have been published.

The Eurocode 4 have defined how to calculate and work with composite elements. On section 8 of Eurocode 4 we found a section that is defined as: Section 8 Composite joint in frames for buildings, where we can found how to calculate some types of connections but more focused on composite slabs than in composite columns. One conclusion is that there are not exhaustive rules to define how a CHS filled of concrete fails, furthermore, we can found some definitions of maximum load that Eurocode recommends, but they do not specify how will fail the joint: because of the steel, because of the concrete compression….

Where we can research about composite joints on Design Guide 9: For structural hollow section column connections. There is a section: section 9 Connections to concrete filled columns, this is exactly the case that it is studied in this thesis. So, on this point, it will be analyse the section of the design guide 9 on the column filled of concrete.

As in the hollow connections, the Design Guide structures divide the composite joint depending on the rotation capacity of that ones:

- Simple shear connections
- Semi-rigid connections
- Rigid (full strength) connections

We can found more types of connections like: unreiforcement welded hollow section beam and column connections, bolted hollow section beam and column connections…

- **Simple-shear connection**

With a simple shear connection the column receipt, as we know, just the shearing; but the column feels a flexural moment that is created by the same shear too.

The research now is to try to know how many load is distributed to the steel and how many to the concrete. On Design Guide it is indicate that full composite action of the cross-section could be assumed. On the same section, Design Guide recommends to us to reduce the concrete strength capacity. That means that in not involve penetration of the hollow section joint, the concrete will have less strength capacity than in front of simple shear connection to RHS that reduction factor is defined:

$$\propto c,2 = 1 - 1{,}2 \cdot \xi \cdot [\frac{\propto c,1 \cdot Ac,c \cdot fc}{Ac \cdot fc,y + \propto c,1 \cdot Ac,c \cdot fc}]] \qquad \text{eq. 41}$$

$\propto c,1$ is 0,85 and ξ is the ratio of the load applied at the shear connection. *[10]*

After including this decrease of resistance on the concrete the calculations can be the same ones that in a steel simple joint. Another indication is that the column deformation is not possible on a concrete filling section, because of that, there is no need to consider. That affirmation is very relevant because there is any expression that defines what happens with the column wall.

After this clarification of a factor that involucrate the concrete resistance, the Design Guide clarify that if you want to calculate a simple shear connection is recommended to follow the indications and the same criteria than a joint with no concrete. Otherwise, the same Design Guide indicates that now the column wall resistance not makes sense to calculate because the new column, filled of concrete, will increase the resistance *[10]*. However, they do not give a hypothesis, criteria or expression to calculate the new wall resistance.

In conclusion, on this part of the calculation we see that the calculations on a composite column are based on the calculations of a simple steel joint.

- **Semi-rigid connections**

One of the most important factors that change from the same joint with a column with concrete inside is the limitation of the rotation. Now the column is more rigid because of that the strength and stiffness increase but the rotation decrease. Because of that, some connections considered semi-rigid with a circular hollow sections behave full-rigid with concrete inside the column.

To sum up, a semi-rigid connection will have more resistance filled by concrete but will be more sensible to bending moments. Only an elastic design approach is allowed to do more conservative calculations of the load.

If we want to focus on the joint: I-beam to CHS column connection, on design guide, we can found an example with fin plates and a "CORONA" studied by Winkel (1998). For the cases where the rotation is relevant the recommendation is based on have a stronger connection than the beam connected, that means, the connection have to be controlled by the beam. Design guide recommends to do less stronger the beam create the fail point on the beam and not on the joint. If the beam is less resistant than the joint is easier to control and to verify where and when the connection will fail.

One expression that must be verify based on the investigation of the Design Guide between I-beam and CHS filled concrete column used for compression loading the yield load of the flange provided that:

$$fb,y \cdot tb \leq fc,y \cdot tc \qquad \textit{eq. 42}$$

To sum up, on semi-rigid joints the connection could be full rigid with the column filled of concrete. For the calculation of the joint, the recommendation is do a stronger joint than the beam and control the collapse of the beam.

With these recommendations, as on the simple shear connections, the rules do not define where and the maximum load the connection will fail.

- **Rigid (full strength) connection**

The rigid connection is the one that we have more information, on those studies the AIJ standard for steel reinforced concrete structures and Eurocode 4 defines different ways to define the higher load.

AIJ adopts the superposition method that means, study the steel and the concrete on a separate way and the minimum load will defined the future fail of the joint. Some studies defined that on the superposition method the maximum load is less than the ones on the reality, but this method must be complemented with experimental evaluations. The issue of this method is that the concrete is not sufficiently ductile and that can cause an error on the unsafe.

With the superposition method it was found some expressions:

- For shear strength of column web panel: that expression is defined on the base of the design strength on the ultimate limit state, it was done by the yield strength with a 1,2 factor.

$$Vc,w* = 1{,}2(Ac,p \cdot \beta \cdot \frac{fc}{10} + Ac,w \cdot \frac{fc,y}{\sqrt{3}}) \qquad \textit{eq. 43}$$

β depends on the section of the column. If the section is:

CHS	RHS
$\beta = 2\frac{hc,w - 2tc,w}{hb - 2td} \leq 4{,}0$ eq. 44	$\beta = 2{,}5\frac{hc,w - 2tc,w}{hb - 2td} \leq 4{,}0$ eq. 45

Table 11: Value of β for CHS and RHS sections

- Flexural strength of beam-to-column connection: for rigid connections with external diagrams we can use the expression

$$Mj,cf *= Mb,f,u + Mb,w,u \quad \text{eq. 46}$$

$$Mb,w,u = m \cdot Wpl,b,w,n \cdot fb,y \quad \text{eq. 47}$$

$$Mb,f,u = Pb,f \cdot (hb - tb,f) \quad \text{eq. 48}$$

The first two expressions (eq. 47 and eq. 48) are the same ones that we can use on full rigid connections without the columns filled of concrete, is the third expression (eq. 48) that is adequate to the case with concrete.
However, most of the full connections without concrete could be used for the calculations of these joints; you must know could be errors on the safe side.

We can see that in other types of connections the expression given to study the joint with the columns filled of concrete are based on the steel joints and most of them without defining an individual expression.

3 PRACTICAL APPLICATION

To apply all the aspects that was studied on the theoretical part and to evaluate and compare how the composite column works, it was decided to choose one type of joint and analyze the same joint with a normal steel tubular column and with the same steel tubular column but filled with concrete.

On this part of the thesis is going to study these two types of joint in two different ways:

- Analytical: using Eurocodes and Design Guide
- Finite element method (FEM): the joints are going to be simulated on a FEM program that can be obtain values that are more accurate.

3.1 Design of the joint

The joint that was decided to study was a simple bolted connection. It is composed with a plate that is weld to the column and bolts to the beam. Because of that is the bolts and the plates that are the connectors from one component to the other.

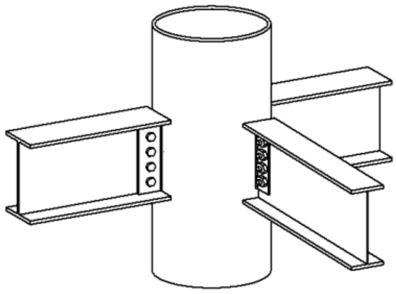


Figure 32: Representation of a simple shear joint of an I-beam to a CHS column

This type of joint is called shear connection, is assumed that can rotate unrestrained. However, simple shear connections of course have some restrictions and cannot rotate free, but it is neglected.

The column receive only the shear from the beam, and is the same shear load that because of its eccentricities creates a flexural moment on the wall column that must be studied too. That means that the bolts acts as articulation and because of that the distribution of the load is something similar to:

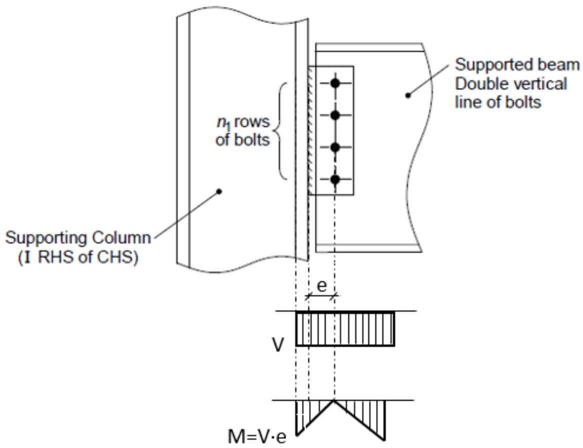


Figure 33: Representation of the internal loads on a simple shear connection

The elements that have been decided for the practical analysis are:

Column:	Beam:	Fin plate:	Bolts:	Weld
Circular Hollow Section (CHS) Diameter 219,1 mm Thickness 8 mm Steel: S275	IPE A 330 Steel: S350	Thickness: 10 mm Steel: S275 Base: 150 mm Height: 228 mm	M20 8.8 3 rows of bolts	e: 6 mm

Table 12: Components of the specific case of the thesis

- COLUMN:

The column is a circular hollow section (CHS) made with steel S275.

Diameter of the column is 219,1 mm and the thickness 8 mm. On the next table, you can see the dimensions and properties:

Outside Diameter D (mm)	Thickness t (mm)	Mass per meter (kg/m)	Area of section A (cm^2)	Ratio for Local Buckling (D/t)	Second moment of area I (cm^4)	Radius of Gyration r (cm)	Elastic Modulus Z (cm^3)	Plastic Modulus Z (cm^3)
219,1	8	41,6	53,1	27,4	2960	7,47	270	357

Table 13: Characteristics of the CHS column

Because of the steel S275.

Designation	Yield strength (N/mm^2)	Ultimate strength (N/mm^2)
S275	275	410

Table 14: Characteristics of the column material. Steal S275.

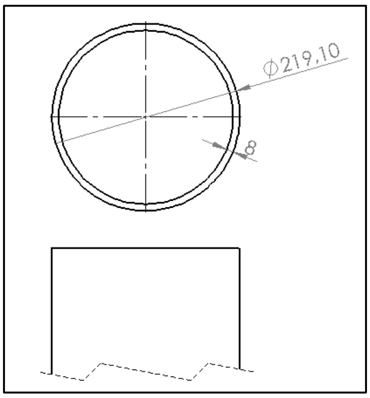


Figure 34: Column measures.

- BEAM:

Beam is an IPEA330. Moreover, because the aim of this thesis is to evaluate the reaction on the column for the beam is choose a steel S355 that means that the resistant of this material is higher than the steel of the fin plate or column.

On the next table, you can see the dimensions and properties:

Geometry									
h(mm)	b(mm)	t_f (mm)	t_w (mm)	r1(mm)	y_s(mm)	d(mm)	A(mm^2)	A_L(m^2 m^{-1})	G(kg/m)
327	160	10	6,5	18	80	271	5474	1,25	43

Section properties									
Axis y					Axis z				
I_y(mm^4)	W_{y1}(mm^3)	$W_{y,pl}$(mm^3)	I_y(mm)	S_y(mm^3)	I_z(mm^4)	W_{z1}(mm^3)	$W_{z,pl}$(mm^3)	I_z(mm)	S_z(mm^3)
1,02E+8	6,26E+5	7,02E+5	136,7	3,51E+5	6,85E+6	8,56E+4	1,33E+5	35,4	6,66E+4

Table 15: Geometry characteristics of the beam of the specific case

Warping and buckling			
I_w	i_w	I_t	i_{pc}
1,72E+11	39,6	1,96E+5	141,2

Table 16: Warping and buckling of the beam of the case

Designation	Yield strength (N/mm^2)	Ultimate strength (N/mm^2)
S355	355	470

Table 17: Material characteristics of the beam. Steel S355

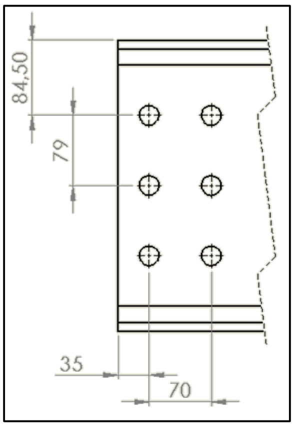


Figure 35: Beam measures with the hollow of the bolts

- Plate

h (mm)	b (mm)	t (mm)
228	150	10

Table 18: Fin plate dimensions

Designation	Yield strength (N/mm^2)	Ultimate strength (N/mm^2)
S275	275	410

Table 19: Material characteristics of the fin plate

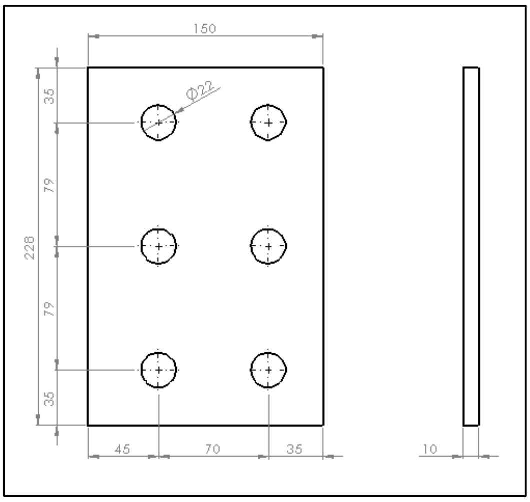


Figure 36: Fin plate measures with the hollow of the bolts

- Bolts

6 bolts M20 with designation 8.8.

Designation	Yield strength (N/mm^2)	Ultimate strength (N/mm^2)
8.8	640	800

Table 20: Characteristics of the bolts used on the specific case

d (mm)	d0 (mm)
20	22

Table 21: Characteristic of the diameters. Bolts diameter and hollow of bolt diameter

- Concrete:

The designation of the concrete is C25/30. On the Eurocode indicates that some of the relevant values are:

f_{ck} (MPa)	f_{cm} (MPa)	f_{ctm} (MPa)	E_{cm} (GPa)
25	33	2,6	30,5

Table 22: Resistance of the concrete C25/30.

3.2 Analytical calculations

As it is shown in 2.1.1 and 2.1.2 there are few calculations to anticipate how the composite structure is going to react.

One of the aims of that thesis is the comparison of a composite and non-composite column, because of that, the next calculations are going to be for a composite and steel cylinder tubular column.

3.2.1 Buckling calculation

One of the commons failures of a column is because of the bucking. It is obvious that the composite structure will resist more, but, how much more load can resist a composite column is the fact that it will be evaluate.

The next calculations are going to be theoretical calculations and for that, we must calculate by Euler equations. Euler's critical load defines the maximum load that a column can support before the buckling.

$$Pcr = \frac{\pi^2 \cdot E \cdot I}{Lcri^2} \quad \text{eq. 49}$$

Where the L_{cri} is the critical Euler length of column.

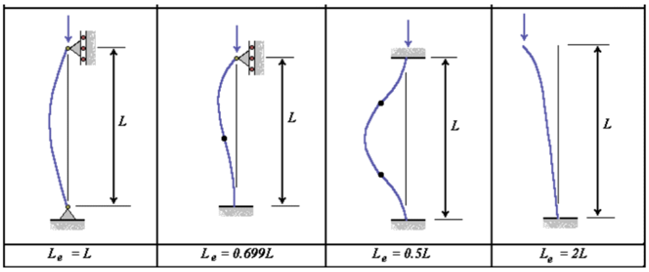


Figure 37: Definition of Lcri of Euler

In that case is going to choose the option that both limits are: rotation fixed and translation fixed.

The length of the column is decided to 3,5 meters because is a common length for that type of elements. Other values are going to be consider following Eurocodes. *[11]*

3.2.1.1 Steel column

For the steel column we need the properties of the section, these properties we can found it in a catalog that includes the section of the column that we want to analyze.

Ones we have the properties of the section (I) we need the properties of the material (E) in this case: Steel. The most common value for the young's modulus in steel is: 210.000 MPa (N/mm^2).

Now we can proceed to calculate with eq. 49:

$$Pcr = \frac{\pi^2 \cdot 210000 \cdot 2960 \cdot 10^4}{(0{,}5 \cdot 3500)^2}$$

$$Pcr = 20.032{,}477\ kN$$

3.2.1.2 Composite column

To calculate the maximum load for the bucking of a composite column it is going to use the same equation. The difference now is how to determinate the section and material properties.

These value is going to be absolutely theoretical because as we know to define a common value that englobes both materials are complex. Otherwise, the simplified method defines: (eq. 10)

$$EI = Ea * Ia + 0{,}6 * Ecm * Ic + Es * Is$$

As it is show in eq. 49:

$$EI = 210000 \cdot 2960 \cdot 10^4 + 0{,}6 \cdot 30500 \cdot 8352{,}37 \cdot 10^4$$

$$EI = 77444{,}8371 \cdot 10^8 Nmm^2$$

$$Pcr = \frac{\pi^2 \cdot 77444{,}8371 \cdot 10^8}{(0{,}5 \cdot 3500)^2}$$

$$Pcr = 24958{,}364\ kN$$

If the load is not on the center of the column exists a combination of axial and bending moment, furthermore, as we know from the theoretical bases 2.1 Behaviour peculiarities of composite we can study and evaluate the maximum axial and bending moment. To calculate the bucking of the composite column we will evaluate the local bucking and searching for the interaction curve. We want to obtain the maximum load that the joint, or in this case the column, can afford.

After developing the interaction curve, we will found a lineal rule for the case that we are studding and see which is the maximum load that the joint can afford.

The calculations of the interaction curves and the next axial compression is based on the Design guide 5: For concrete filled hollow sections columns under static and seismic loading.

Initial data:

Concrete	
f_{ck}(MPa)	25
f_{cm}(MPa)	33
f_{ctm}(MPa)	2,6
A_c (mm^2)	32397,37
I_c (mm^4)	83523700
E_{cm}(GPa)	31
Steel column	
d (mm)	219,1
t (mm)	8
f_y(MPa)	275
A_a(mm^2)	5310
I_a (mm^4)	29600000

Safety factors	
ϒa	1,1
ϒc	1,5
$ϒ_s$	1,15
Column length	
L(mm)	3500

Table 23: Initial data to calculate the local bucking

For knowing the interaction curve, first we need to study each one of the points that it is composed.

- Squash load (eq. 6) (eq. 11):

$$Npl,Rd = Aa \cdot fyd + Ac \cdot fcd + As \cdot fsd$$

$$Npl,Rd = 5310 \cdot \frac{275}{1,1} + 32397,37 \cdot \frac{25}{1,5}$$

$$Npl,Rd = 1867456\ N = 1876,456\ kPa$$

$$0,2 < \delta < 0,9$$

$$\delta = \frac{Aa \cdot fyd}{Npl,Rd}$$

$$\delta = \frac{5310 \cdot \frac{275}{1,1}}{1867456} = 0,711$$

- Check of local bucking (eq. 14)

  Being a circular hollow section:

$$\frac{d}{t} < 77$$

$$\frac{219,1}{8} = 27,39 < 77$$

- Axial compression
  Effective stiffness (eq. 10):

$$(EI)e = Ea \cdot Ia + 0,8 \cdot Ecd \cdot Ic + Es \cdot Is$$

$$(EI)e = 63892300 \cdot 10^8\ N/mm2$$

Bucking load (eq. 9):

$$Ncr = \frac{(EI)e \cdot \pi^2}{L^2}$$

$$Ncr = 5147691{,}051\ N$$

Relative slenderness (eq. 7):

$$\bar{\lambda} = \sqrt{\frac{Npl, Rd}{Ncr}} = \sqrt{\frac{1867456}{5147691{,}051}} = 0{,}664$$

Bucking curve *a*:

$\bar{\lambda}$	0,00	0,01	0,02	0,03	0,04	0,05	0,06	0,07	0,08	0,09
0,0	1,000	1,000	1,000	1,000	1,000	1,000	1,000	1,000	1,000	1,000
0,1	1,000	1,000	1,000	1,000	1,000	1,000	1,000	1,000	1,000	1,000
0,2	1,000	0,998	0,996	0,993	0,991	0,989	0,987	0,984	0,982	0,980
0,3	0,977	0,975	0,973	0,970	0,968	0,966	0,963	0,961	0,958	0,955
0,4	0,953	0,950	0,947	0,945	0,942	0,939	0,936	0,933	0,930	0,927
0,5	0,924	0,921	0,918	0,915	0,911	0,908	0,905	0,901	0,897	0,894
0,6	0,890	0,886	0,882	0,878	0,874	0,870	0,866	0,861	0,857	0,852
0,7	0,848	0,843	0,838	0,833	0,828	0,823	0,818	0,812	0,807	0,801
0,8	0,796	0,790	0,784	0,778	0,772	0,766	0,760	0,753	0,747	0,740
0,9	0,734	0,727	0,721	0,714	0,707	0,700	0,693	0,686	0,680	0,673
1,0	0,666	0,659	0,652	0,645	0,638	0,631	0,624	0,617	0,610	0,603
1,1	0,596	0,589	0,582	0,576	0,569	0,562	0,556	0,549	0,543	0,536
1,2	0,530	0,524	0,518	0,511	0,505	0,499	0,493	0,487	0,482	0,476
1,3	0,470	0,465	0,459	0,454	0,448	0,443	0,438	0,433	0,428	0,423
1,4	0,418	0,413	0,408	0,404	0,399	0,394	0,390	0,385	0,381	0,377
1,5	0,372	0,368	0,364	0,360	0,356	0,352	0,348	0,344	0,341	0,337
1,6	0,333	0,330	0,326	0,323	0,319	0,316	0,312	0,309	0,306	0,303
1,7	0,299	0,296	0,293	0,290	0,287	0,284	0,281	0,279	0,276	0,273
1,8	0,270	0,268	0,265	0,262	0,260	0,257	0,255	0,252	0,250	0,247
1,9	0,245	0,243	0,240	0,238	0,236	0,234	0,231	0,229	0,227	0,225
2,0	0,223	0,221	0,219	0,217	0,215	0,213	0,211	0,209	0,207	0.205

Table 24: Bucking curve. Eurocode.

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}}$$

$$\Phi = 0{,}5[1 + 0{,}21 \cdot (\bar{\lambda} - 0{,}2) + \bar{\lambda}^2]$$

$$\Phi = 0{,}5[1 + 0{,}21 \cdot (0{,}664 - 0{,}2) + 0{,}664^2] = 0{,}7692$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{0{,}7692 + \sqrt{0{,}7692^2 - 0{,}664^2}}$$

$$\chi = 0{,}86$$

Check for creep and shrinkage:

$$\bar{\lambda} < \frac{0{,}8}{1 - \delta} = \frac{0{,}8}{1 - 0{,}711} = 2{,}768$$

$$\bar{\lambda} = 0{,}664 < 2{,}768$$

$$No\ influence\ on\ the\ bearing\ capacity$$

Check for bearing capacity (eq. 5):

$$Nsd \leq NRd$$

$$Nsd \leq Npl,Rd \cdot \ \chi = 1606012 \text{ N}$$

$$Nsd < 1606{,}012\ kN$$

You must remember that load is in an axial way, without eccentricity.

- Cross-section interaction curve:

Plastic section moduli:

$$Wps = 0$$

$$Wpc = \frac{(d - 2 \cdot t)^3}{6} - Wps = \frac{(219{,}1 - 2 \cdot 8)^3}{6} - 0$$

$$Wpc = 1396299{,}299\ mm^3$$

$$Wpa = \frac{d^3}{6} - Wpc - Wps = \frac{219{,}1^3}{6} - 1396299{,}299 - 0$$

$$Wpa = 356676{,}3467\ mm^3$$

Interaction point D:

$$MD,Rd = Mmax,Rd = Wpa \cdot fyd + \frac{1}{2} \cdot Wpc \cdot fcd + Wps \cdot fsd$$

$$MD,Rd = 356676{,}3467 \cdot \frac{275}{1{,}1} + \frac{1}{2} \cdot 1396299{,}299 \cdot \frac{25}{1{,}5} + 0$$

$$MD,Rd = 100804917{,}2\ Nmm$$

$$ND,Rd = \frac{1}{2} \cdot Npl,c,Rd = \frac{1}{2} \cdot Ac \cdot fcd = \frac{1}{2} \cdot 32397{,}37 \cdot \frac{25}{1{,}5}$$

$$ND,Rd = 269978{,}083\ N$$

Interaction point C and B:

$$NC,Rd = 2 \cdot ND,Rd = Npl,c,Rd = Ac \cdot fcd$$

$$NC,Rd = 32397{,}37 \cdot \frac{25}{1{,}5} = 539956{,}167$$

For a circular hollow section filled of concrete h_n:

$$hn = \frac{Npl,c,Rd - Npl,s,Rd}{2 \cdot d \cdot fcd + 4 \cdot t \cdot (2 \cdot fyd - fcd)}$$

$$hn = 23{,}713\ mm$$

$$2 \cdot hn = 47{,}43\ mm$$

Plastic section moduli of the cross sectional areas in the region of $2 \cdot h_n$

$$Wpsn = 0$$

$$Wpcn = (d - 2 \cdot t) \cdot hn^2 = (219.1 - 2 \cdot 8) \cdot 23{,}713^2 = 114209{,}143$$

$$Wpan = 2 \cdot t \cdot hn^2 = 2 \cdot 8 \cdot 23{,}713^2 = 8997{,}274$$

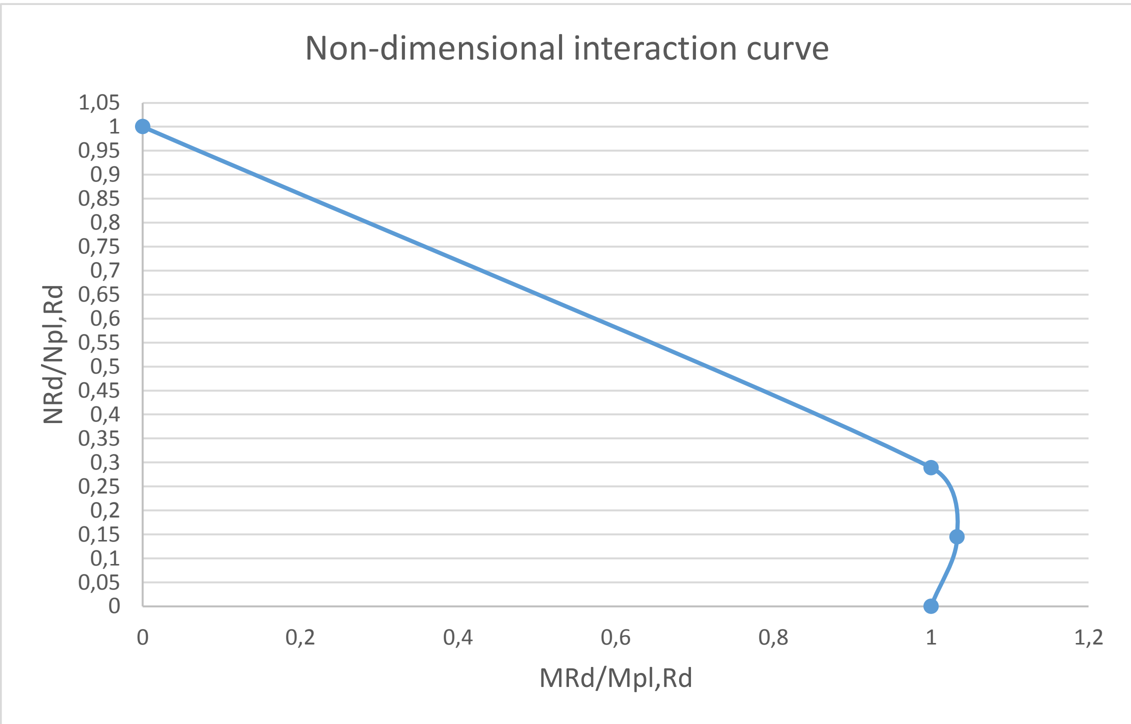
$$Mn, Rd = Wpan \cdot fyd + \frac{1}{2} \cdot Wpcn \cdot fcd + Wpsn \cdot fsd$$

$$Mn, Rd = 8997{,}274 \cdot \frac{275}{1{,}1} + \frac{1}{2} \cdot 114209{,}143 \cdot \frac{25}{1{,}5} = 3201061{,}286$$

$$Mpl, Rd = MB, Rd = 97603853\ Nmm$$

Non-dimensional interaction curve:

	$N_{Rd}/N_{pl,Rd}$	$M_{Rd}/M_{pl,Rd}$
A	0	1
B	1	0
C	1	0,28914
D	1,032796	0,14457



Graph 1: Non-dimensional of interaction on this specific case.

Now the question is: Which is the maximum load that the column can afford on this case?

We know that the bending moment is connected with the shear, because the bending moment appears because of the eccentricity of the shear, so we can know with is the maximum combination of both that the system can have before fail.

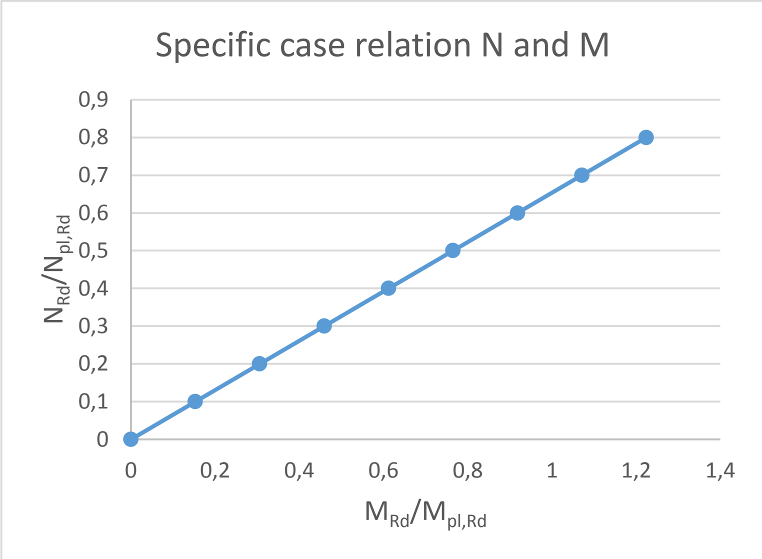
The condition that must be verify:

$$Msd = Nsd \cdot e$$

With this expression, we can found the rule that follows:

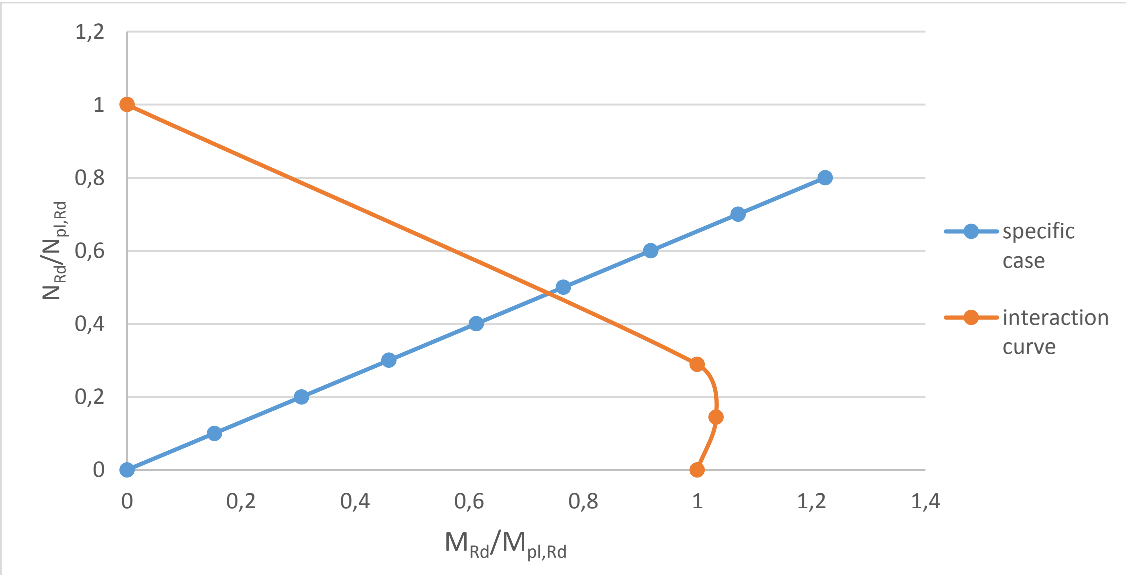
$N_{Sd}/N_{pl,Rd}$	$M_{Sd}/M_{pl,Rd}$
0	0
0,1	0,15306414
0,2	0,30612827
0,3	0,45919241
0,4	0,61225654
0,5	0,76532068
0,6	0,91838481
0,7	1,07144895
0,8	1,22451308

Table 25: specific points of graph



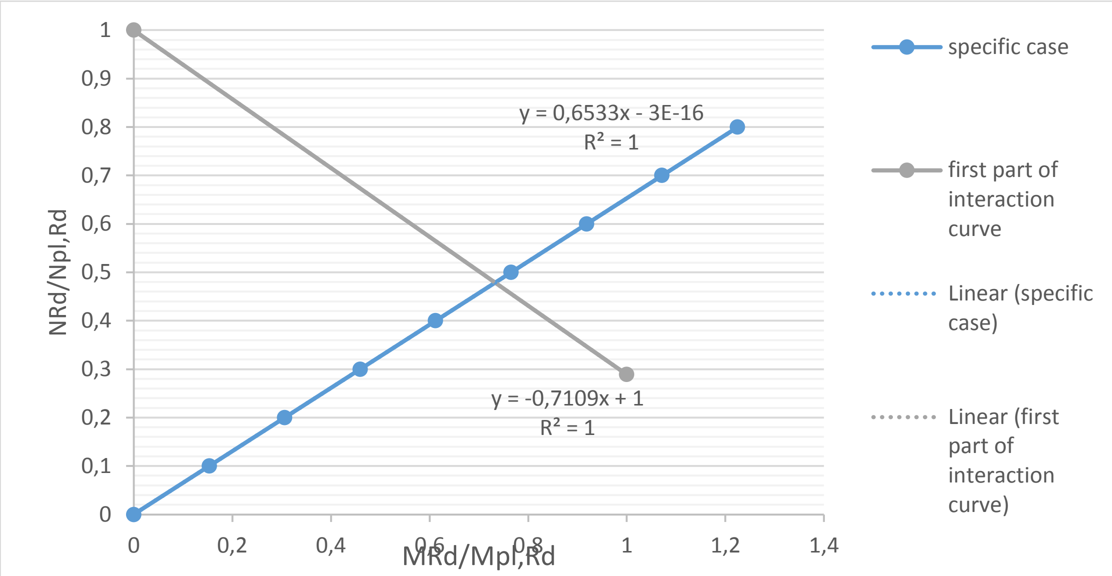
Graph 2: Relation between $N_{Rd}/N_{pl,Rd}$ and $M_{Rd}/M_{pl,Rd}$ on the specific case

If we mixt both graphs we can found with is the highest combination that the column can afford:



Graph 3: Interaction curve and specific case line.

Now the relevant part is analyse the point where both curves math.



Graph 4: Interaction curve and specific case graph with their lineal function

We can found that the point is defined by:

X	0,73303
Y	0,478889

$$x = \frac{MRd}{Mpl,Rd}$$

$$MRd = 71546552{,}3\ Nmm$$

$$MRd = 71{,}547\ kNm$$

$$y = \frac{NRd}{Npl,Rd}$$

$$NRd = 894304{,}216\ N$$

$$\boldsymbol{NRd = 894,304\ kN}$$

Therefore, for the meaning of buckling, the shearing load must be higher than 894,304kN on the beam.

3.2.2 Joint calculation

3.2.2.1 Steel column

For that type of connection, only made by steel, we must check Eurocode 3: Design of steel structures.

To be a successful joint, it is necessary to be verify:

- BEAM
  - Shearing resistance
  - Bearing resistance
  - Block tearing
- FIN PLATE
  - Shearing resistance
  - Bearing resistance
  - Block tearing
- BOLT
  - Shearing resistance
  - Bearing resistance
- WELD
- COLUMN
  - Punching shear resistance
  - Supporting column wall

The lowest maximum load that is calculated in that verifications will define the maximum load of the join and where.

- BEAM
  - Shearing resistance

$$VRd = \frac{Avz \cdot \frac{fy}{\sqrt{3}}}{\gamma M0} = \frac{26{,}99 \cdot 10^2 \cdot \frac{355}{\sqrt{3}}}{1{,}05} = 526{,}843kN$$

  - Bearing resistance (eq. 30)

$$Fb,u = \frac{k1 \cdot \propto b \cdot fu \cdot d \cdot t}{\gamma M2}$$

Where k1 is the minimum value:

For edge bolts:

$$2{,}8\frac{e2}{d0} - 1{,}7 = 2{,}8\frac{35}{22} - 1{,}7 = 2{,}75$$

$$1{,}4\frac{p2}{d0} - 1{,}7 = 1{,}4\frac{70}{22} - 1{,}7 = 2{,}75$$

$$\mathbf{2,5}$$

For inner bolts:

$$1{,}4\frac{p2}{d0} - 1{,}7 = 1{,}4\frac{70}{22} - 1{,}7 = 2{,}75$$

$$\mathbf{2,5}$$

Where ∝b is the minimum value:

∝d

$$\frac{fub}{fu} = \frac{800}{430} = 1{,}70$$

$$1$$

Where ∝d is the minimum value:

End bolts:

$$\frac{e1}{3 \cdot do} = \frac{84{,}5}{3 \cdot 22} = 1{,}28$$

Inner bolts:

$$\frac{p1}{3 \cdot do} - \frac{1}{4} = \frac{79}{3 \cdot 22} - \frac{1}{4} = 0{,}947$$

Finally we can obtain $F_{v,u}$ value:

$$Fb, u = \frac{2{,}5 \cdot 0{,}94697 \cdot 470 \cdot 22 \cdot 6{,}5}{1{,}25} = 127{,}292\ kN$$

For 3 bolts:

$$(Fb, u)total = 6 * Fb, u = \mathbf{763,750\ kN}$$

- Block tearing (eq. 35)

$$Veff = 0{,}5 \cdot fu \cdot \frac{Ant}{\gamma M2} + \frac{Anv \cdot \frac{fy}{\sqrt{3}}}{\gamma M0}$$

$$Veff = 0{,}5 \cdot 470 \cdot \frac{(35-22/2) \cdot 6{,}5}{1{,}25} + \frac{(84{,}5+79 \cdot 2-22 \cdot 2-11) \cdot 6.5 \cdot \frac{355}{\sqrt{3}}}{1{,}05}$$

$$Veff = \mathbf{325,883\ kN}$$

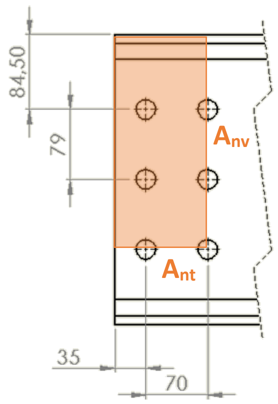


Figure 38: Block tearing beam

- FIN PLATE
  - Shearing resistance

$$Fv, Rd = \frac{\alpha v \cdot fu \cdot As}{\gamma M2}$$

The value of αv is 0,6 because the class of the bolt is 8.8
On Eurocode we can check the value of As. Bolt of 20 cm of diameter have 245 mm of As.

$$Fv, Rd = \frac{0{,}6 \cdot 800 \cdot 245}{1{,}25}$$
$$Fv, Rd = 94{,}08\ kN$$

For 6 bolts that there are on every fin plate:

$$n \cdot Fv, Rd = \mathbf{564,48\ kN}$$

- Bearing resistance (eq. 30)

$$Fb, u = \frac{k1 \cdot \propto b \cdot fu \cdot d \cdot t}{\gamma M2}$$

Where k1 is the minimum value of:

For edge bolts:

$$2{,}8\frac{e2}{d0} - 1{,}7 = 2{,}8\frac{35}{22} - 1{,}7 = 2{,}75$$

$$1{,}4\frac{p2}{d0} - 1{,}7 = 1{,}4\frac{70}{22} - 1{,}7 = 2{,}75$$

$$\mathbf{2,5}$$

For inner bolts:

$$1{,}4\frac{p2}{d0} - 1{,}7 = 1{,}4\frac{70}{22} - 1{,}7 = 2{,}75$$

$$\mathbf{2,5}$$

Where ∝b is the minimum value:

∝d

$$\frac{fub}{fu} = \frac{800}{430} = 1{,}70$$

1

Where ∝d is the minimum value:

End bolts:

$$\frac{e1}{3 \cdot do} = \frac{35}{3 \cdot 22} = \mathbf{0,53}$$

Inner bolts:

$$\frac{p1}{3 \cdot do} - \frac{1}{4} = \frac{79}{3 \cdot 22} - \frac{1}{4} = 0{,}947$$

Finally we can obtain $F_{v,u}$ value:

$$Fb, u = \frac{2{,}5 \cdot 0{,}53 \cdot 430 \cdot 22 \cdot 10}{1{,}25} = 95{,}66\ kN$$

For 6 bolts:

$$(Fv, u)total = 6 \cdot Fv, u = \mathbf{574,00\ kN}$$

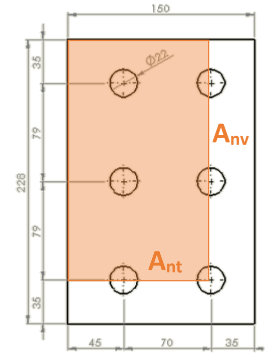


Figure 39: Block tearing fin plate

- Block tearing (eq. 35)

$$Veff = 0{,}5 \cdot fu \cdot \frac{Ant}{\gamma M2} + \frac{Anv \cdot \frac{fy}{\sqrt{3}}}{\gamma M0}$$

$$Veff = 0{,}5 \cdot 430 \cdot \frac{(45+70-22-11)\cdot 10}{1{,}25} + \frac{(35+79+79-2\cdot 22-11)\cdot 10 \cdot \frac{275}{\sqrt{3}}}{1{,}05}$$

$$Veff = \mathbf{343{,}15\ kN}$$

- BOLT
  - Shearing resistance (eq. 29)

$$Fv,Rd = \frac{\propto v \cdot fu \cdot A}{\gamma M2} = \frac{0{,}6 \cdot 800 \cdot 245}{1{,}25}$$

$$Fv,Rd = 94{,}08\ kN$$

$$(Fv,Rd)total = Fv,Rd \cdot 3(bolts)$$

$$(Fv,Rd)total = 564{,}48\ kN$$

- WELD

Hypothesis: upper and lower weld are not considered.

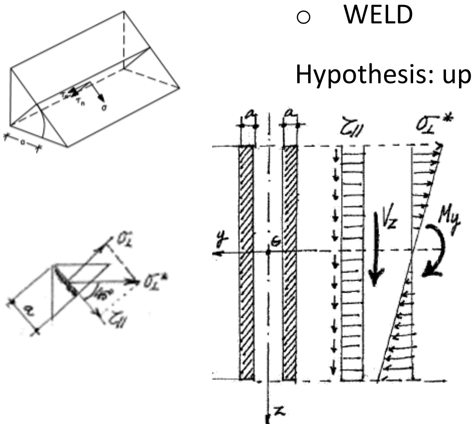


Figure 40: Representation of the weld, loads and stress that affects it.

$$\sqrt{\sigma_\perp{}^2 + 3 \cdot (\tau_\perp{}^2 + \tau\|^2)} \leq \frac{fu}{\beta w \cdot \gamma Mw} \ (\text{eq. } 27)$$

$$\tau_\perp = \sigma_\perp$$

$$\tau\| = \frac{F}{2 \cdot (l \cdot a)}$$

$$\sigma_\perp = \frac{1}{\sqrt{2}} \cdot \sigma_\perp * \left(\sigma_\perp * = \frac{M}{W} = \frac{F \cdot e}{2} \cdot \frac{6}{a \cdot L^2}\right)$$

$$\sqrt{\left(\frac{1}{\sqrt{2}} \cdot \frac{F \cdot e}{2} \cdot \frac{6}{a \cdot L^2}\right)^2 + 3 \cdot \left(\left(\frac{1}{\sqrt{2}} \cdot \frac{F \cdot e}{2} \cdot \frac{6}{a \cdot L^2}\right)^2 + \left(\frac{F}{2 \cdot l \cdot a}\right)^2\right)} \leq \frac{fu}{\beta w \cdot \gamma Mw}$$

$$\sqrt{\left(\frac{1}{\sqrt{2}} \cdot \frac{F \cdot 80}{2} \cdot \frac{6}{6 \cdot 228^2}\right)^2 + 3\left(\left(\frac{1}{\sqrt{2}} \cdot \frac{F \cdot 80}{2} \cdot \frac{6}{6 \cdot 228}\right)^2 + \left(\frac{F}{2 \cdot 228 \cdot 6}\right)^2\right)} \leq \frac{430}{0{,}85 \cdot 1{,}25}$$

If we analyse the maximum value of F to verify the expression F must be:

$$F < \mathbf{371{,}907\ kN}$$

If we evaluate the value of F:

$$F \leq 659{,}377\ kN$$

COLUMN

- Punching shear resistance

$$tp \leq tc \cdot \frac{fuc}{fy,p}$$

$$10 \leq 8 \cdot \frac{430}{275}$$
$$10 \leq 12{,}5$$

- Supporting column wall (Figure 28: Example of the calculation of column wall resistance. Eurocode 3.)

$$FRd,u = \frac{5 \cdot fy,c \cdot tc^2 \cdot (1 + 0{,}25 \cdot \eta) \cdot 0{,}67}{\gamma Mu}$$

Where η:

$$\eta = \frac{h1}{dc} = \frac{228}{219} = 1{,}041$$

$$FRd,u = \frac{5 \cdot 275 \cdot 8^2 \cdot (1 + 0{,}25 \cdot 1{,}041) \cdot 0{,}67}{1{,}1}$$
$$FRd,u = 67{,}54432\ kN$$

That is the horizontal load, if we want to know the higher shear load:

$$Mf = FRd,u \cdot h$$
$$Mf = F \cdot e$$
$$F = \frac{FRd,u \cdot h}{e} = \frac{67{,}544 \cdot 228}{80}$$
$$F = 192{,}501$$

RESULTS SUMMARY:

BEAM	Shearing resistance	526,843
	Bearing resistance	763,750
	Block tearing	325,883
FIN PLATE	Shearing resistance	564,480
	Bearing resistance	574,000
	Block tearing	343,151
BOLTS	Shearing resistance	564,480
WELDS	Resistance	371,907
COLUMN	Punching shear resistance	10≤12,5
	Supporting column wall	**192,501**

Table 27: Results summary of steel column joint.

That join can support a maximum load of 192,501kN or before the column wall fail.

3.2.2.2 Composite column

As we know and it was on the main topics on the theoretical part of the thesis, the calculations of the composite columns are not a rule nowadays, so is not something that we can know how to develop.

So in this part we will try to do some relevant calculations that we have nowadays.

How we can know why the composite joint breaks, for the beam and the bolts the criteria is the same, we have the same beam, fin plate and bolts, so:

BEAM	Shearing resistance	336,444
	Bearing resistance	308,16
	Block tearing	210,176
FIN PLATE	Shearing resistance	362,905
	Bearing resistance	257,94
	Block tearing	318,66
BOLTS	Shearing resistance	564,48
WELDS	Resistance	659,377

Table 28: Some results for composite column joint.

For the column, obviously, the results and the criteria change radically.

If we try to follow the Design guide 9, we can found on the section: Connections to concrete filled columns/ Simple shear connections/ connection design: "*for the design of the connection itself, it is generally recommended that the same criteria as given in chapter 5 (for hollow sections without concrete filling) be used. Concrete filling of the hollow section column prevents inward deformation of the column face, so the one column face rotational failure mode identified in chapter 5 (for just stiffened seat connections, in section 5.9) need not be considered with concrete filled columns. However, there is one important provision to these recommendations relating to fire conditions.*" *[10]*

Of course, the design guide indicate us to do the same calculations than before without considering the column wall failure. That means that our joint will fail on:

BEAM	Shearing resistance	336,444
	Bearing resistance	308,16
	Block tearing	**210,176**
FIN PLATE	Shearing resistance	362,905
	Bearing resistance	257,94
	Block tearing	318,66
BOLTS	Shearing resistance	564,48
WELDS	Resistance	659,377

Table 29: Some results for composite column joint. Re-marking with is the failure load.

Block tearing of the beam, but what happens if we want to keep understanding on this new joint how is the concrete and steel reaction on the column wall. Does not exist any norm that explain it to us.

Knowing that we do not have any rule that define it we will try to do some calculations that can orientated the joint. we decided to try to calculate with the same method that is calculated the joints between a column and a base plate made of concrete.

To study how many load supports the concrete we can follow the same criteria than in a column joint with a concrete base plate. We will calculate the design compression flange Fc,Rd:

$$Fc, Rd = fjd * beff * leff$$

$$fjd = \frac{0.6\, fcu}{\gamma c} = \frac{0,6 * 30}{1,5} = 12\ MPa$$

$$c = t * \left(\frac{fy}{3 * fjd * \gamma M0}\right)^{0,5} = 26,97$$

$$Fc, Rd = 12 * (2 * 26,97 + 10) * (114 * 26,97)$$

$$Fc, Rd = 108,163\ kN$$

$$F = \frac{Fc, Rd * 228}{e} = 308,265\ kN$$

COLUMN	column wall	308,265

Because we know that this calculations could not represent the reality we will calculate for different fin plate thickness to compare it:

COLUMN	fin plate thickness 10 mm	308,265 kN
	fin plate thickness 5 mm	138,536 kN
	fin plate thickness 15 mm	506,682 kN
	fin plate thickness 20 mm	734,484 kN
	fin plate thickness 50 mm	2721,204 kN

3.3 Numerical modelling

3.3.1 Buckling modelling

To calculate the critical load of the column in ANSYS it is going to do a linear buckling analysis. ANSYS allowed doing an Eigen buckling analysis. Because of that, the column will be modeled with their section and material properties and then a 1N force is going to be introduced on the top.

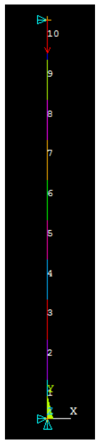


Figure 41: Column representation with ANSYS APDL.

The conditions are:

- Displacement: First and last node is going to be restricted without any movement. These nodes cannot have a displacement or rotation, except one, the vertical movement of the top node. That vertical axis have to be free if we want to analyze the local buckling.
- Load: it is going to be introduce one vertical load with the value of 1 N. this value is decided to be 1 because we can see on the solutions the critical load directly. The solutions give to us the multiply factor, if we introduce a load with that multiply factor the column will collapse.

3.3.1.1 Steel column

For this simulation the conditions are:

EX		2.1E+005
Cross-sectional area	AREA	5310
Area moment of inertia	IZZ	29600000

Figure 42: Linear isotropic properties material. Steel column.

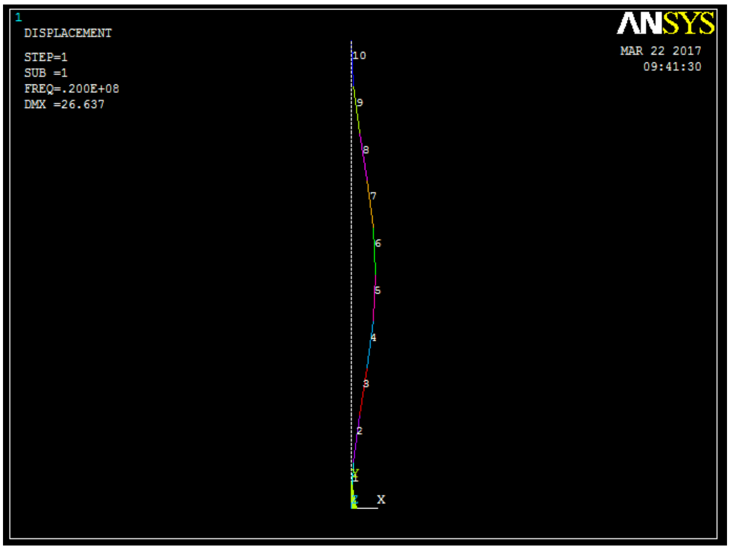
The results are:

```
 *****  INDEX OF DATA SETS ON RESULTS FILE  *****

 SET   TIME/FREQ    LOAD STEP   SUBSTEP  CUMULATIVE
   1 0.20037E+08        1          1         1
   2 0.41017E+08        1          2         1
   3 0.80387E+08        1          3         1
   4 0.12198E+09        1          4         1
   5 0.18298E+09        1          5         1
```

Figure 43: solutions for the first 5 modes of buckling. Steel column.

That means for the first module of collapse of the column the load have to be multiply by 20037000. Because of the load have a value 1N we can know directly that for the first module of buckling the critical load is:



$$Pcr = 20.037,00\ kN$$

Figure 44: First mode of bucking. Steel column.

The representations of the different modules are:

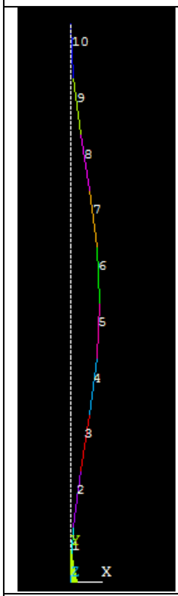
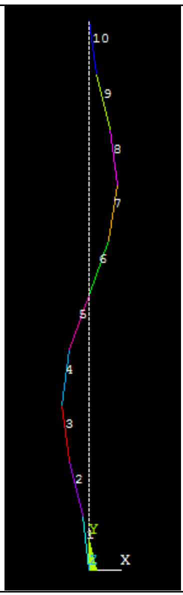
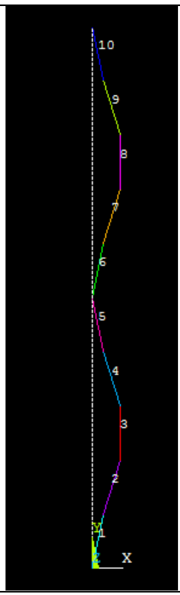
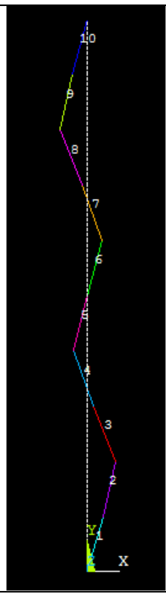
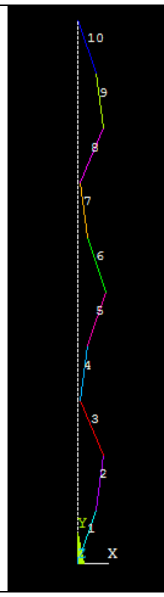
1st modulus	2nd modulus	3rd modulus	4th modulus	5th modulus
				
P_{cri}=20.037,00 kN	P = 41.017 kN	P = 80.387 kN	P = 121.980 kN	P = 182.980 kN

Table 30: 5 fist modes and value of the load. Steel column.

With these representations we can see that for the 3rd, 4th and 5th it is not enough dividing the column just in 10 elements. In that case, the only case that is relevant is the first.

3.3.1.2 Composite column

That case is more complex. The global material properties of the column are not clear. We know that the Eurocode 3 allowed defining EI as (eq. 10):

$$EI = Ea * Ia + 0,6 * Ecm * Ic + Es * Is$$

Therefore, it is going to do the same process that in the steel column case but with different EI. The results are:

```
*****  INDEX OF DATA SETS ON RESULTS FILE  *****

 SET   TIME/FREQ    LOAD STEP   SUBSTEP  CUMULATIVE
   1 0.24964E+08        1           1         1
   2 0.51103E+08        1           2         1
   3 0.10015E+09        1           3         1
   4 0.15198E+09        1           4         1
   5 0.22797E+09        1           5         1
```

Figure 45: first 5 modes of bucking. Composite column.

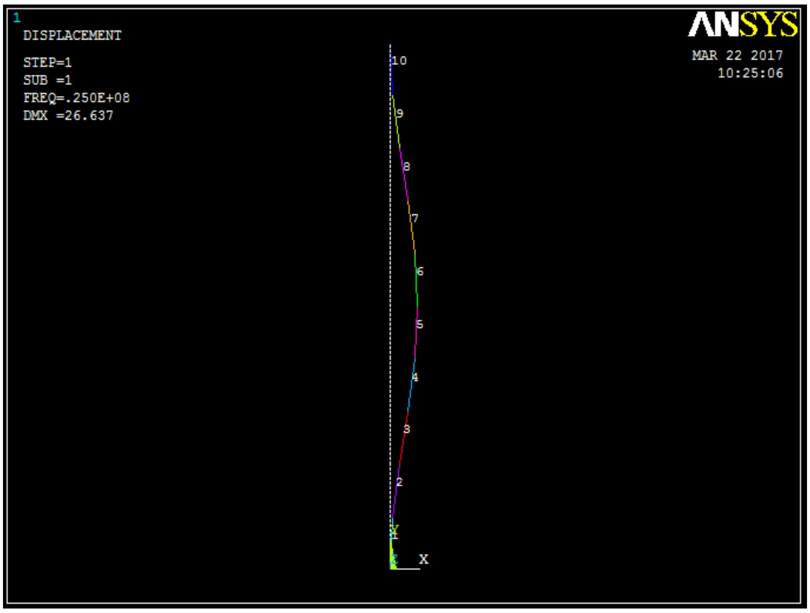


Figure 46: First mode of bucking. Composite column.

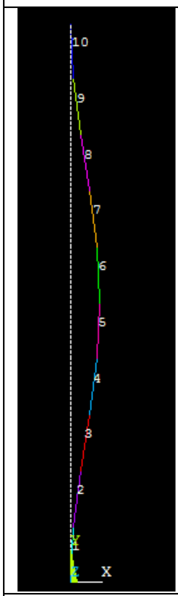
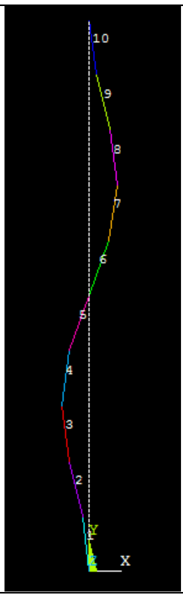
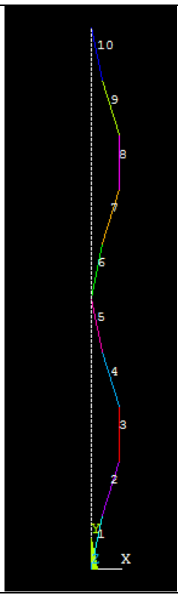
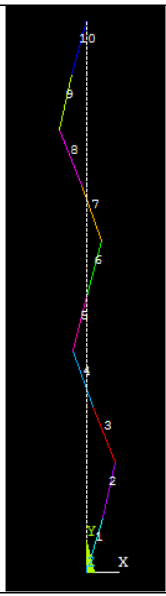
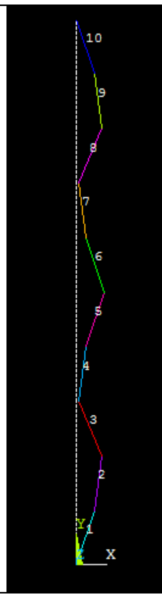
1st modulus	2nd modulus	3rd modulus	4th modulus	5th modulus
				
Pcri=24.964 kN	P = 51.103 kN	P = 100.150 kN	P = 151.980 kN	P = 227.970 kN

Figure 47: Fist modes and value of the load. Composite column

3.3.2 Joint modelling

For the joint modelling was decided to use ANSYS workbench. It was decided to use Workbench and not APDL because of the modelling of connections and more than one element.

On one hand, on ANSYS APDL you do not have any error, the model should be free from errors.

On the other hand, ANSYS workbench have an easier interface and the update of geometries and boundaries too. Solve of solid and not shells are clearer.

To sum up, if you want to do an simulation to evaluate how and where is fail and see the solid and the solutions on more than one point it is recommended to use Workbench because is more easily done, but if you have a simple model and you want precise values of solutions is appropriate to use ANSYS APDL.

3.3.2.1 Steel modelling

Material properties about this steel is defined in Eurocode 2. Where we can found

Structural steel grade at 16 mm	Minimum yield strength at nominal thickness 16 mm	
	Ksi	N/mm^2
S275	36000	275

Table 31: Material properties S275.

Structural steel grade	Tensile strength MPa at Nom thickness between 3 mm and 16 mm
S275	370-530 MPa

Table 32: Material properties S275

On the case we are in a Non-lineal case, we do not want to see when the steel starts to deform or before the plasticity, we want to evaluate the joint before the broke. That means that we will not see just the first elastics moments of the material we will see the plasticity of the material and that means that we need to define the material as a: Nonlinear material.

The rule that is choose to follow is: **Bilinear Isotropic Hardening**

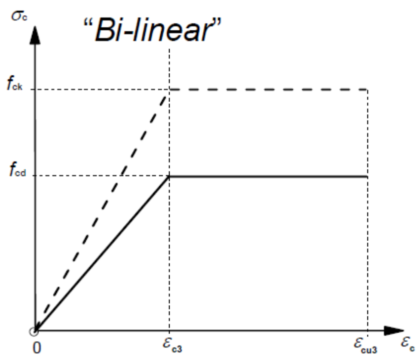


Figure 48: Qualitative representation of a stress-strain bilinear curve

The plasticity model defines the permanent deformation of the model. Exists different models that can define the plasticity depending on the material, such as bilinear or multi-lineal. For steel, we can suppose that the material follow a lineal rule on the elasticity deformation and another lineal rule for the plasticity deformation.

The reaction on the tension and compression is suppose that is equal; the yield surface is considered Von Mises. The Von Mises yield criteria is defined by the principal stresses. *[12]*

The input parameters for the definition of this method are:

$$fy = 275\ MPa$$

$$E = 210.000 MPa$$

$$\nu = 0,3$$

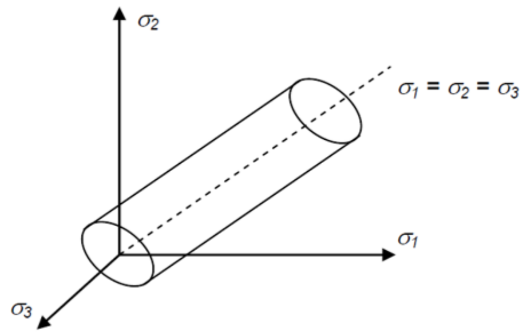


Figure 49: Representation of Von Misses criteria

With these parameters, we can draw the first part of the bilinear method. (The elastic part)

If we want to analyse the second part (plasticity part) we must introduce the inputs tangent modulus. The tangent modulus is equivalent to the Young's modulus, some studies defined the tangent modulus like the 0,0001 fraction of the young's modulus. On that case, because we want to evaluate the plasticity like a perfect plasticity we will defined the tangent modulus like a 0 MPa.

Plastic modulus H:

H>0 Hardening Plasticity

H=0 Perfect Plasticity

H<0 Softening Plasticity

Moreover, that means that the Et=0 for the perfect plasticity.

Therefore, at final we can defined the material on the ANSYS program like:

Property	Value	Unit
Isotropic Elasticity		
Derive from	Young's Modulus an...	
Young's Modulus	2,1E+05	MPa
Poisson's Ratio	0,3	
Bulk Modulus	1,75E+11	Pa
Shear Modulus	8,0769E+10	Pa
Bilinear Isotropic Hardening		
Yield Strength	275	MPa
Tangent Modulus	0	MPa

Figure 51: material properties included in ANSYS

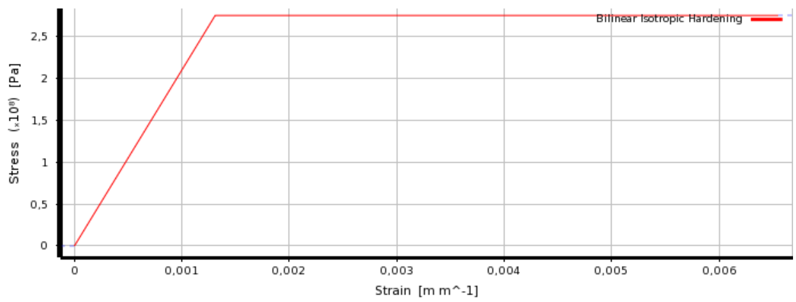


Figure 50: Bilinear Isotropic Hardening curve

We can analyses that the isotropic elasticity defines the first part of the bilinear, and the bilinear isotropic hardening defines the seconds inputs, where yield strength is the maximum stress that the material can supported before the plasticity.

3.3.2.2 Concrete modelling

On the ANSYS program, we have a material that defines properly the concrete, Solid65. *[14]*

Solid65 is a material that can simulate the cracking in tension and crushing in compression. That means that have the capacity of different behaviour in compression and tension.

One of the restrictions for using this material is that cannot be used on a shell model, must be a solid because the material is represented in 3D. This material is defined by eight nodes and three grades of freedom each one: translation in all the directions.

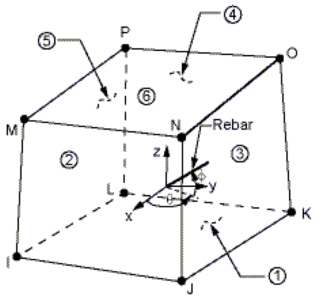


Figure 52: Solid65 material represented in ANSYS [14]

The most important aspect of this material is the property of representation non-linearity of the material, can crash, crack and have plasticity deformation.

The concrete material will be defined by diferents properties on codes:

- Linear isotropic: to define poisson and young modulus

```
ET,MATID,SOLID65    !MATERIAL SOLID65
MP,EX, MATID,30500  !EX-Elastic moduli (also EY, EZ).
MP,PRXY,MATID,0.2   !PRXY poison
```

- Multilinear Isotropic Hardening: to defined the stress-strain curve more acurate. It is decided to defined like a multilinear isotropic hardening curve, now the question is which curve we must introduce to represent the reality.
  Eurocode 2 indicates which type of curve must be used depending on the simulation:

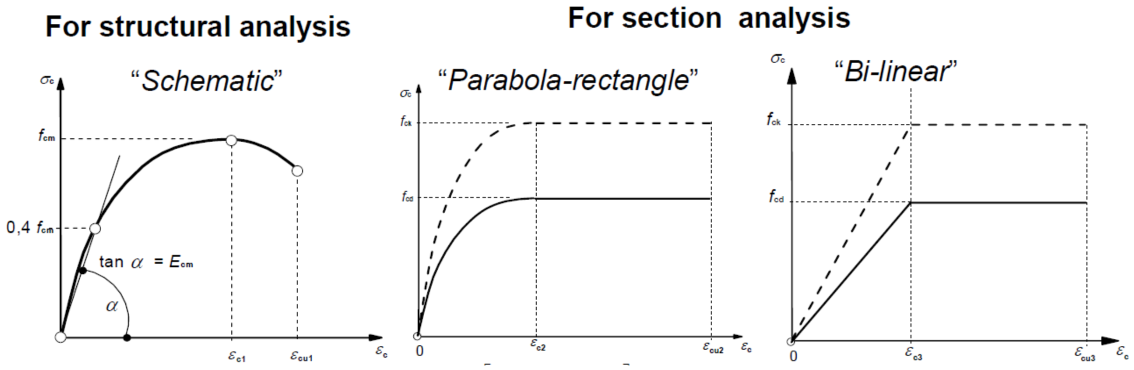


Figure 53: 3 different types to represent stress-strain curve. Eurocode 2. [6]

As we can see in our case we are in a structural analysis so we can use the Schematic curve. On the same Eurocode 2 indicates the ecuation that must follow:

$$\frac{\sigma c}{fcm} = \frac{k * \eta - \eta^2}{1 + (k-2) * \eta} \qquad \text{eq. 50}$$

Where $\eta = \frac{\varepsilon c}{\varepsilon c1}$

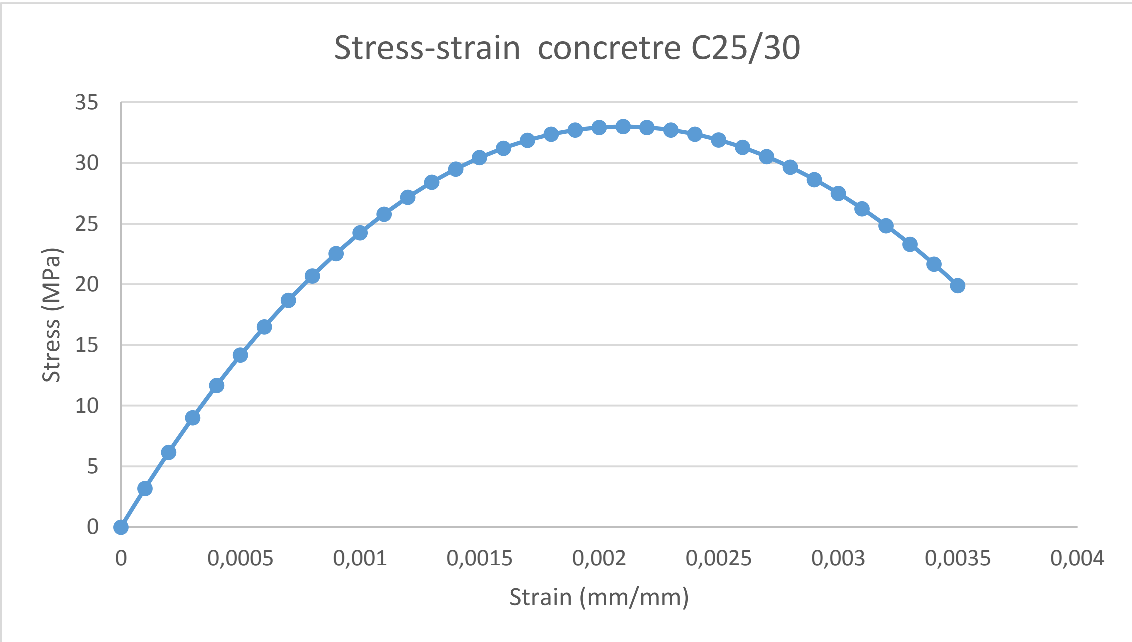
$\varepsilon c1$ is indicate on the *Table 33*: Strength classes for concrete. (Eurocode 2)

$k = 1{,}05 * Ecm * \frac{|\varepsilon c1|}{fcm}$ (fcm indicate on the *Table 33*: Strength classes for concrete. (Eurocode 2))

	Strength classes for concrete													
f_{ck} (MPa)	12	16	20	25	30	35	40	45	50	55	60	70	80	90
$f_{ck,cube}$ (MPa)	15	20	25	30	37	45	50	55	60	67	75	85	95	105
f_{cm} (MPa)	20	24	28	33	38	43	48	53	58	63	68	78	88	98
f_{ctm} (MPa)	1,6	1,9	2,2	2,6	2,9	3,2	3,5	3,8	4,1	4,2	4,4	4,6	4,8	5,0
$f_{ctk,0,05}$ (MPa)	1 1	1,3	1,5	1,8	2,0	2,2	2,5	2,7	2,9	3,0	3,1	3,2	3,4	3,5
$f_{ctk,0,95}$ (MPa)	2,0	2,5	2,9	3,3	3,8	4,2	4,6	4,9	5,3	5,5	5,7	6,0	6,3	6,6
E_{cm} (Gpa)	27	29	30	31	32	34	35	36	37	38	39	41	42	44
ε_{c1} (‰)	1,8	1,9	2,0	2,1	2,2	2,25	2,3	2,4	2,45	2,5	2,6	2,7	2,8	2,8
ε_{cu1} (‰)	3,5									3,2	3,0	2,8	2,8	2,8
ε_{c2} (‰)	2,0									2,2	2,3	2,4	2,5	2,6
ε_{cu2} (‰)	3,5									3,1	2,9	2,7	2,6	2,6
n	2,0									1,75	1,6	1,45	1,4	1,4
ε_{c3} (‰)	1,75									1,8	1,9	2,0	2,2	2,3
ε_{cu3} (‰)	3,5									3,1	2,9	2,7	2,6	2,6

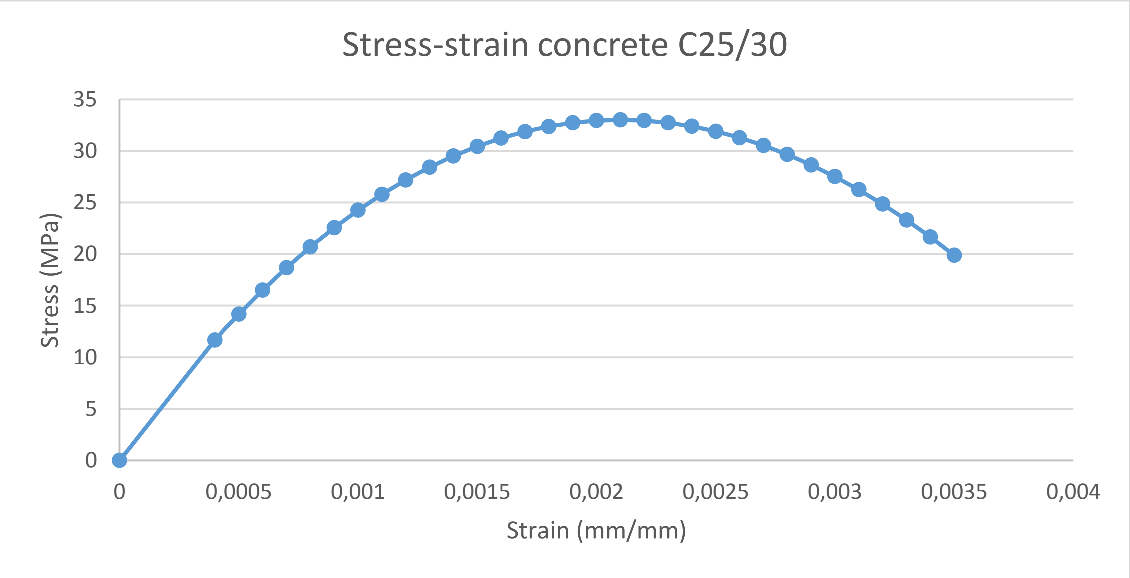
Table 33: Strength classes for concrete. (Eurocode 2) [6]

In that case the concrete is C25 and because of that we can define the stress and strain curve as:



Graph 5: Stress-strain of concrete C25/30. Eurocode 2.

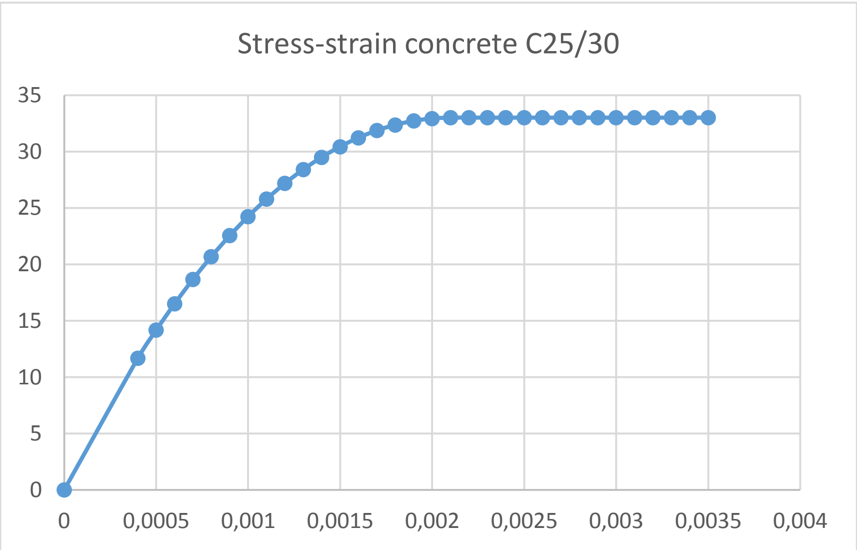
If we introduce all the curve on the ANSYS programe an error appears. The ANSYS always calculate if the youngh modulus of the curve is the same than the ones that we had introduce on the lineal isotropic data. Because of that the curve must be lineal on the elastic part, considered the firts 40% of the fcm.



Graph 6: Stress-strain curve introduced in ANSYS concrete material.

Because the Newton-Rapson method of calcualtion can not afford negative gradients the first simulations have been done with just the positive gradient of the curve, and we saw that we arribe correctly to the solution. Otherwise, just in case the program needs valuels higher than Ɛc1 of the curve it was decided to represent as a material with a perfect plasticity. That does not affect on the reality of the solutions. The final curve and code to implement that properties are:

```
TB,MISO,MATID,1,35,0
TBTEMP,22
TBPT,,0.000433,13.2
TBPT,,0.0005,14.1636
TBPT,,0.0006,16.4997
TBPT,,0.0007,18.6741
TBPT,,0.0008,20.6885
TBPT,,0.0009,22.5442
TBPT,,0.001,24.2432
TBPT,,0.0011,25.7867
TBPT,,0.0012,27.1763
TBPT,,0.0013,28.4136
TBPT,,0.0014,29.5
TBPT,,0.0015,30.4369
TBPT,,0.0016,31.2258
TBPT,,0.0017,31.8682
TBPT,,0.0018,32.3655
TBPT,,0.0019,32.719
TBPT,,0.002,32.9301
TBPT,,0.0021,33
TBPT,,0.0022,33
TBPT,,0.0023,33
TBPT,,0.0024,33
TBPT,,0.0025,33
TBPT,,0.0026,33
```



Graph 7: Stress-strain curve introduced in ANSYS concrete material

- Concrete Material Data: there is the option with the material SOLID65 to introduce some data. This data is refered to the plasticity parametres of the concrete. In this case it was decided to use the William-Wranke Five-Parameter Criterion *[13]*. On the FEM program (ANSYS) there is the option that if you include a 0 on the value of a parameter the program will calculate with the information that it has about the material.

  Each property is defined by a number and the meanings are the next ones:

Constant	Meaning	Value
1	Shear transfer coefficients for an open crack (βt)	0.4
2	Shear transfer coefficient for a closed crack (βc)	0.8
3	Uniaxial tensile cracking stress (fr)	2.6
4	Uniaxial crushing stress (positive) (fc)	33.3
5	Biaxial crushing stress (positive)	0
6	Ambient hydrostatic stress state for use with constants 7 and 8	0
7	Biaxial crushing stress (positive) under the ambient hydrostatic stress state (constant 6)	0
8	Uniaxial crushing stress (positive) under the ambient hydrostatic stress state (constant 6)	0
9	Stiffness multiplier for cracked tensile conditions, used if KEYOPT(7) = 1 (defaults to 0.6).	0

  Table 34 : Concrete material data

  The shear transfer coefficient, β, represents conditions of the crack face. The value of β ranges from 0 to 1, with 0 representing a smooth crack (complete loos of shear transfer) and 1 representing a rough crack (no loss of shear transfer).

  The code to implement those parameters is:

```
TB,CONCRE, MATID,1,9
TBTEMP,0
TBDATA,1,0.4,0.8,2.6,33
```

  Another way to write it, could be more intuitive:

```
TB,CONC,1,1,9,         !CONCRETE PROPERTIES (NON-METAL PLASTICITY)
TBTEMP,0
TBDATA,1,0.4           !SHEAR TRANSFER COEFFICIENTS FOR AN OPEN CRACK
TBDATA,2,0.8           !SHEAR TRANSFER COEFFICIENT FOR A CLOSED CRACK
TBDATA,3,2600000       !UNIAXIAL TENSILE CRACKING STRESS
TBDATA,4,33000000      !UNIAXIAL CRUSHING STRESS
TBDATA,5,0             !BIAXIAL CRUSHING STRESS
TBDATA,6,0             !AMBIENT HYDROSTATIC STRESS STATE FOR USE WITH 7 AND 8
TBDATA,7,0             !BIAXIAL CRUSHING STRESS UNDER THE AMBIEN…
TBDATA,8,0             !UNIAXIAL CRUSHING STRESS
TBDATA,9,0             !STIFFNESS MULTIPLIER FOR CRACK TENSILE CONDITIONS
```

With these three codes (Linear isotropic, Multilinear isotropic and Concrete data) the concrete will be precise enough for the static structural analysis that we want to evaluate.

3.3.2.3 Geometry

The initial model is composted by more than one element:

- Beam
- Bolts
- Column

The aim of the thesis is see the connections and the answer of the column when is filled by concrete, because of that, it was decided to simplify the model as much as we can.

Of course the entire column must be modelled and the concrete, the elements that will be deleted are the bolts and the beam and the element that will be simplify will the fin plate.

We can consider that the representations is accurate because the reaction on the column will be the same than with the beam and bolts.

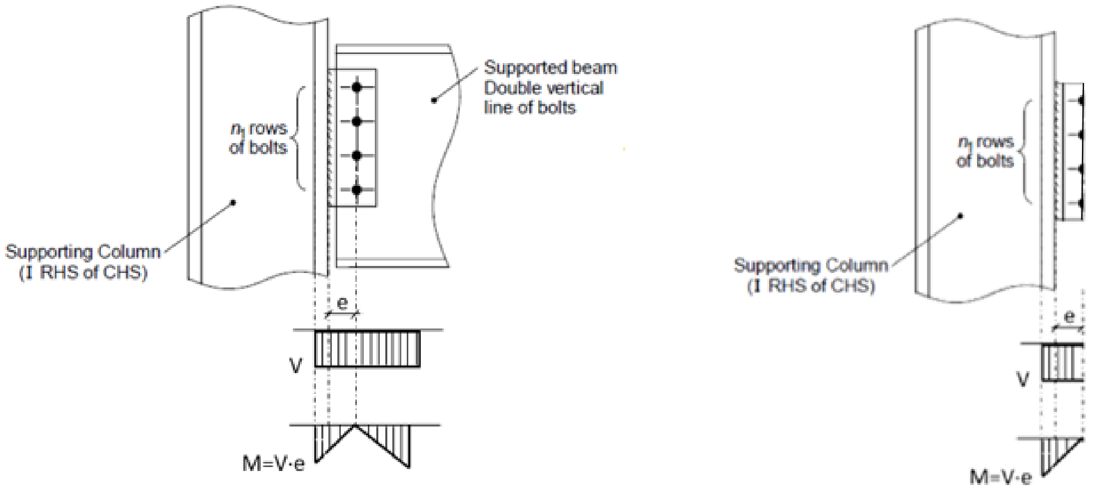


Figure 54: representation of the internal loads and simplification of the real model

To represent on accurate way the reality but being simpler the final model is:

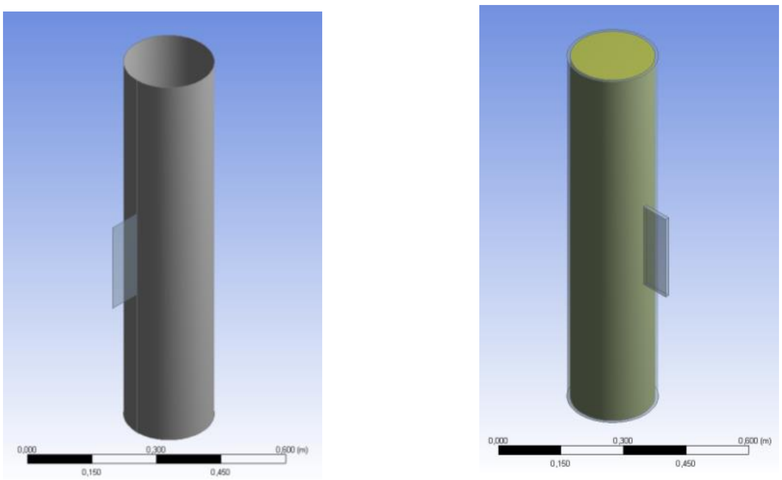


Figure 55: Model of the steel and composite column with the fin plate.

The modelling of the fin plate is the half of it. That means that the load will be located on the point that we consider the joint.

For the representation of the contact, on this thesis, it was suppose that with a simple contact region between the contact and the steel is enough. However, on the reality between steel and concrete must be introduce a friction surface. Because of the complex and the time, it was decided to represent by a simple contact at in this case and the results are accurate at all.

3.3.2.4 Finite Element Mesh

The meshing is one of the most important points in a FEM analysis. Depending on the number of elements the results are more accurate or not, but it was studied that arrives one moment than you can include more elements that the results will be very similar but the calculations (matrix) grows to much for the computer program. That means that is need to follow a principle to evaluate when is enough materials for not wasting time doing analysis but having accurate results.

In this case, we decided to study the same model with different size of elements and define which one is the ones that we considered enough.

The size of the mesh is not enough when you are doing a FEM analysis; depending on the shape of the model is need to define a number of divisions to control better the model. Furthermore, when the model is created by a shell the meshing could be easier to control, you do not have to control de width of the elements, when is a solid we have to be careful with the proportion width-high it is studied that the difference between them cannot be higher than 5.

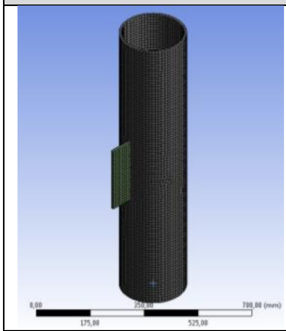
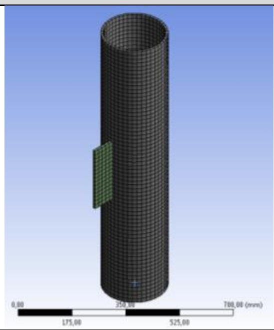
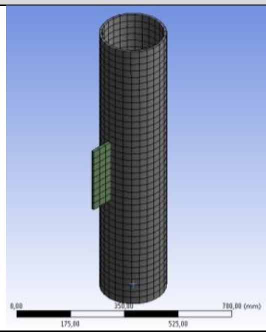
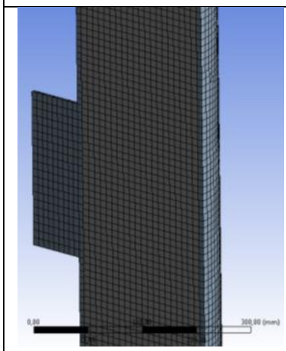
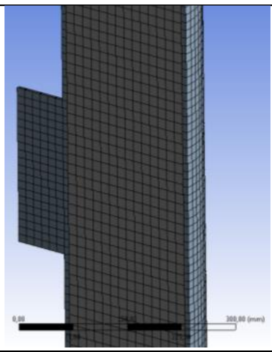
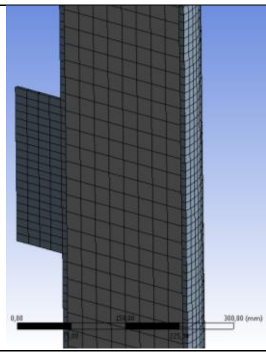
Mesh 10 mm	Mesh 15 mm	Mesh 25 mm
		
6.767 elements	2.979 elements	1.067 elements
		
25.337 elements	48.685 elements	4.575 elements

Figure 56: Different size of mesh in steel and composite model and the number of elements the represents.

On section of the results 3.3.2.6 Results you can found different solutions with different size of mesh, sometimes with some restrictions.

Doing the analysis was need to do some restrictions to control the mesh and be accurate with the solutions. Some of the restrictions are:

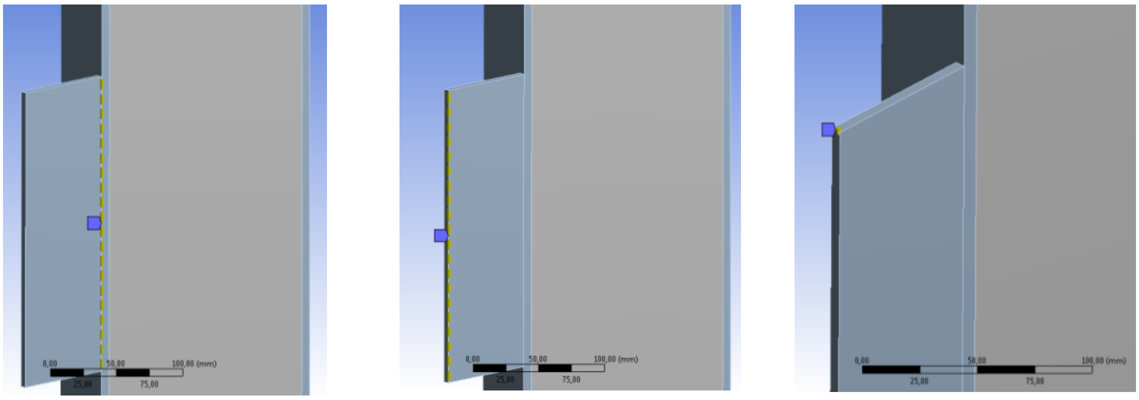


Figure 57: Representation of some limitations to have a precise mesh

To make sure that the fin plate is well defined it was decided to limit the number of elements in the edges. 20 elements on the length and 2 elements on the width symmetry, that means 4 elements in all width plate.

3.3.2.5 Loading and Boundary Conditions

The column length was evaluate too, and in some results you can find how the answer varieties because of this fact. The general rule was to have a 1 m column, that was considered enough accurate and comparing with the number of elements that a 3,5 meters can increase (3,5 m is the length that columns used to have because of the floor high on a building.

Because of that, the model used to be 1 m column and the boundary conditions, that do not variety with the different length, are fixed on the top and the bottom of the column.

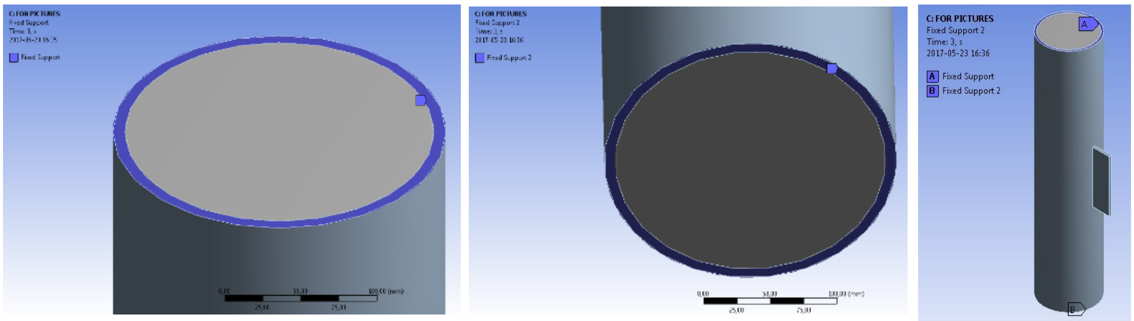


Figure 58: Boundary conditions for the column.

To have a better image and analysis sometimes it was impose the symmetry condition.

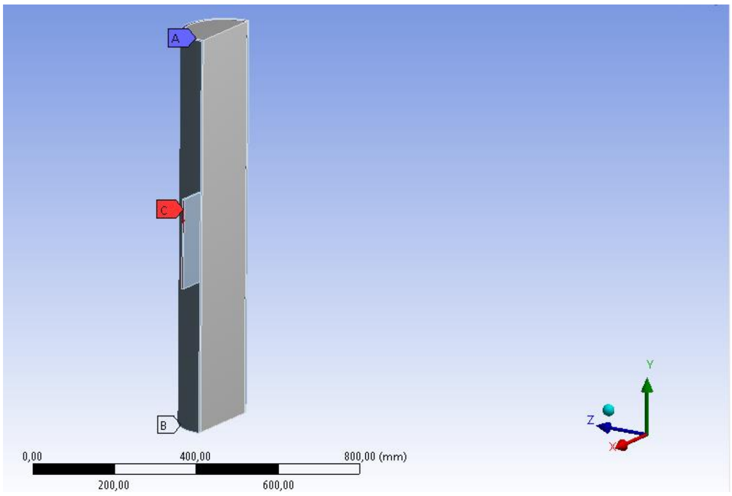


Figure 59: Boundary conditions and external load. Symmetric representation.

3.3.2.6 Results

On this section of the thesis, we will see the results that we obtain from the ANSYS program.

3.3.2.6.1 Steel joint

As we talked in the lasts points, the modelling of the steel joints is just with one material: Steel and with the column without filled of concrete. The qualitative solutions of the joint are:

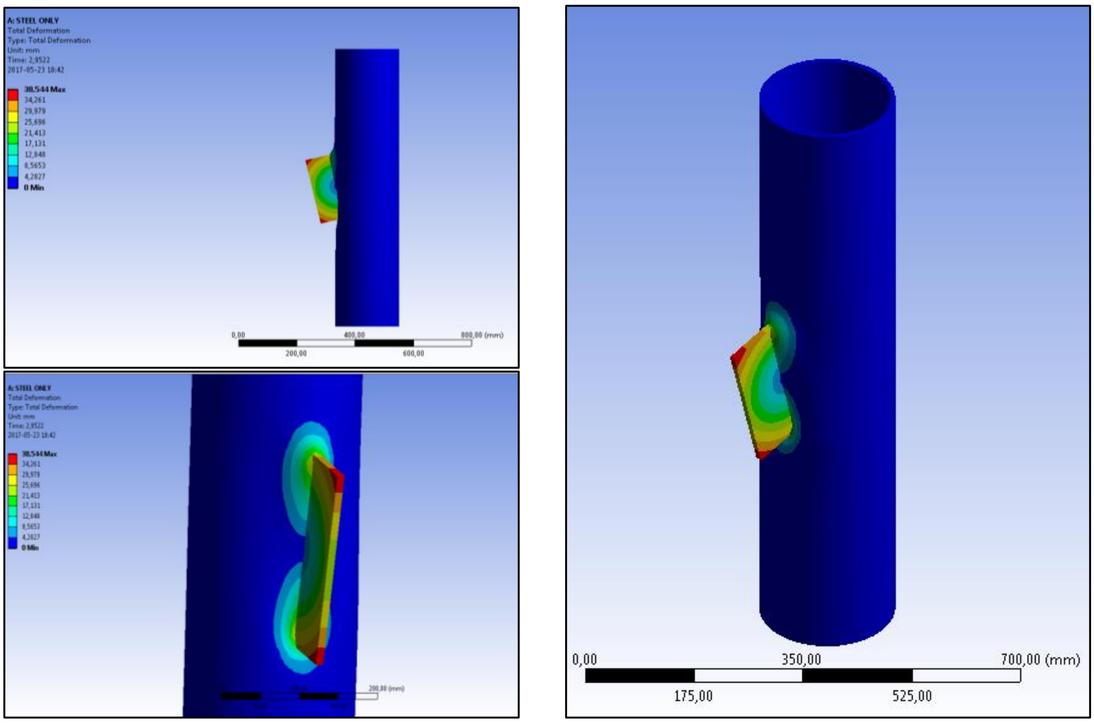


Figure 60: Deformation qualitative pictures. ANSYS. Mesh 10 mm. View results in: 21 (0,5xAuto results).

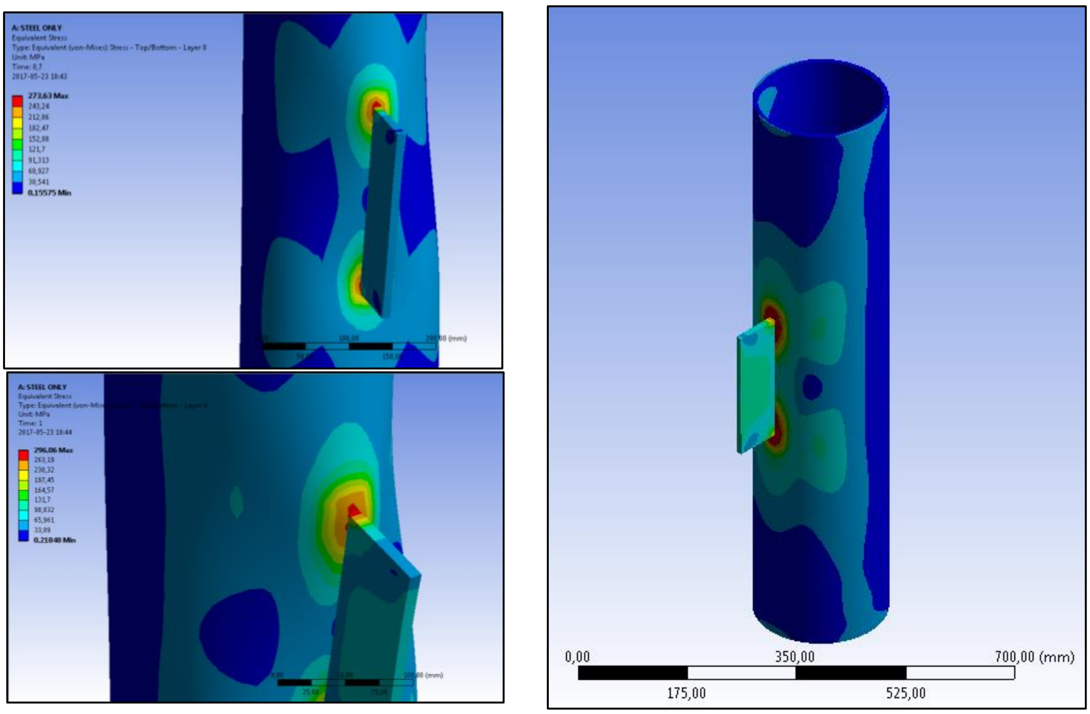
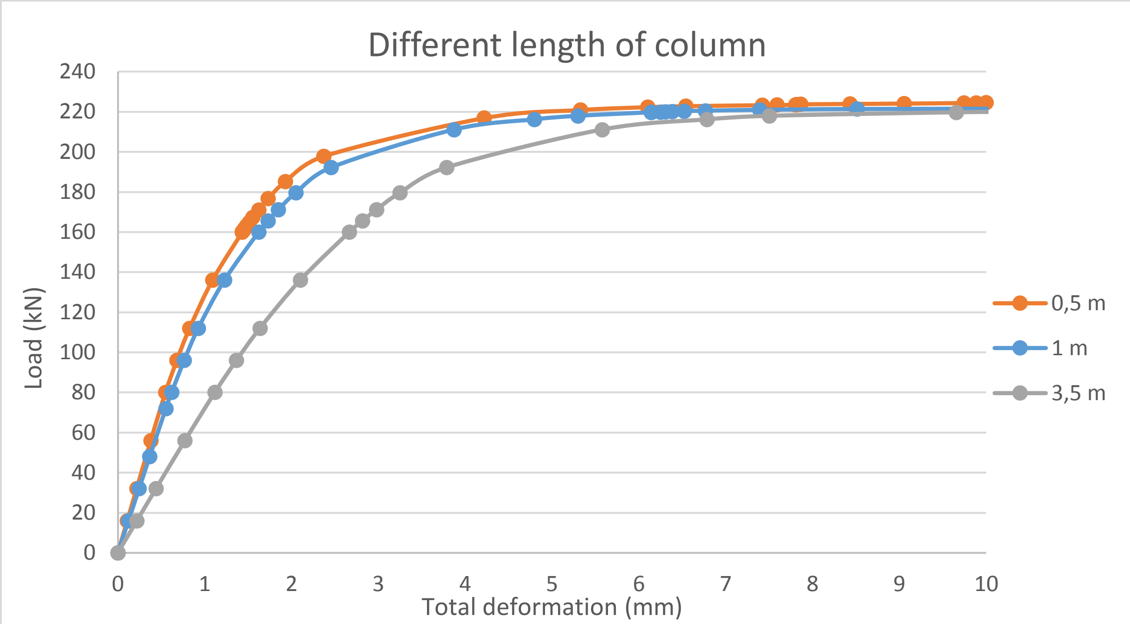


Figure 61: Von Mises stress qualitative pictures. ANSYS. Mesh 10 mm. View results left ones in: 21 (0,5xAuto results) and right one in: 1.0 (True Scale)

When you are developing or studding a model by a FEM, you cannot define just with one simulation. The solutions depends on different parameters, and because of that, must be compared different resolutions.

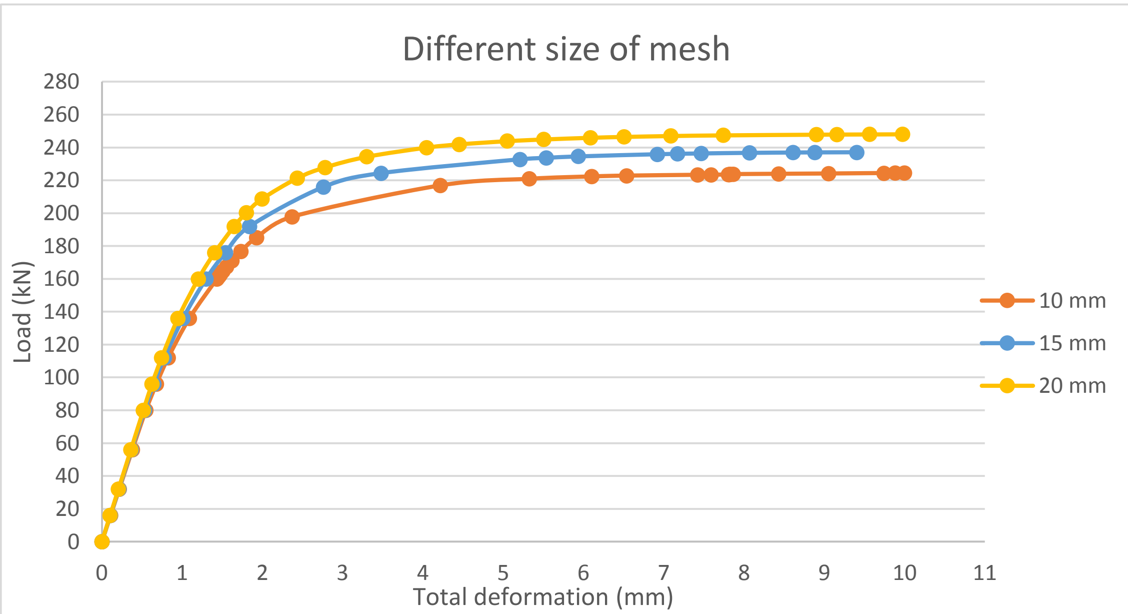
a) Depending on the length of the column: one of the facts that can change the resistance of the joint could be the length of the column, because of that, it was decided to study the column with different sizes:



Graph 8: Resolution with different length of steel column joint.

As we can see, the length of the column is not something that creates huge variations on the results because of that it was decided to work with a 1 m column.

b) Depending on the size of mesh: always with some restrictions, talked in section 3.3.2.4 Finite Element Mesh, the size of the mesh could be different:



Graph 9: Resolution with different size of mesh.

3.3.2.6.2 Composite joint

If we evaluate the qualitative results, we can see where the joint deflects and resist more:

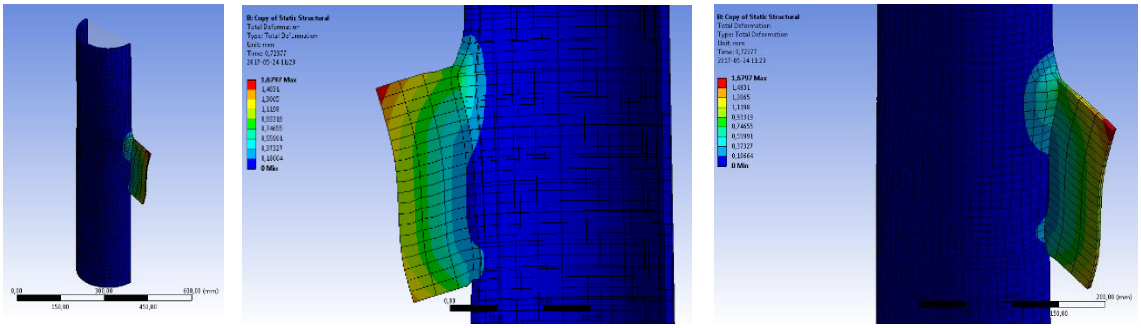


Figure 65: Deformation of composite column joint. ANSYS. Mesh 30 mm. View in: 21 (0,5xAuto results)

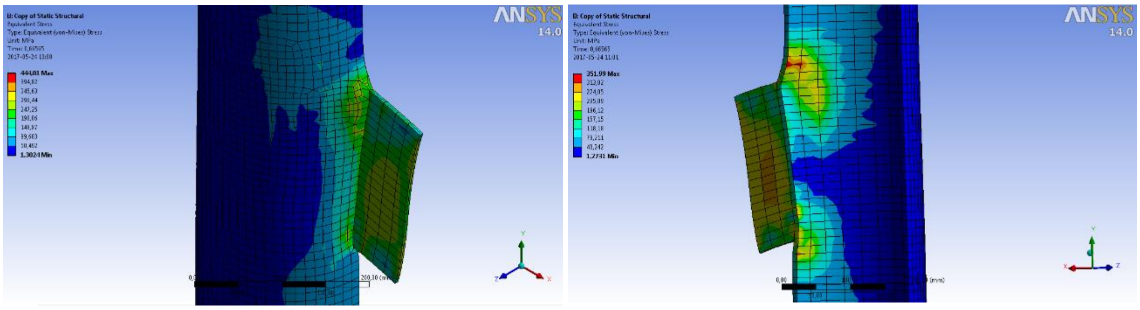


Figure 64: Von misses stress. Steel element of the composite joint. View in: 21 (0,5xAuto results)

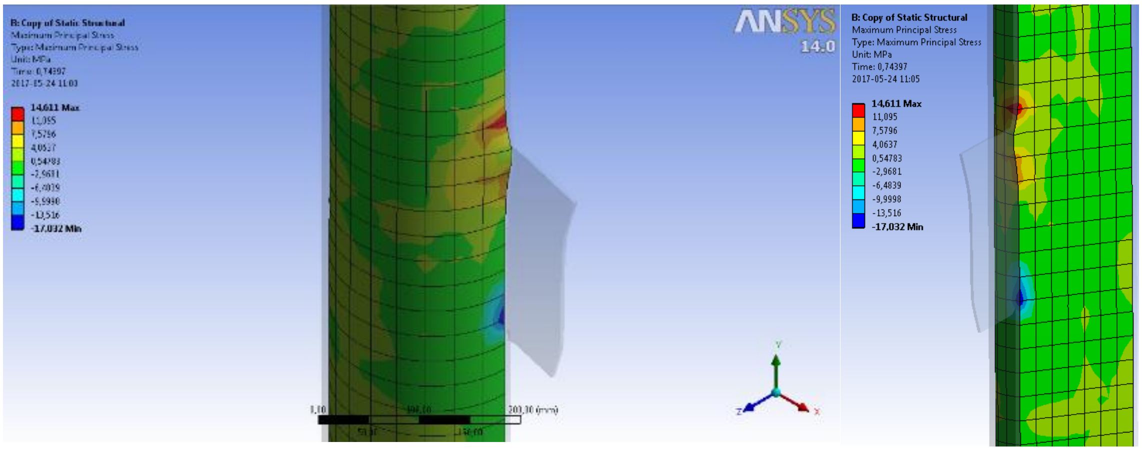


Figure 63: Maximum principal stress. Concrete element on composite joint. View in: 21 (0,5xAuto results).

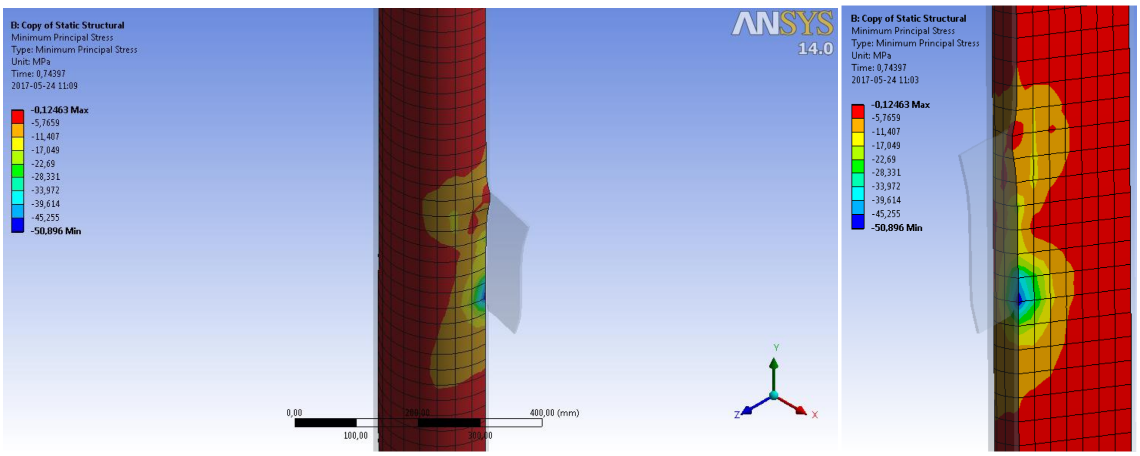
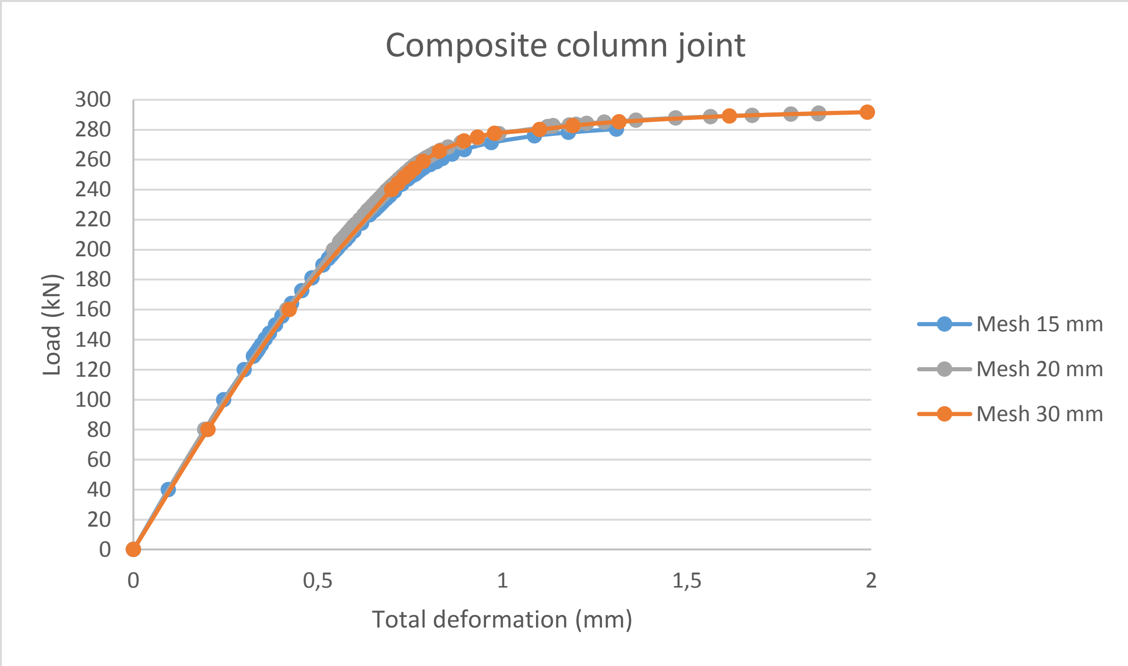


Figure 62: Minimum principal stress. Concrete element on composite joint. View in: 21 (0,5xAuto results).

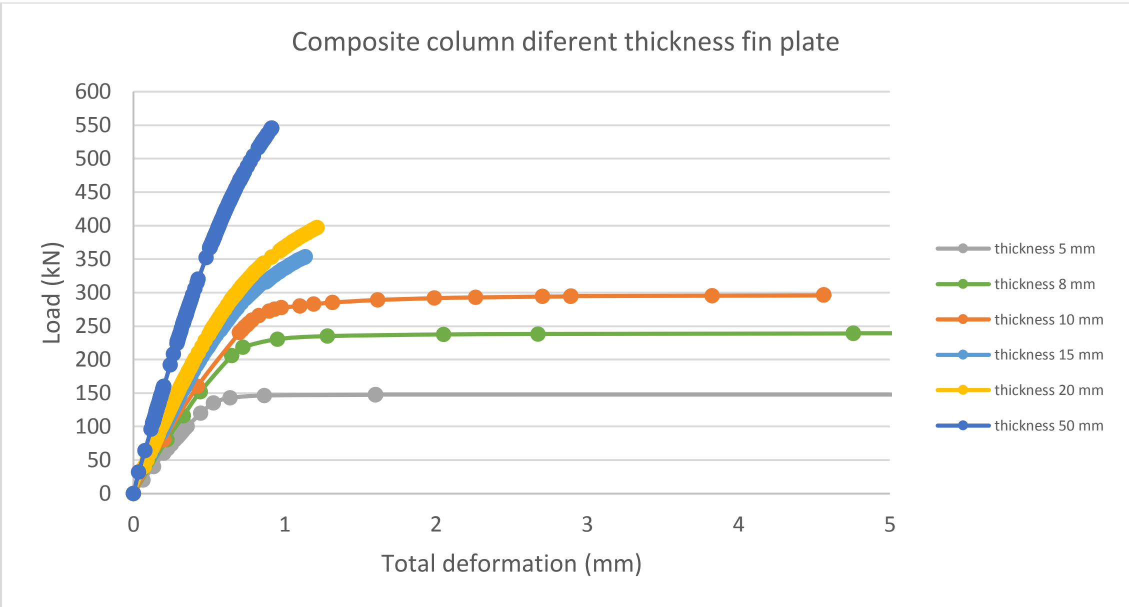
For the steel composite column, we have more information and it was quite accurate. For composite column, as we said on the theoretical part of the thesis the solutions are not very sure. Because of that, we will change some parameters to see how affects the different dimensions of the column. For example:

- Different size of mesh



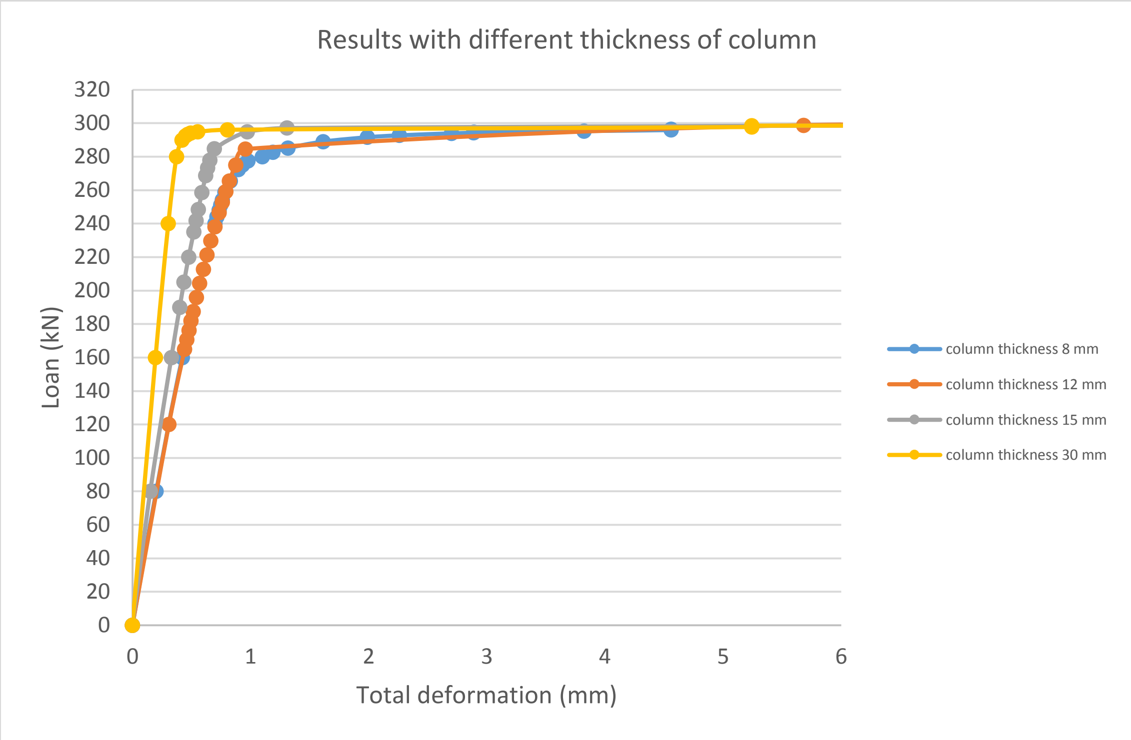
Graph 10: Results for load-deformation of composite column joint. Results with different types of mesh

- Different thickness plate:



Graph 11: Results for load-deformation of composite column joint. Results with different fin plate thickness

- Different thickness of the column:



Graph 12: Results for load-deformation of composite column joint. Results with different column thickness

We can see that with different thickness of column, the failure load is very similar; the only part that is altered is the elastic area. That is very interesting because means that the elastic area depends on the thickness of the column, however, the plastic area is not and the joint will fail on the same load. One of the most important conclusion that we can do in with that studies with different thickness of the column is that the joint breaks because of the concrete.

However, the change on the thickness of the plate made the joint more resistance. With a bigger thickness of the plate the joint will resist more. We can explain it with the less concentration of load on the same point of the column. The load is distributed in a higher surface and that means less stress on the column.

4 COMPARISON OF NUMERICAL MODELING AND ANALYTICAL CALCULATIONS RESULTS

The consideration of the parameters is:

- For steel joint: 10 mm mesh because is not too much elements and we can do it as precise as we consider
- For composite joint: 15 mm mesh, we can do it with less but the time with a size of 10 mm increase to much comparing with the difference of the results. We can consider that a mesh of 15 mm is precise.

All the columns will have a 1 m length because as we saw in steel results the solution does not change to mush and a column of 3,5 m is many elements to evaluate.

4.1 Bucking comparison

The comparison between the results are:

		Analytical calculations (kN)	Numerical calculation (kN)	Error (MEF/Analytical)
BUCKING	Steel column	20.032,477	20.037,00	1,0002
	Composite column	24.958,364	24.964,00	1,0002

Table 35: Comparison between numerical and analytical results for both types of joints.

4.2 Joint comparison

To compare the analytical and numerical solutions of the joints is not as easy as the bucking. The bucking analysis is compare between two values; nevertheless, the joint comparison is between a value and a graph.

The value is the calculation that we calculate that the joint will fail, and the graph is the solution that we obtain from the ANSYS.

4.2.1 Steel column

First, choose the one that we considered that will be the resolution:

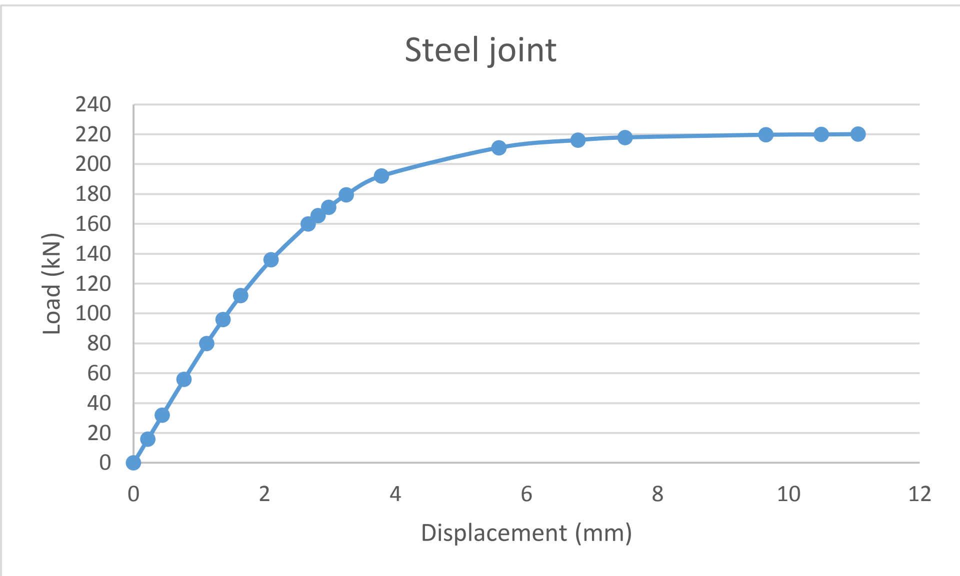


Table 36: Representation of the load-deformation of the steel column joint. Mesh 10 mm, length of column 1 m.

Now we have to consider with is the maximum load that the joint can afford. For this decision, it is need to cross two linear rules: elastic and plastic. Study the equation that follows and evaluate the point where they cross

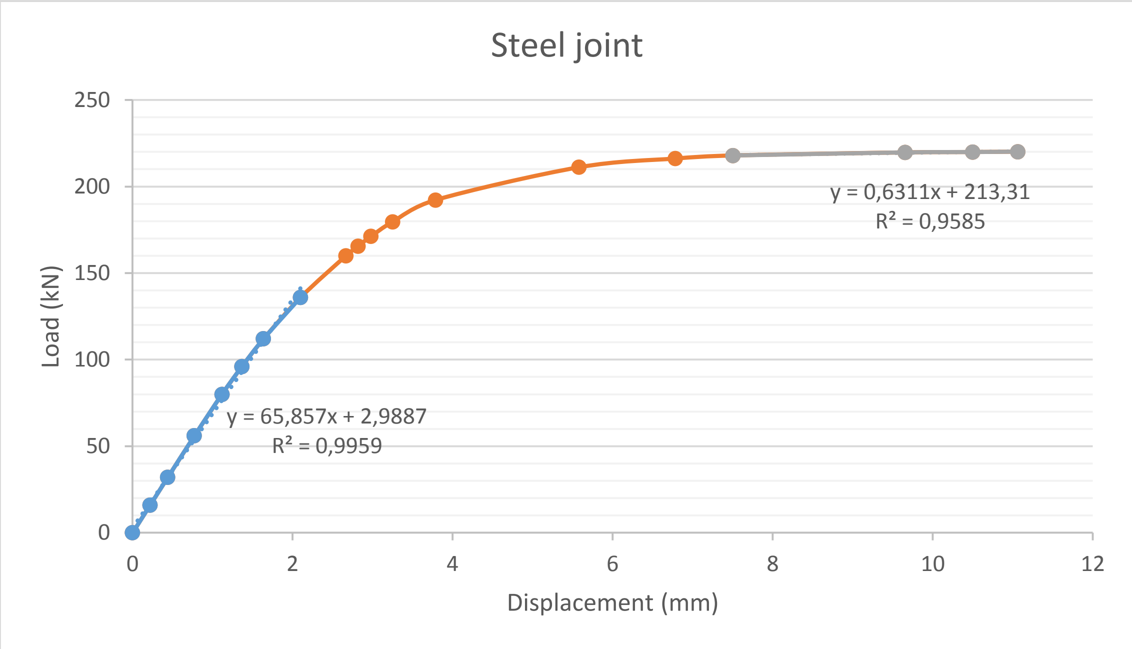
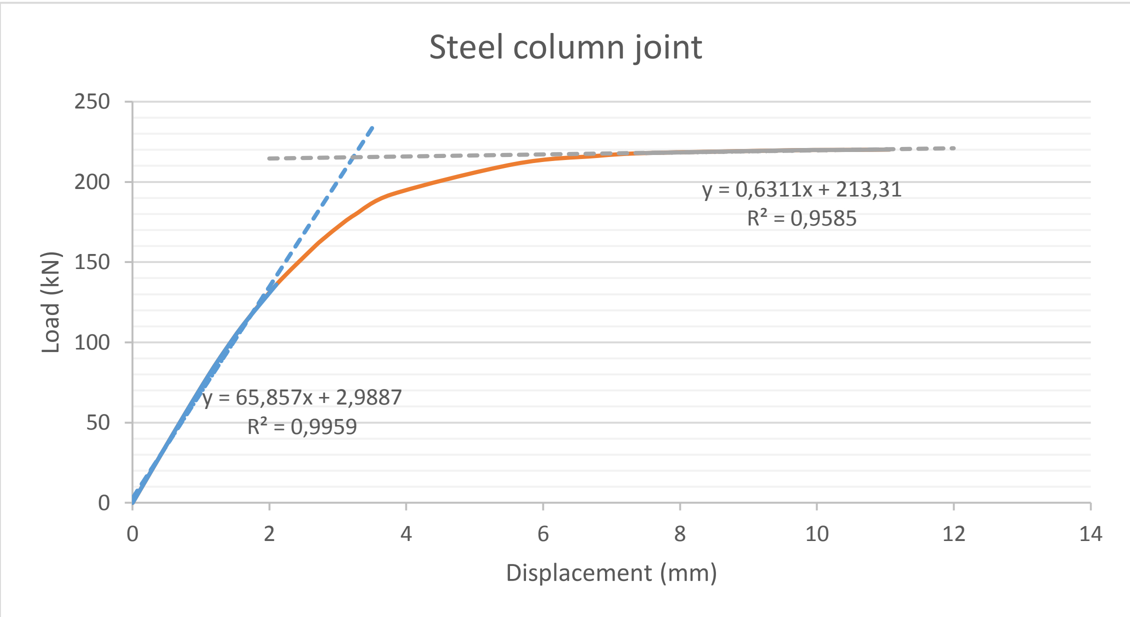


Table 37: Representation of the load-deformation with the elastic and plastic linear function of the steel column joint. Mesh 10 mm, length of column 1 m.



Graph 13: Representation of the load-deformation with the elastic and plastic linear function crossing on 1 point of the steel column joint. Mesh 10 mm, length of column 1 m.

LOAD	215,34kN

DEFORMATION	3,22 mm

	Analytical calc.	Numerical calc.	Error (MEF/Analytical)
Steel column	192,501kN	215,34kN	1,118

Table 38: Comparison between analytical and numerical solutions of steel column joint.

Furthermore, the analytical calculations are done with a security factor that the FEM (ANSYS) does not use, so the value of the analytical calculation without the security factor is:

- Supporting column wall

$$FRd,u = \frac{5 * fy,c * tc^2 * (1 + 0{,}25 * \eta) * 0{,}67}{\cancel{\gamma Mu}}$$

Where η:

$$\eta = \frac{h1}{dc} = \frac{228}{219} = 1{,}041$$

$$FRd,u = \frac{5 \cdot 275 \cdot 8^2 \cdot (1 + 0{,}25 \cdot 1{,}041) \cdot 0{,}67}{\cancel{1{,}1}}$$

$$FRd,u = 75{,}115\ kN$$

That is the horizontal load, if we want to know the higher shear load:

$$Mf = FRd,u \cdot h$$

$$Mf = F \cdot e$$

$$F = \frac{FRd,u \cdot h}{e} = \frac{75{,}115 \cdot 228}{80}$$

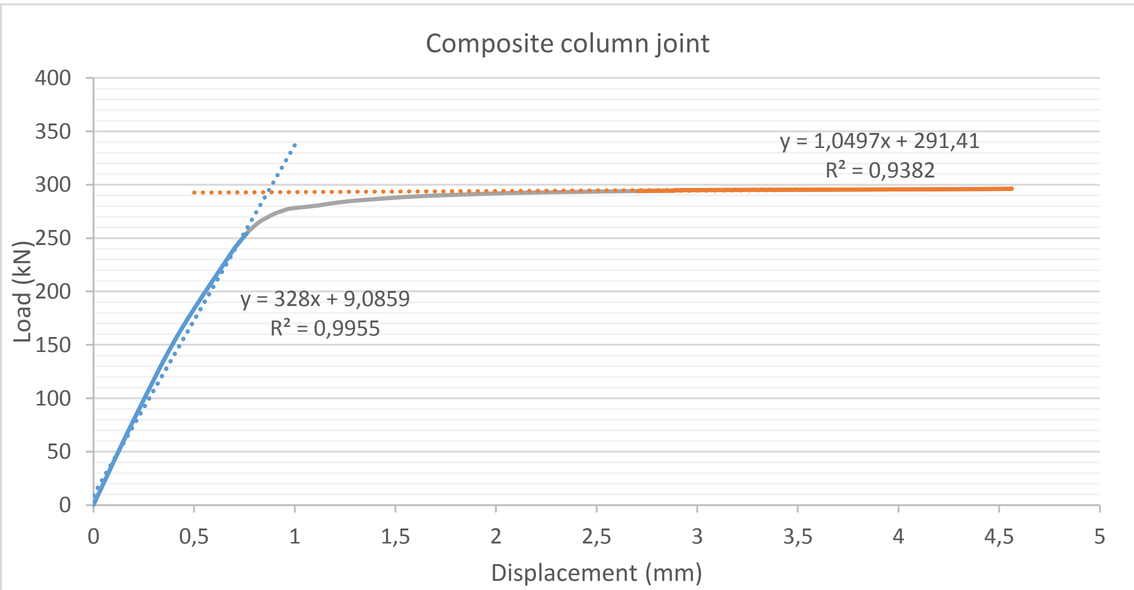
$$F = 214{,}08\ kN$$

	Analytical calculations (kN)	Numerical calculation (kN)	Error (MEF/Analytical)	Error
Steel column	214,08	215,34	1,00589	1,697 %

Table 39: Comparison between analytical and numerical solutions of steel column joint. Without security factor

4.2.2 Composite column

The same happens than with the steel column joint, first we have to choose which is the graph that represents better the specific case. On that moment we decided that the most accurate results are: mesh 30 mm, 1 meter length of the column, fin plate 10 mm and thickness of the column 8 m.



Graph 14: Representation of the load-deformation curve for the composite column joint. Mesh 15 mm. With the plastic and elastic linear functions crossing in 1 point.

So, the final failure load it is considered:

LOAD	292,32 kN

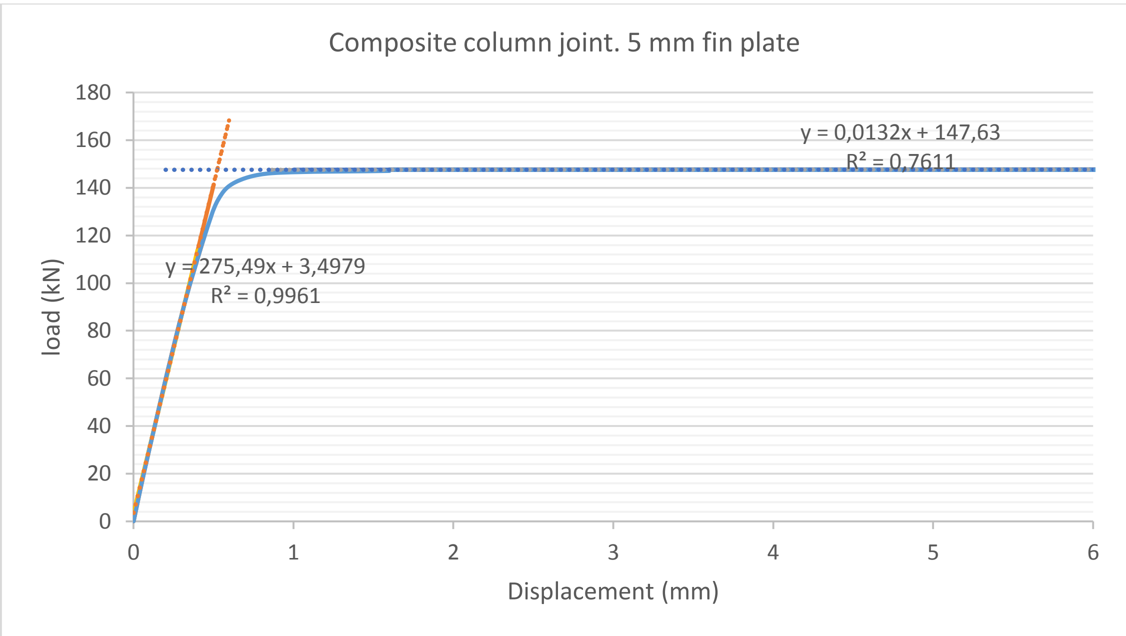
DEFORMATION	0,86 mm

We cannot compare with any hand value because there is any norm that told us which is the theoretical value, but we can compare it with the value that simulates the calculation between a steel column and a concrete plate.

Analytical calc.	Numerical calc.	Error (MEF/Analytical)	Error
308,265 kN	292,32 kN	0,9482	5,4546 %

Furthermore, to know if this hand calculations really works we analyses the different types of fin plates thickness.

- 5 mm fin plate:



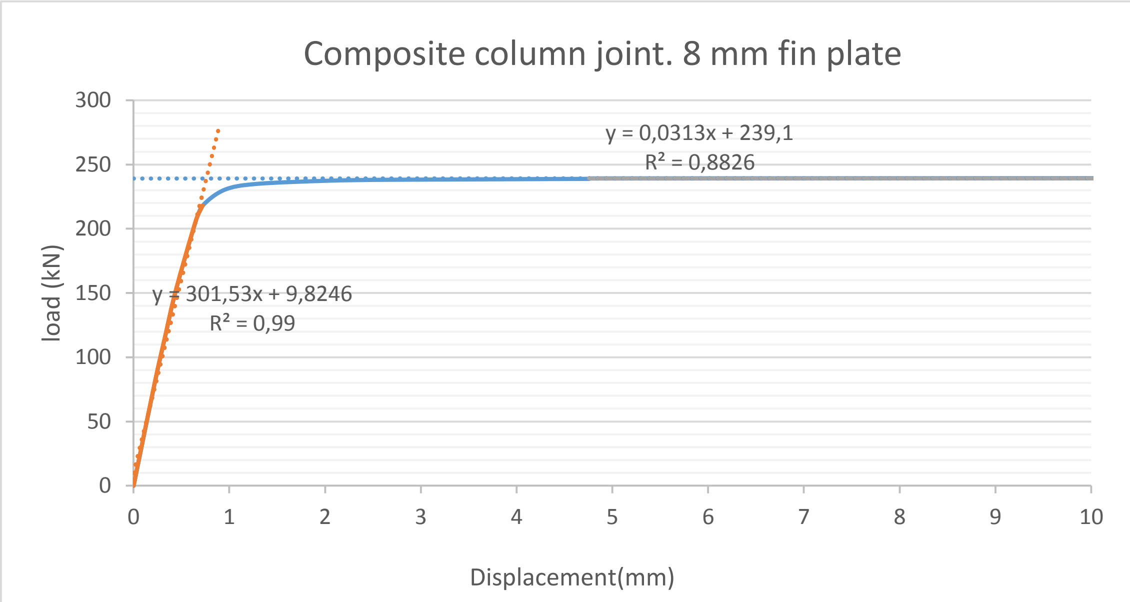
Graph 15: load-deformation of the composite column joint. 5 mm fin plate

LOAD	147,6369 kN

DEFORMATION	0,52321 mm

Analytical calc.	Numerical calc.	Error (MEF/Analytical)	Error
139,536 kN	147,6369 kN	1,0580	5,487 %

- 8 mm fin plate:



Graph 16: load-deformation of the composite column joint. 8 mm fin plate

LOAD	239,1714 kN

DEFORMATION	0,760452 mm

Analytical calc.	Numerical calc.	Error (MEF/Analytical)	Error
237,198 kN	239,1714 kN	1,008	0,825 %

In addition, we have to remember that does not affect with the column thickness, and as we saw on the Graph 12: Results for load-deformation of composite column joint. Results with different column thickness) the thickness of the column does not affects on the joint plasticity, just on the elasticity and the makes that they have a similar fail load.

5 COMPARISON OF STEEL COLUMN AND CONCRETE COLUMN

One of the most important goal of the thesis is know how the concrete column joint will react in comparison with the steel column.

		Maximum load (kN)	Comp/Steel
BUCKING	Steel column	20.032,477	1,25
	Composite column	24.958,364	

Table 40: Bucking summary results

		Maximum load (kN)	Percentage
JOINT	Steel column	215,3450	35,74%
	Composite column	292,3164	
		Deformation (mm)	Percentage
	Steel column	3,22	73,29%
	Composite column	0,86	

Table 41: Joint results summary

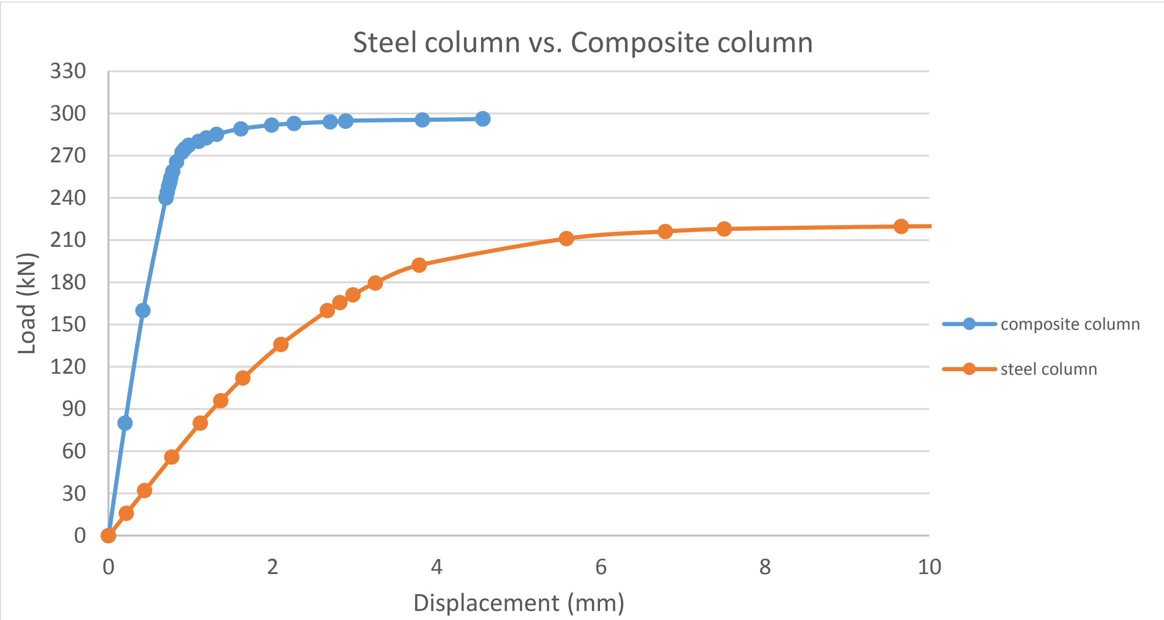


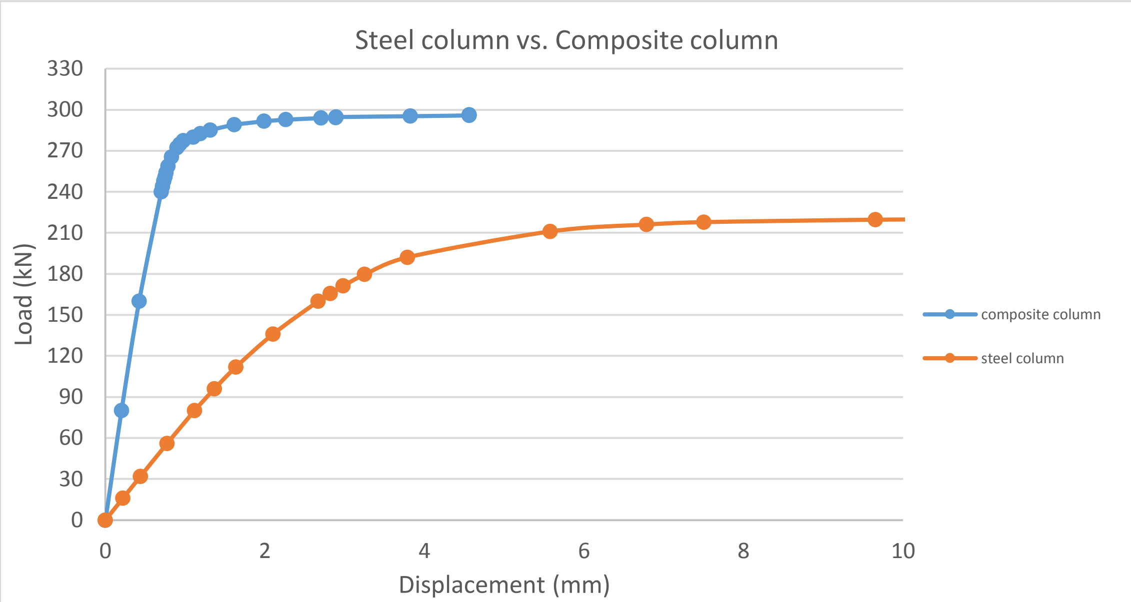
Table 42: Steel column joint compared to composite column joint

We can see that the composite column supports 3,22 times more than the steel column, otherwise, the steel column is more flexible and that is sometimes an advantage.

6 CONCLUSIONS AND PROPOSALS

From my point of view, there are two principal conclusions that are the most important ones:

1) On a joint with a steel tubular column, the joint will resist three times if it is filled of concrete.



Graph 17: steel column vs. composite column.

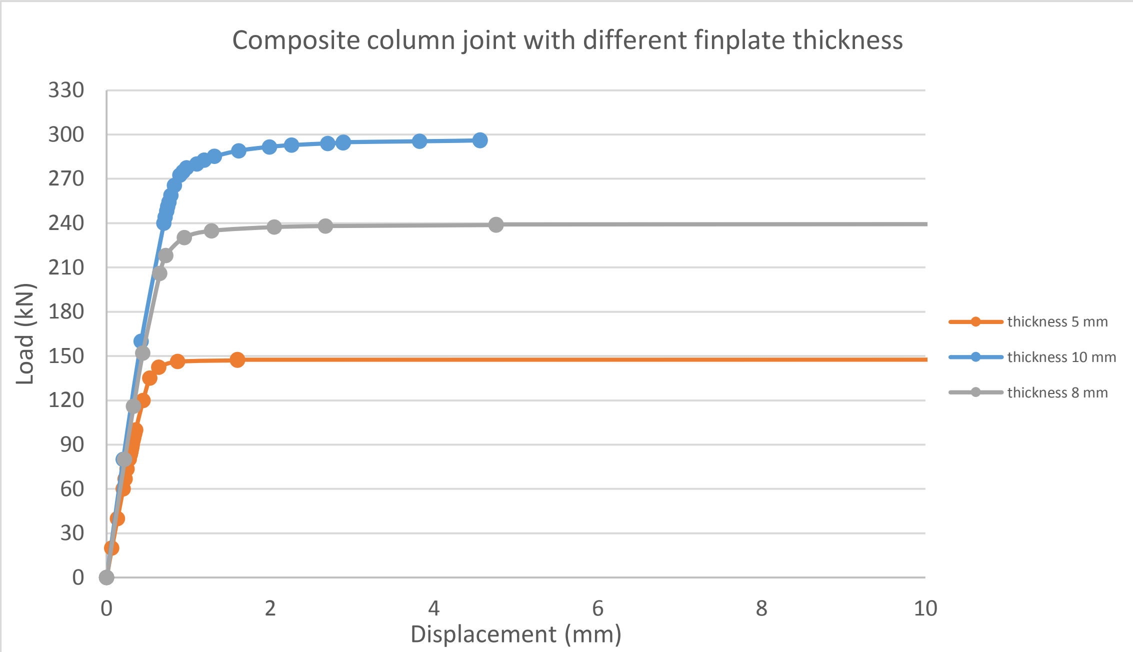
For me this is one of the most representative graphs on this thesis. One of the aim goals was knows how the composite column on the same situation than a steel one reacts. Here we can see that it is much more resistance, moreover, we can see that the joint is much more rigid too.

On the Design Guide, as we saw on the theoretical part of the thesis, they inform to us: "the stiffness is about twice that of the unfilled counterparts" [10] on that case we can see that the stiffness of the composite column is bigger than the double, it is 3,74 times stiffness than the steel column.

	Deformation (mm)
Steel column	3,22
Composite column	0,86

For the load they do not give an imposition of any load or study, just commend to us that exists confidential results on studies that are not available or have been published with insufficient information for a full interpretation, eg. the EC Composite Jacket Project, Tebbett et al. (1979 and 1982). [10].
One of the recommendations nowadays is: have the beam less resistance than the joint and you can control your joint.

2) Could be possible that the composite column follows the same rule that the steel columns joint on a concrete plate. We can found the regulation in Eurocode 3 to calculate: “6.2.5 Equivalent T-stub in compression.”



Graph 18: Composite column joint with different fin plate thickness

	Analytical calc.	Numerical calc.	Error (MEF/Analytical)	Error
5 mm fin plate	139,536 kN	147,6369 kN	1,0580	5,487 %
8 mm fin plate	237,198 kN	239,1714 kN	1,008	0,825 %
10 mm fin plate	308,265 kN	292,32 kN	0,9482	5,4546 %

Of course for being sure that is true it must need an experimental analysis and more data. Otherwise could be a first calculation to have an orientation of how the maximum load will be.

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