Saimaa University of Applied Sciences
Technology Lappeenranta
Civil and Construction Engineering

Lidia Sergeeva

WQ-BEAM AND COLUMN CONNECTION

Bachelor’s Thesis 2012

ABSTRACT
Lidia Sergeeva
AKO-Standards (WQ-BEAM AND COLUMN CONNECTION)
71 pages, 3 appendices
Saimaa University of Applied Sciences, Lappeenranta
Civil and Construction Engineering
Structural Design
Thesis, 2012
Instructors: Senior lecturer Petri Himmi, Saimaa University of Applied Sciences
Master of Science in engineering Fia Inkala, Aaro Kohonen Oy

The thesis examined the connection between WQ-beam for hollow core slabs and hollow section column. The purpose of this thesis was to simplify the engineering, to study the connection and to do the design.

This thesis was commissioned by Aaro Kohonen Oy. The company needs a simplified calculation sheet and a standard card for the WQ-beam connection following the standards by Eurocodes for every engineer to use and find easy information about the connection. In addition, a dimensioning tool with Excel for the connection was made.

The thesis examined the most common hollow section steel structures in general and the joint types. The work focused on making the welding between end plate and WQ-beam, and between column and consol. The welds and the shear force in the plate for the fire case were examined. In every examination the standards according to Eurocode have been used.

Keywords: Eurocode, WQ-beam, hollow core slab.

CONTENTS

SYMBOLS

b	width of a cross section
h	depth of a cross section
t	thickness
b_p	width of a plate
t_p	thickness of a plate
γ_M	general partial factor
γ_{Mi}	particular partial factor
f_y	yield strength
f_u	ultimate strength
F_{Ed}	design loading on the structure
σ	stress
ψ	stress or strain ratio
γ_{M0}	partial factor for resistance of cross-sections whatever the class is
γ_{M1}	partial factor for resistance of members to instability assessed by member checks
γ_{M2}	partial factor for resistance of cross-sections in tension to fracture
$\sigma_{x,Ed}$	design value of the local longitudinal stress
$\sigma_{z,Ed}$	design value of the local transverse stress
τ_{Ed}	design value of the local shear stress
N_{Ed}	design normal force
$M_{y,Ed}$	design bending moment, y-y axis
$M_{z,Ed}$	design bending moment, z-z axis
N_{Rd}	design values of the resistance to normal forces
$M_{y,Rd}$	design values of the resistance to bending moments, y-y axis
$M_{z,Rd}$	design values of the resistance to bending moments, z-z axis
V_{Ed}	design shear force
A_v	shear area
$\sigma_\perp$	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
A_w	design throat area
β_w	appropriate correlation factor
$F_{w,Ed}$	design value of the weld force per unit length
$F_{w,Rd}$	design weld resistance per unit length
$f_{vw.d}$	design shear strength of the weld
CHS	for “circular hollow section”
RHS	for “rectangular hollow section”

1 INTRODUCTION TO EUROCODES

The Eurocodes (European technical standards) are the Europe-wide means for the structural design of buildings and other civil and engineering works. (http://ec.europa.eu/enterprise/sectors/construction/eurocodes/index_en.htm)

The Eurocodes will form a set of 56 European standards providing calculation methods to determine the efforts applied to each element which plays a structural role in structures submitted to given actions and to check if the mechanical strength of each element is sufficient to resist to this effort. These calculation methods will be used:

• to design buildings and civil engineering works regardless the type of works or materials used (concrete structures, steel, composite steel/concrete, masonry, timber, aluminium). It also contains specific calculation rules for geotechnical works and rules on earthquake resistance;

• to check the stability and mechanical resistance of structures, including structures submitted to fire;

• to define the dimensions of structural elements;

• to specify the required performance and durability of the product to be incorporated into the structure. (http://www.irccbuildingregulations.org/pdf/4-04.pdf)

Eurocodes serve as reference documents for the following purposes :

– as a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential equirement N°1 – Mechanical resistance and stability – and Essential Requirement N°2 – Safety in case of fire;

– as a basis for specifying contracts for construction works and related engineering services;

– as a framework for drawing up harmonized technical specifications for construction products (ENs and ETAs).

The Structural Eurocode programme comprises the following standards generally consisting of a number of Parts:

EN 1990 Eurocode: Basis of structural design
EN 1991 Eurocode 1: Actions on structures
EN 1992 Eurocode 2: Design of concrete structures
EN 1993 Eurocode 3: Design of steel structures
EN 1994 Eurocode 4: Design of composite steel and concrete structures
EN 1995 Eurocode 5: Design of timber structures
EN 1996 Eurocode 6: Design of masonry structures
EN 1997 Eurocode 7: Geotechnical design
EN 1998 Eurocode 8: Design of structures for earthquake resistance
EN 1999 Eurocode 9: Design of aluminium structures

Eurocode standards recognize the responsibility of regulatory authorities in each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level where these continue to vary from State to State.

The Eurocode standards provide common structural design rules for everyday use for the design of whole structures and component products of both a traditional and an innovative nature. Unusual forms of construction or design conditions are not specifically covered and additional expert consideration will be required by the designer in such cases. (EN 1993)

The National Annexes allow each member state to take into account its own local differences concerning geography, climate and traditional building practices. The safety level however remains the responsibility of the government of each member state and differs from state to state.

Whenever the EN Eurocodes are used for a structure, the National Annex of the state in which the structure is built has to be used. (http://www.eurocodes-online.com/index.php/en_US/en/about-the-eurocodes/national-annexes)

The designing of steel structures in this project needs:

1. The Euorocode 1993-1-3 for general rules and rules for building.
It consists of basis of design, basic requirements; material properties, requirements for it; connecting devices; resistance of cross-sections: tension, compression, bending moment, etc.

2. The Eurocode 1993-1-8 for the design of joints: applied forces and moments, resistance of joints; connections made with bolts; welded connections; hollow section joints.

3. National Annex.
This standard gives alternative procedures, values and recommendations with notes indicating where national choices may have to be made. The National Standard implementing EN 1993-1-8 and EN 1993-1-3 should have a National Annex containing all Nationally Determined Parameters for the design of steel structures to be constructed in the relevant country. (EN 1993)

2 BASICS OF EUOROCODE 1993

2.1 Design working life, durability and robustness

Depending upon the type of action affecting durability and the design working life (see EN 1990) steel structures should be

- designed against corrosion by means of:
  - suitable surface protection
  - the use of weathering steel
  - the use of stainless steel
- detailed for sufficient fatigue life
- designed for wearing
- designed for accidental actions
- inspected and maintained.

The design working life should be taken as the period for which a building structure is expected to be used for its intended purpose.

To ensure durability, buildings and their components should either be designed for environmental actions and fatigue if relevant or else protected from them.

The effects of deterioration of material, corrosion or fatigue where relevant should be taken into account by appropriate choice of material or by structural redundancy and by the choice of an appropriate corrosion protection system. (EN 1993-1-1: 2005)

2.2 Material properties

The nominal values of the yield strength f_y and the ultimate strength f_u for structural steel should be obtained:
a) either by adopting the values $f_y = R_{eh}$ and $f_u = R_m$ direct from the product standard
b) or by using the simplification given in *Table 1* and *Table 2.* (EN 1993-1-1: 2005 (E))

2.3 Ductility requirements

For steels a minimum ductility is required that should be expressed in terms of limits for:

- the ratio f_u / f_y of the specified minimum ultimate tensile strength f_u to the specified minimum yield strength f_y;
- the elongation at failure on a gauge length of $5{,}65(A_0)^{1/2}$ (where A_0 is the original cross-sectional area);
- the ultimate strain ε_u, where ε_u corresponds to the ultimate strength f_u.

NOTE The limiting values of the ratio f_u / f_y , the elongation at failure and the ultimate strain εu may be defined in the National Annex. The following values are recommended:

- $f_u / f_y \geq 1{,}10$;
- elongation at failure not less than 15%;

– $\varepsilon_u \geq 15\varepsilon_y$, where ε_y is the yield strain ($\varepsilon_y = f_y / E$). (EN 1993-1-3)

2.4 Fracture toughness

The material should have sufficient fracture toughness to avoid brittle fracture of tension elements at the lowest service temperature expected to occur within the intended design life of the structure.

NOTE The lowest service temperature to be adopted in design may be given in the National Annex. (EN 1993-1-3)

For building components under compression a minimum toughness property should be selected. (EN 1993-1-1: 2005 (E))

NOTE The National Annex may give information on the selection of toughness properties for members in compression. (EN 1993-1-1: 2005 (E))

Table 1. Nominal values of yield strength f_y and ultimate tensile strength f_u for hot rolled structural steel (EN 1993-1-3)

Standard and steel grade	Nominal thickness of the element t [mm]			
	$t \leq 40$ mm		40 mm $< t \leq 80$ mm	
	f_y [N/mm^2]	f_u [N/mm^2]	f_y [N/mm^2]	f_u [N/mm^2]
EN 10025-2				
S 235	235	360	215	360
S 275	275	430	255	410
S 355	355	510	335	470
S 450	440	550	410	550
EN 10025-3				
S 275 N/NL	275	390	255	370
S 355 N/NL	355	490	335	470
S 420 N/NL	420	520	390	520
S 460 N/NL	460	540	430	540
EN 10025-4				
S 275 M/ML	275	370	255	360
S 355 M/ML	355	470	335	450
S 420 M/ML	420	520	390	500
S 460 M/ML	460	540	430	530
EN 10025-5				
S 235 W	235	360	215	340
S 355 W	355	510	335	490
EN 10025-6				
S 460 Q/QL/QL1	460	570	440	550

Table 2. Nominal values of yield strength f_y and ultimate tensile strength f_u for structural hollow sections (EN 1993-1-3)

Standard and steel grade	Nominal thickness of the element t [mm]			
	$t \leq 40$ mm		40 mm $< t \leq 80$ mm	
	f_y [N/mm^2]	f_u [N/mm^2]	f_y [N/mm^2]	f_u [N/mm^2]
EN 10210-1				
S 235 H	235	360	215	340
S 275 H	275	430	255	410
S 355 H	355	510	335	490
S 275 NH/NLH	275	390	255	370
S 355 NH/NLH	355	490	335	470
S 420 NH/NHL	420	540	390	520
S 460 NH/NLH	460	560	430	550
EN 10219-1				
S 235 H	235	360		
S 275 H	275	430		
S 355 H	355	510		
S 275 NH/NLH	275	370		
S 355 NH/NLH	355	470		
S 460 NH/NLH	460	550		
S 275 MH/MLH	275	360		
S 355 MH/MLH	355	470		
S 420 MH/MLH	420	500		
S 460 MH/MLH	460	530		

2.5 Design values of material coefficients

The material coefficients to be adopted in calculations for the structural steels covered by this Eurocode Part should be taken as follows (EN 1993-1-3):

– Modulus of elasticity $E = 210000 N/mm^2$

– Shear modulus $G = \dfrac{E}{2(1+\nu)} \approx 81000 N/mm^2$

– Poisson's ratio in elastic stage $\nu = 0{,}3$

– Coefficient of linear thermal expansion $\alpha = 12 \times 10^{-6} \, perK$ (for $T \leq 100^\circ C$).

2.6 Cross-section requirements for plastic global analysis

At plastic hinge locations, the cross-section of the member which contains the plastic hinge should have a rotation capacity of not less than the required at the plastic hinge location. (EN 1993-1-3)

In a uniform member sufficient rotation capacity may be assumed at a plastic hinge if both the following requirements are satisfied:
a) the member has *Class 1* cross-sections at the plastic hinge location;
b) where a transverse force that exceeds *10 %* of the shear resistance of the cross section is applied to the web at the plastic hinge location, web stiffeners should be provided within a distance along the member of *h/2* from the plastic hinge location, where h is the height of the cross section at this location. (EN 1993-1-3)

Where the cross-sections of the member vary along their length, the following additional criteria should be satisfied:

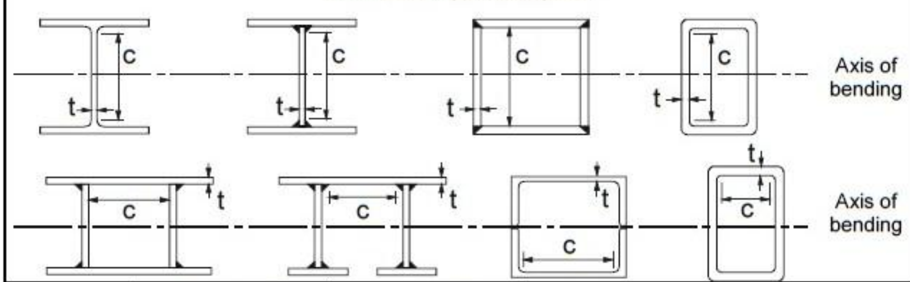
a) Adjacent to plastic hinge locations, the thickness of the web should not be reduced for a distance each way along the member from the plastic hinge location of at least *2d*, where d is the clear depth of the web at the plastic hinge location.
b) Adjacent to plastic hinge locations, the compression flange should be Class 1 for a distance each way along the member from the plastic hinge location of not less than the greater of:

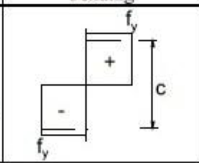
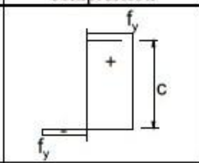
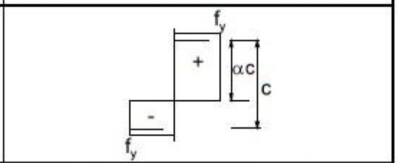
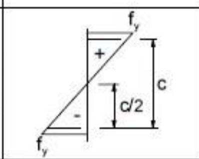
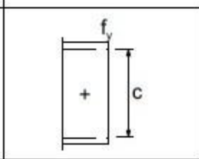
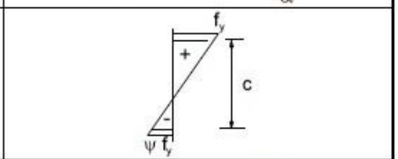
- *2d*, where d is as defined in a)
- the distance to the adjacent point at which the moment in the member has fallen to *0,8* times the plastic moment resistance at the point concerned.

c) Elsewhere in the member the compression flange should be *class 1* or *class 2* and the web should be *class1*, *class 2* or *class 3*. (EN 1993-1-3)

Maximum width-to-thickness ratios for compression parts. (EN 1993-1-3)

Internal compression parts



Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \le 72\varepsilon$	$c/t \le 33\varepsilon$	when $\alpha > 0,5: \ c/t \le \dfrac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0,5: \ c/t \le \dfrac{36\varepsilon}{\alpha}$
2	$c/t \le 83\varepsilon$	$c/t \le 38\varepsilon$	when $\alpha > 0,5: \ c/t \le \dfrac{456\varepsilon}{13\alpha - 1}$ when $\alpha \le 0,5: \ c/t \le \dfrac{41,5\varepsilon}{\alpha}$
Stress distribution in parts (compression positive)			
3	$c/t \le 124\varepsilon$	$c/t \le 42\varepsilon$	when $\psi > -1: \ c/t \le \dfrac{42\varepsilon}{0,67 + 0,33\psi}$ when $\psi \le -1^{*)}: \ c/t \le 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

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

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

2.7 Tension

The design value of the tension force N_{Ed} at each cross section should satisfy (EN 1993-1-3):

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1{,}0 \text{ , (1)}$$

For sections with holes the design tension resistance $N_{t,Rd}$ should be taken as the smaller of:

- the design plastic resistance of the gross cross-section (EN 1993-1-3):

$$N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{Mo}} \text{ , (2)}$$

- the design ultimate resistance of the net cross-section at holes for fasteners (EN 1993-1-3):

$$N_{u,Rd} = \frac{0{,}9 A_{net} \cdot f_u}{\gamma_{M2}} \text{ , (3)}$$

2.8 Compression

The design value of the compression force N_{Ed} at each cross-section should satisfy (EN 1993-1-3):

$$\frac{N_{Ed}}{N_{c,Rd}} \leq 1{,}0 \text{ , (4)}$$

The design resistance of the cross-section for uniform compression N should be determined as follows (EN 1993-1-3):

$$N_{c,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} \text{ , for class 1, 2 or 3 cross-sections, (5)}$$

$$N_{c,Rd} = \frac{A_{eff} \cdot f_y}{\gamma_{M0}} \text{ , for class 4 cross-sections, (6)}$$

For the design buckling resistance of a compression member should be taken as:

$$N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} \text{ , for class 1, 2 or 3 cross-sections, (7)}$$

$$N_{b,Rd} = \frac{\chi \cdot A_{eff} \cdot f_y}{\gamma_{M1}}$$, for class 4 cross-sections, (8)

where χ is the reduction factor for the relevant buckling mode.

The calculation of the value χ see EN 1993-1-3, 6.3.1.2.

2.9 Bending moment

The design moment resistance of any joint may be derived from the distribution of internal forces within that joint and the design resistances of its basic components to these forces. (EN 1993-1-3)

In a beam-to-column joint or beam splice in which a plastic hinge is required to form and rotate under any relevant load case, the welds should be designed to resist the effects of a moment at least equal to the smaller of (EN 1993-1-3):

- the design plastic moment resistance of the connected member $M_{pl,Rd}$
- *α* times the design moment resistance of the joint $M_{j,Rd}$

where:
α = 1,4 - for frames with respect to sway;
α = 1,7 - for all other cases.

The design value of the bending moment M_{Ed} at each cross-section should satisfy (EN 1993-1-3):

$$\frac{M_{Ed}}{M_{c,Rd}} \leq 1{,}0$$, (9)

where $M_{c,Rd}$ is determined considering fastener holes.

The design resistance for bending about one principal axis of a cross-section is determined as follows (EN 1993-1-3):

$$M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl} \cdot f_y}{\gamma_{M0}}$$, for *class 1 or 2* cross sections, (10)

$$M_{c,Rd} = M_{el,Rd} = \frac{W_{el,\min} \cdot f_y}{\gamma_{M0}}$$, for *class 3* cross sections, (11)

$$M_{c,Rd} = \frac{W_{eff,\min} \cdot f_y}{\gamma_{M0}}$$, for *class 4* cross sections, (12)

Fastener holes in the tension flange may be ignored provided that for the tension flange (EN 1993-1-3):

$$\frac{A_{f,net} \cdot 0{,}9 f_u}{\gamma_{M2}} \geq \frac{A_f \cdot f_y}{\gamma_{M0}}, \text{ (13)}$$

where A_f is the area of the tension flange.

2.10 Shear

The design value of the shear force V_{Ed} at each cross section should satisfy (EN 1993-1-3):

$$\frac{V_{Ed}}{V_{c,Rd}} \leq 1{,}0, \text{ (14)}$$

where $V_{c,Rd}$ is the design shear resistance.

For plastic design $V_{c,Rd}$ is the design plastic shear resistance $V_{pl,Rd}$ as given in (15). For elastic design $V_{c,Rd}$ is the design elastic shear resistance calculated using (16) and (18).

In the absence of torsion the design plastic shear resistance is given by (EN 1993-1-3):

$$V_{pl,Rd} = \frac{A_v (f_y / \sqrt{3})}{\gamma_{M0}}, \text{ (15)}$$

where A_v is the shear area.

For verifying the design elastic shear resistance $V_{c,Rd}$ the following criterion for a critical point of the cross section may be used unless the buckling verification in section 5 of EN 1993-1-5 applies (EN 1993-1-3):

$$\frac{\tau_{Ed}}{f_y / (\sqrt{3}\gamma_{M0})} \leq 1{,}0, \text{ (16)}$$

where τ_{Ed} may be obtained from (EN 1993-1-3):

$$\tau_{Ed} = \frac{V_{Ed} \cdot S}{I \cdot t} \text{ (17)}$$

where V_{Ed} is the design value of the shear force;

S is the first moment of area about the centroidal axis of that portion of the cross-section between the point at which the shear is required and the boundary of the cross-section;

I is second moment of area of the whole cross section;

t is the thickness at the examined point.

For *I*- or *H*-sections the shear stress in the web may be taken as (EN 1993-1-3):

$$\tau_{Ed} = \frac{V_{Ed}}{A_w} \qquad \text{if } \frac{A_f}{A_w} \geq 0{,}6\text{, (18)}$$

where A_f is the area of one flange;
A_w is the area of the web: $A_w = h_w\, t_w$.

3 BEAM-COLUMN CONNECTION

3.1 Steel beams

3.1.1 Different kind of steel beams

Steel beams and components are used in the construction of large structures, and come in a variety of shapes and sizes, each designed for a particular section. Always consider the load-bearing capacity of steel before installing it as a support or building component.

I-Beams

I-beams have a central section and four horizontal flanges so that a cross-section of the beam resembles the letter *I*. They are used as basic structural components in building projects of all kinds, from large garden sheds to 100-story skyscrapers. (http://www.ehow.com/about_5701590_types-steel-beams-connections.html)

I-beams are often used as the way that they work for bending and shear force.

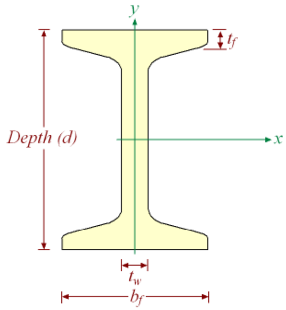


Figure 1. I-Beams
*(*http://www.efunda.com*)*

Wide-Flange Beams

A wide-flange I-Beam has longer flanges than normal *I-beams*, so that the beam in cross-section resembles the letter H.
(http://www.ehow.com/about_5701590_types-steel-beams-connections.html)

H-beams are often used as columns or if there is lateral torsional buckling.

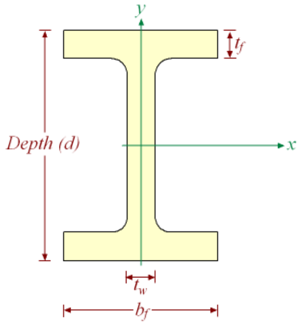


Figure 2. Wide-Flange Beam
*(*http://www.efunda.com*)*

T-Beams

Bar-size T beams or "tees" have a short horizontal flange and long vertical component known as a stem. The beam is shaped like the letter T. The beam is specified by flange width, stem depth, stem thickness and weight. (http://www.ehow.com/about_5701590_types-steel-beams-connections.html)

T-beams are not really used in Finland.

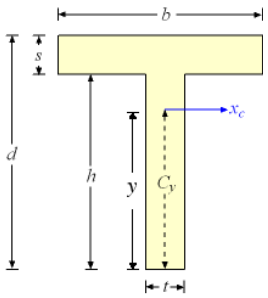


Figure 3. T-Beam shape (http://www.efunda.com)

Channels

Channels are flat, shallow beams in the shape of a U, with a set of parallel short flanges on one side of the beam. They can be used as flat load-bearers attached to a horizontal surface, such as a flat roof or a poured concrete foundation.
(http://www.ehow.com/about_5701590_types-steel-beams-connections.html)

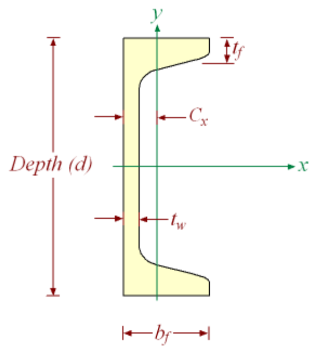


Figure 4. Channel shape beam (http://www.efunda.com)

Flats and Rails

Flats are steel beams without flanges. A rail is a steel beam with a thick, wide base, a short vertical stem and a thick, rounded horizontal section, as in the lengths of railroad track. (http://www.ehow.com/about_5701590_types-steel-beams-connections.html)

RHS-beams

Rectangular Hollow Sections (RHS) have superior structural performance compared to conventional steel sections. RHS are commonly used in welded steel frames where members experience loading in multiple directions. The RHS has superior resistance to lateral torsional buckling.

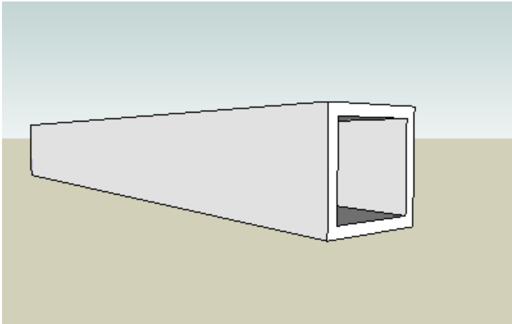


Figure 5. RHS-beam
(http://sketchup.engineeringtoolbox.com/square-hollow-sections-c_47.html)

Welded beams

Welded beams have usually *I-beam* or *RHS-beam* shape. They are welded with flat plates. Welded beam is designed for the use in steel framework structures of residential, civil, farming and industrial buildings.

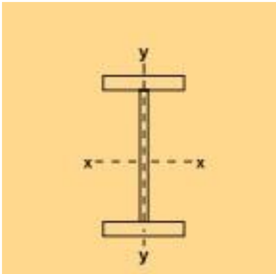


Figure 6. Welded I-beam
(http://units.civil.uwa.edu.au/teaching/CIVL4111?f=275134)

3.1.2 WQ-beam system

The WQ beam is intended to speed up the construction of multi-storey buildings. The WQ-beam system can be used in any multi-storey buildings, such as offices, hospitals, hotels and in residential construction.

The WQ beam is:
- A rapidly installed and torsionally rigid box beam
- Used in intermediate floors and ceilings
- Used either with hollow-core slabs or thin-shell slabs.

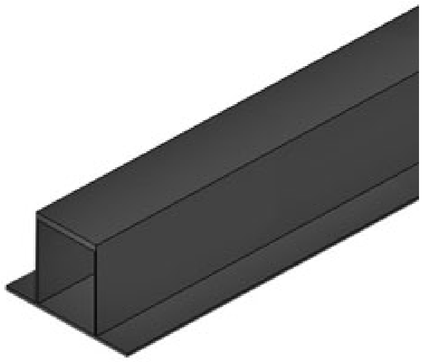


Figure 7. 3D-model of WQ-beam (http://www.ruukki.com/Products-and-solutions/Building-solutions/Steel-frame-structures/WQ-beam-system/WQ-centre-beam#tab1)

The WQ beam does not usually require any support during installation.

The height of the beam is equal to that of the hollow-core slabs, thus maximising the free height of the storey. In the finished structure, only the bottom flange of the beam is visible.

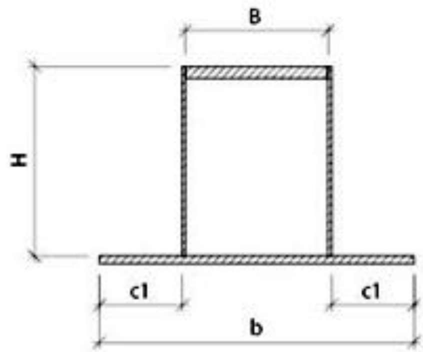


Figure 8. WQ-beam cross section (http://www.ruukki.com/Products-and-solutions/Building-solutions/Steel-frame-structures/WQ-beam-system/WQ-centre-beam#tab1)

After the installation of the hollow-core slabs, the joint castings are done, the screed is cast and the parts exposed to fire are protected against fire. WQ beams can be designed as simple single-span, cantilever and continuous beams.

WQ-beam is installed and locked in the place with the pillar or wall bracket. In Finland, WQ-beams are normally designed with one span and simply supported.

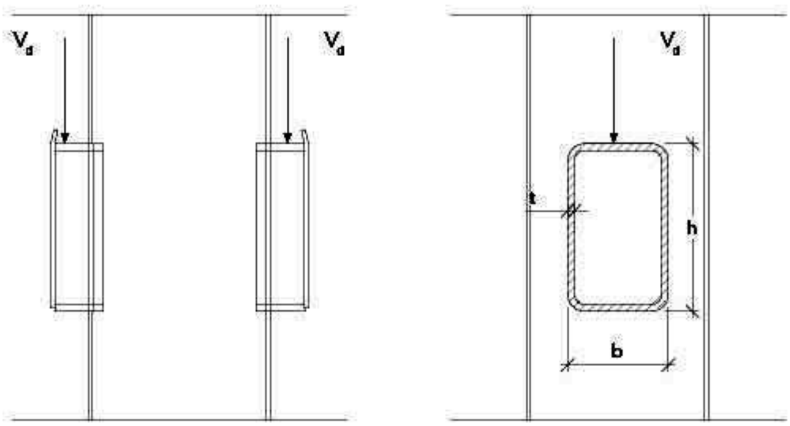


Figure 9. Design of the WQ beam console joint (http://www.ruukki.com/Products-and-solutions/Building-solutions/Steel-frame-structures/WQ-beam-system/WQ-centre-beam#tab1)

Design standard is Eurocode but country's local standards can also be applied. (http://www.ruukki.com/Products-and-solutions/Building-solutions/Steel-frame-structures/WQ-beam-system/WQ-centre-beam#tab1)

In the manufacturing process the plates are blast cleaned, then they are flame cut to profiles of the web and flanges by the measure. After welding the profiles are drilled with needed holes. Steel plates in welded profiles are normally manufactured with strength class S355.

WQ-beam is usually calculated according to the Eurocode standards, but it is also possible to calculate with the Finnish Building Code or local standards. The design takes into account the ultimate limit state, service limit sate, the fire situation and the load combination.

WQ-beams are installed at the site in order with the installation plan and locked in place with the pillar or column consol.

The consol must be designed so that can withstand the full floor installation and operating loads.

3.2 Steel columns

3.2.1 Types of column sections

There are three basic types of steel columns.

1. Rolled steel is usually I-beam, RHS, CHS and other shapes.
2. Built or fabricated columns. This type of columns is not very popular in use in Finland.
3. Composite columns. In Europe the I-shape composite column is mostly used. In Finland, RHS or CHS composite columns are used.

* Rolled Steel Sections.

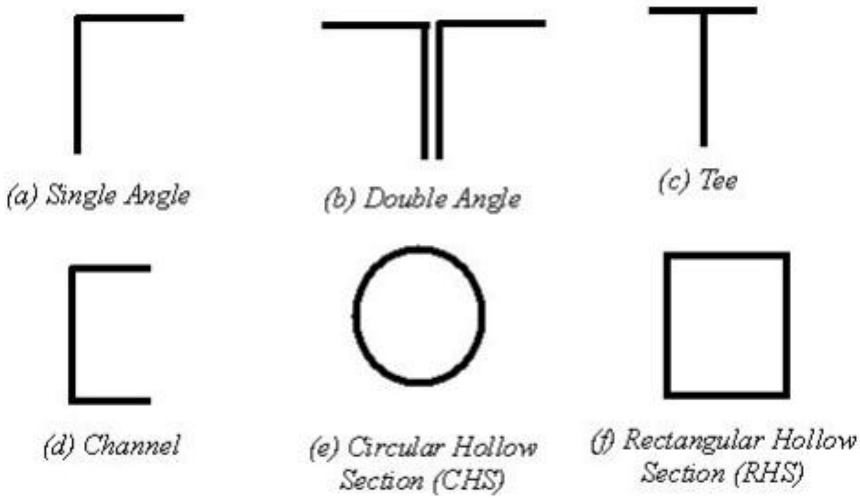


*Figure 10. Cross Section Shapes for Rolled Steel Compression Members (*http://nptel.iitm.ac.in/courses/IIT-MADRAS/Design_Steel_Structures_I/5_compression/8_types_of_column_sections.pdf*)*

(a) Single Angle. are satisfactory for bracings and for light trusses.
(b) Top chord members of roof trusses are usually made up of *double angles* back-to-back. The pair of angles used, has to be connected together, so they will act as one unit. Welds may be used at intervals – with a spacer bar between the connecting legs. Alternately "stitch bolts", washers and "ring fills" are placed between the angles to keep them at the proper distance apart (e.g. to enable a gusset to be connected). Such connections are called tack connections and the terms tack welding or tacks bolting are used.
(c) When welded roof trusses are required, there is no need for gusset plates and *T sections* can be employed as compression members.
(d) Single channels or C-sections are generally not satisfactory for use in compression, because of the low value of radius of gyration in the weak direction. They can be used if they are supported in a suitable way in the weak direction.
(e) Circular hollow sections are perhaps the most efficient as they have equal values of radius of gyration about every axis.
(f) The next best in terms of structural efficiency will be the *square hollow sections* (SHS) and *rectangular hollow sections*, both of which are increasingly becoming popular in tall buildings, as they are easily fabricated and erected. Welded tubes of circular, rectangular or square sections are very satisfactory for use as columns in a long series of windows and as short columns in walkways. For many structural applications the weight of hollow sections required would be only 50% of that required for open profiles like I or C sections.

When the available sections are not suitable, a suitable section may be built-up either by welding or by lacing or battening two sections separated by a suitable distance.(http://nptel.iitm.ac.in/courses/IIT-MADRAS/Design_Steel_Structures_I/5_compression/8_types_of_column_sections.pdf)

- Built-up column or fabricated compression members.

Compression members composed of two angles, channels, or tees back-to-back in contact or separated by a small distance shall be connected together by tack riveting, tack bolting or tack welding so that the individual sections do not buckle between the tacks before the whole member buckles.

When compression members are required for large structures like bridges, it will be necessary to use built-up sections. They are particularly useful when loads are heavy and members are long (e.g. top chords of Bridge Trusses). Built up sections (*Figure 11* (a) and *Figure 11* (b)) are popular when heavy loads are encountered. The cross section consists of two channel sections connected on their open sides with some type of lacing or latticing (dotted lines) to hold the parts together and ensure that they act together as one unit. The ends of these members are connected with "batten plates" which tie the ends together. Box sections of the type shown in *Figure 11* (a) or *Figure 11* (b) are sometimes connected by such solid plates either at intervals (battened) or continuously along the length.

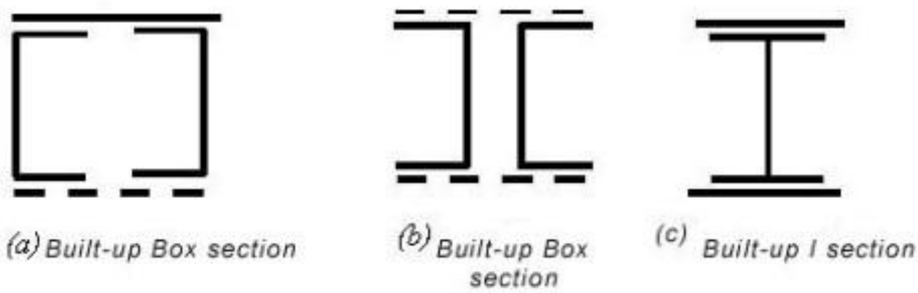


Figure 11. Cross Section Shapes for Built-up or fabricated Compression Members
*(*http://nptel.iitm.ac.in/courses/IIT-MADRAS/Design_Steel_Structures_I/5_compression/8_types_of_column_sections.pdf*)*

A pair of channels connected by cover plates on one side and latticing on the other is sometimes used as top chords of bridge trusses. The gussets at joints can be conveniently connected to the inside of the channels. Plated *I sections* or *built-up I sections* are used when the available rolled *I sections* do not have sufficient strengths to resist column loads (*Figure 11* (c)). Columns with open webs may be classified as *laced columns* or *battened columns.* In *Figure 12*, the two channel sections of the column are connected together by batten plates or laces which are shown by dotted lines. A typical lacing or batten plate is shown in *Figure 12.*

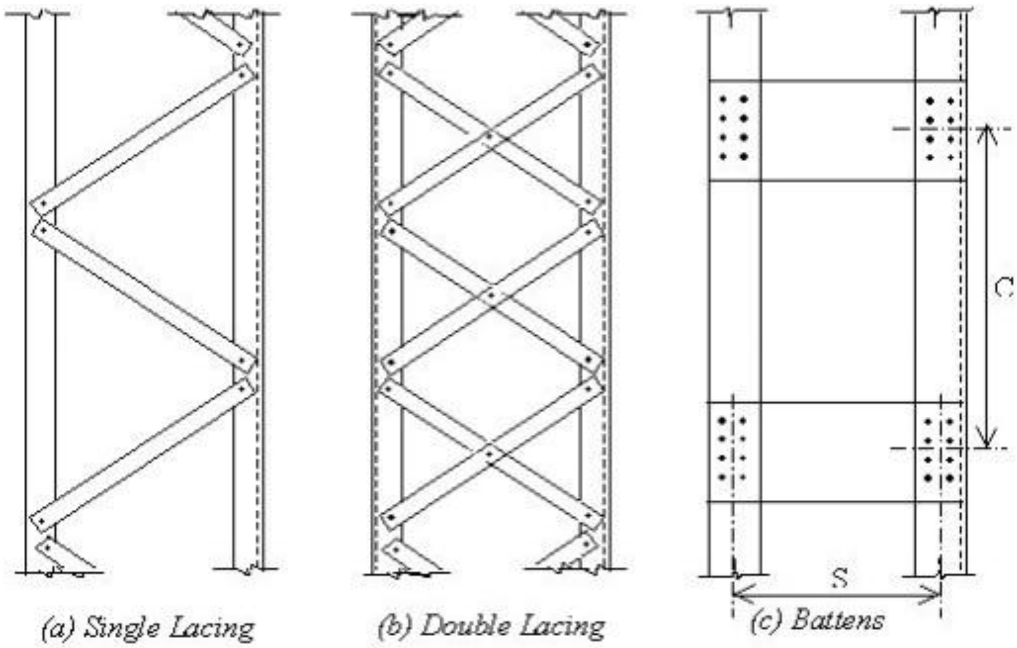


Figure 12. Built-up column members
*(*http://nptel.iitm.ac.in/courses/IIT-MADRAS/Design_Steel_Structures_I/5_compression/8_types_of_column_sections.pdf*)*

In laced columns, the lacing should be symmetrical in any two opposing faces to avoid torsion. Lacings and battens are not combined in the same column. (http://nptel.iitm.ac.in/courses/IIT-MADRAS/Design_Steel_Structures_I/5_compression/8_types_of_column_sections.pdf)

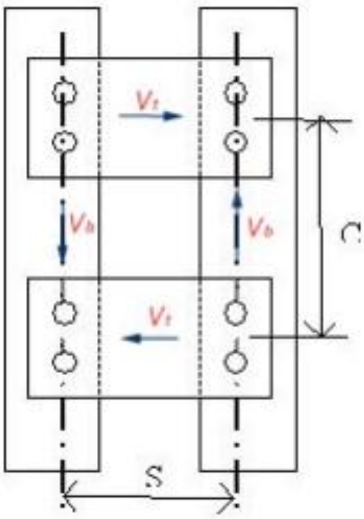


Figure 13. Battened Column
*(*http://nptel.iitm.ac.in/courses/IIT-MADRAS/Design_Steel_Structures_I/5_compression/8_types_of_column_sections.pdf*)*

3.2.2 Composite hollow sections

Composite column consists of a hollow steel section with a rebar cage placed inside. The column is filled with concrete on-site. The combination of steel and concrete provides superior load-bearing capability in normal and fire situations. On-site assembly is fast and the surface of the column is smooth and ready for final coating.

Applications:
- Offices
- Logistic centres and warehouses
- Recreational buildings
- Industrial buildings
- Multi-storey residential buildings (http://www.ruukki.com/Products-and-solutions/Building-solutions/Steel-frame-structures/Composite-column#tab2)

Figure 14. Composite column (http://www.ruukki.com/Products-and-solutions/Building-solutions/Steel-frame-structures/Composite-column#tab2)

Composite demonstration

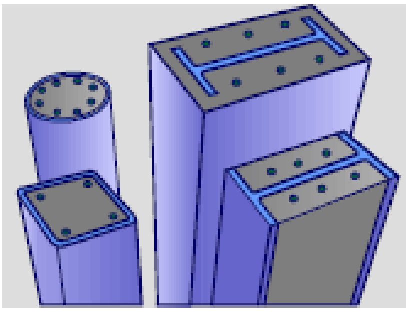


Figure 15. Different cross-sections

A hollow structural section (HSS) is a type of metal profile with a hollow tubular cross section. In some countries they are referred to instead as a structural hollow section (SHS). Most HSS are of circular or rectangular section (***Figure15***), although other shapes are available, such as elliptical. (http://www.indiantubecompany.com/metallic-hollow-section.html)

Figure 16. Solid Profiles

Advantages:

- High resistance and slender columns
- Advantages in case of biaxial bending
- No edge protection

Disadvantages:

- High material costs for profiles
- Difficult casting
- Additional reinforcement is needed for fire resistance

Column cross-sections:

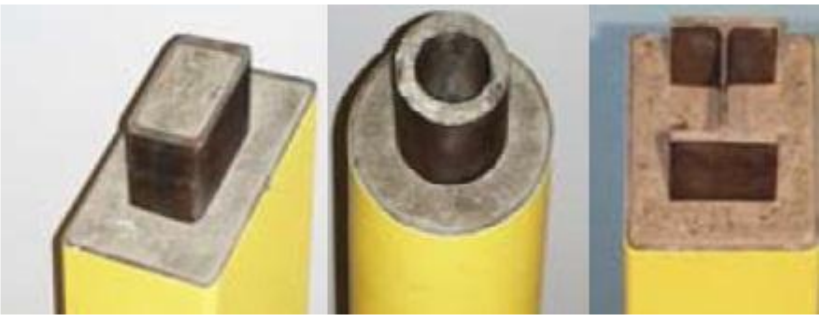


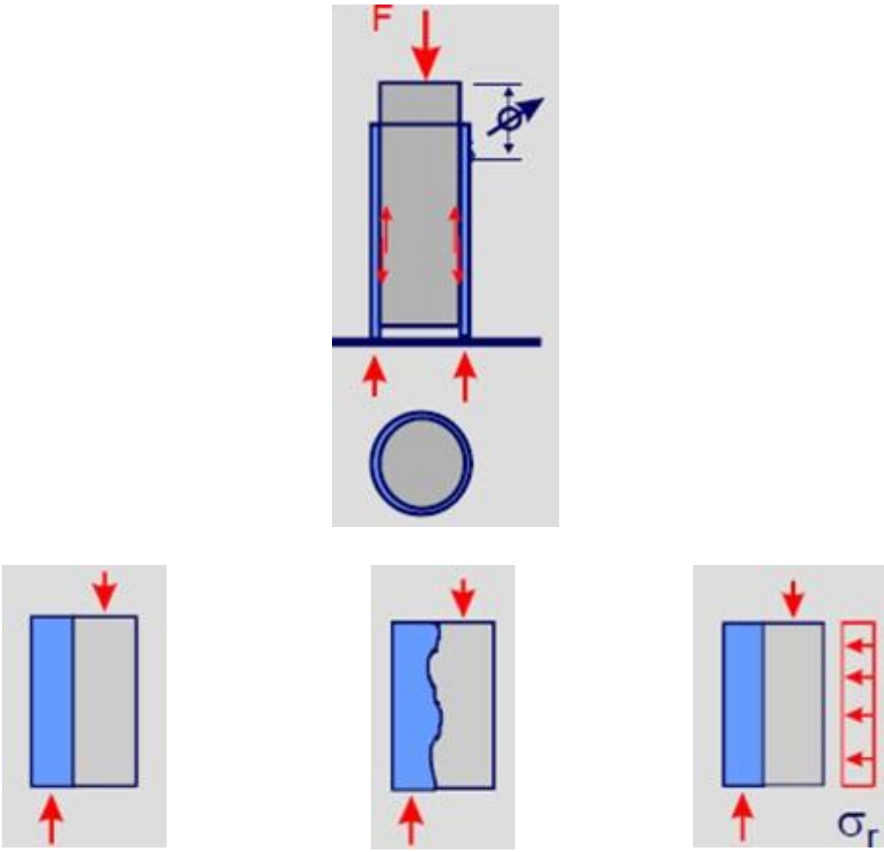
Figure 17. Hollow sections with additional inner profiles

Figure 18. Partially concrete encased sections

HSS, especially rectangular sections, are commonly used in welded steel frames where members experience loading in multiple directions. Square and circular HSS have very efficient shapes for this multiple-axis loading as they have uniform geometric and thus uniform strength characteristics along two or more cross-sectional axes; this makes them good choices for columns. They also have excellent resistance to torsion.

HSS can also be used as beams, although wide flange or I-beam shapes are in many cases a more efficient structural shape for this application. However, the HSS has superior resistance to lateral torsional buckling.

The flat square surfaces of rectangular HSS can ease construction, and they are sometimes preferred for architectural aesthetics in exposed structures, although elliptical HSS are becoming more popular in exposed structures for the same aesthetic reasons. (http://www.indiantubecompany.com/metallic-hollow-section.html)



a) Pure bond (adhesion)   b) Mechanical interlock   c) Friction

Figure 19. Steel and concrete cooperation in the joint

Outside the area of load introduction, longitudinal shear at the interface between concrete and steel should be verified where it is caused by transverse loads and(or) end moments. Shear connectors should be provided, based on the distribution of the design value of longitudinal shear, where this exceeds the design shear strength τ_{Rd} .

In absence of a more accurate method, elastic analysis, considering long term effects and cracking of concrete may be used to determine the longitudinal shear at the interface.

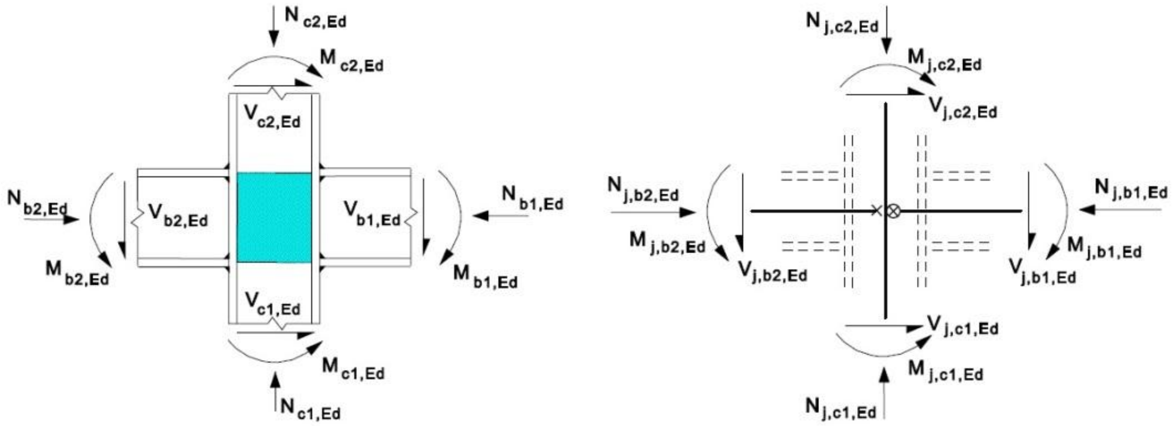
3.3 Connection between columns and beams

3.3.1 Basics about the connection

Joint configurations should be designed to resist the internal bending moments $M_{b1,Ed}$ and $M_{b2,Ed}$, normal forces $N_{b1,Ed}$ and $N_{b2,Ed}$ and shear forces $V_{b1,Ed}$ and $V_{b2,Ed}$ applied to the joints by the connected members. (*Figure 20*).

To model a joint in a way that closely reproduces the expected behaviour, the web panel in shear and each of the connections should be modelled separately, taking into account the internal moments and forces in the members, acting at the periphery of the web panel, see *Figure 20(a)* and *Figure 21*.

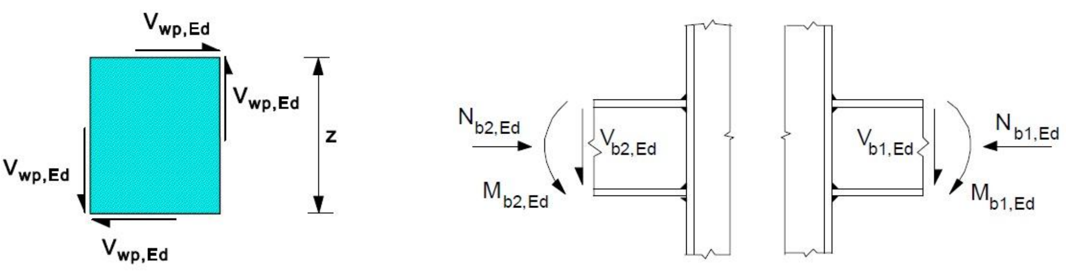
In a double-sided, beam-to-column joint each joint should be modelled as a separate rotational spring, as shown in *Figure 22*, in which each spring has a moment-rotation characteristic that takes into account the behaviour of the web panel in shear as well as the influence of the relevant connections. (EN-1993-1-8)



a) Values at periphery of web panel

b) Values at intersection of member centerlines

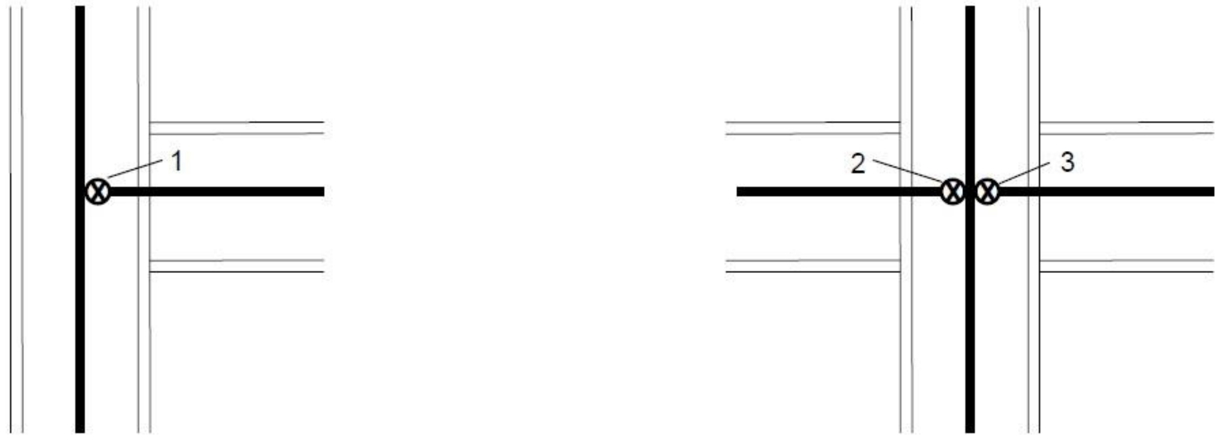
Figure 20. Forces and moments acting on the joint (EN-1993-1-8)



a) Shear forces in web panel

b) Connections, with forces and moments in beams

Figure 21. Forces and moments acting on the web panel (EN-1993-1-8)



Single-sided joint configuration Double-sided joint configuration

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

Figure 22. Modelling the joint (EN-1993-1-8)

When determining the design moment resistance and rotational stiffness for each of the joints, the possible influence of the web panel in shear should be taken into account by means of the transformation parameters β_1 and β_2.

More accurate values of β_1 and β_2 based on the values of the beam moments $M_{j,b1,Ed}$ and $M_{j,b2,Ed}$ at the intersection of the member centrelines, may be determined from the simplified model shown in *Figure 20(b)* as follows:

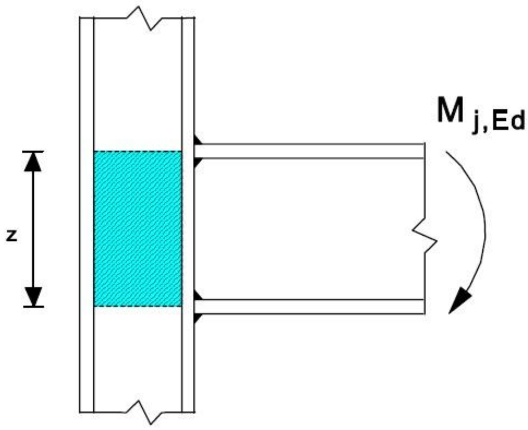
$$\beta_1 = \left|1 - \frac{M_{j,b_2,Ed}}{M_{j,b_1,Ed}}\right| \le 2, \quad (19)$$

$$\beta_2 = \left|1 - \frac{M_{j,b_1,Ed}}{M_{j,b_2,Ed}}\right| \le 2, \quad (20)$$

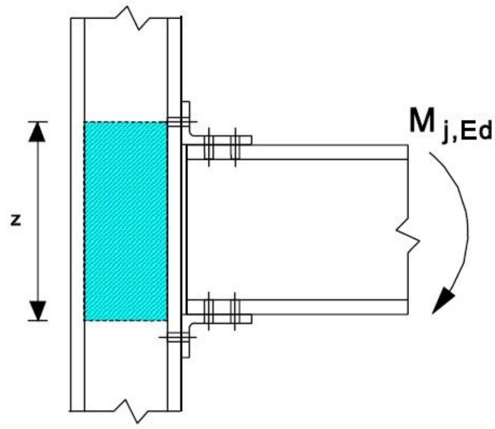
where:
$M_{j,b1,Ed}$ is the moment at the intersection from the right hand beam;
$M_{j,b2,Ed}$ is the moment at the intersection from the left hand beam. (EN-1993-1-8)

3.3.2 Different types of connection

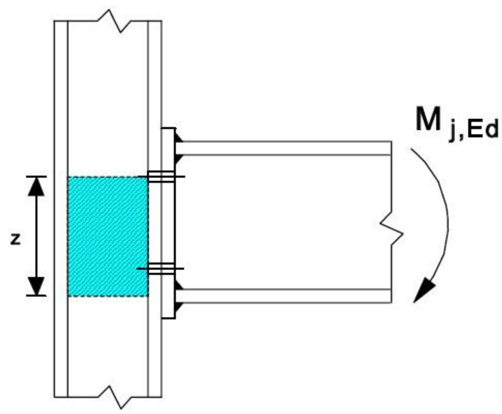
- Welded connection. (EN-1993-1-8)



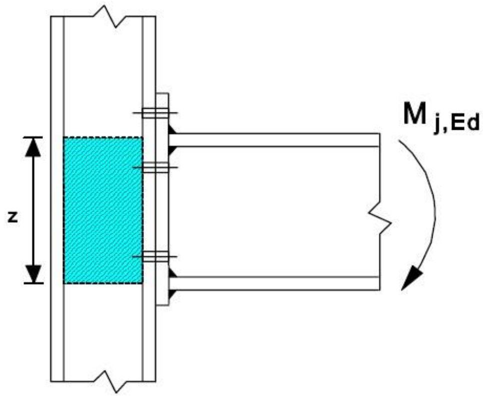
- Bolted connection with angle flange cleats. (EN-1993-1-8)



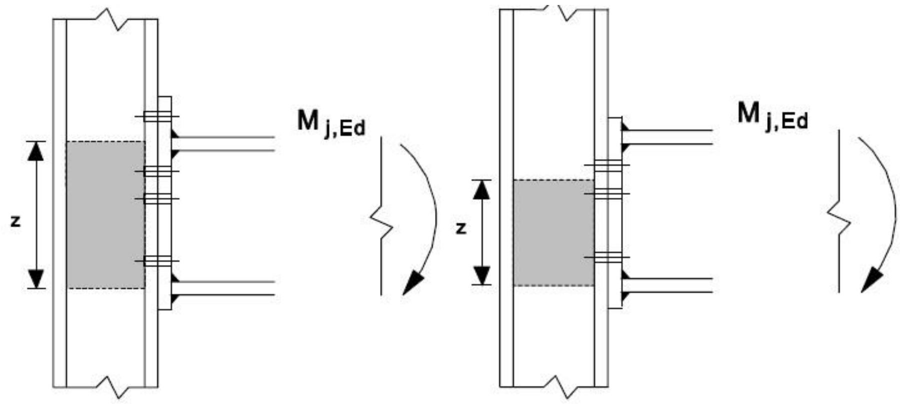
- Bolted end-plate connection with only one bolt-row active in tension. (EN-1993-1-8)



- Bolted extended end-plate connection with only two bolt-rows active in tension. (EN-1993-1-8)



- Other bolted end-plate connections with two or more boltrows in tension. (EN-1993-1-8)



The applied design moment $M_{j,Ed}$ should satisfy (EN-1993-1-8):

$$\frac{M_{j,Ed}}{M_{j,Rd}} \leq 1, \text{ (21)}$$

If the axial force N_{Ed} in the connected beam exceeds 5% of the design resistance, $N_{pl,Rd}$, the following conservative method may be used (EN-1993-1-8):

$$\frac{M_{j,Ed}}{M_{j,Rd}} + \frac{N_{j,Ed}}{N_{j,Rd}} \leq 1, \text{ (22)}$$

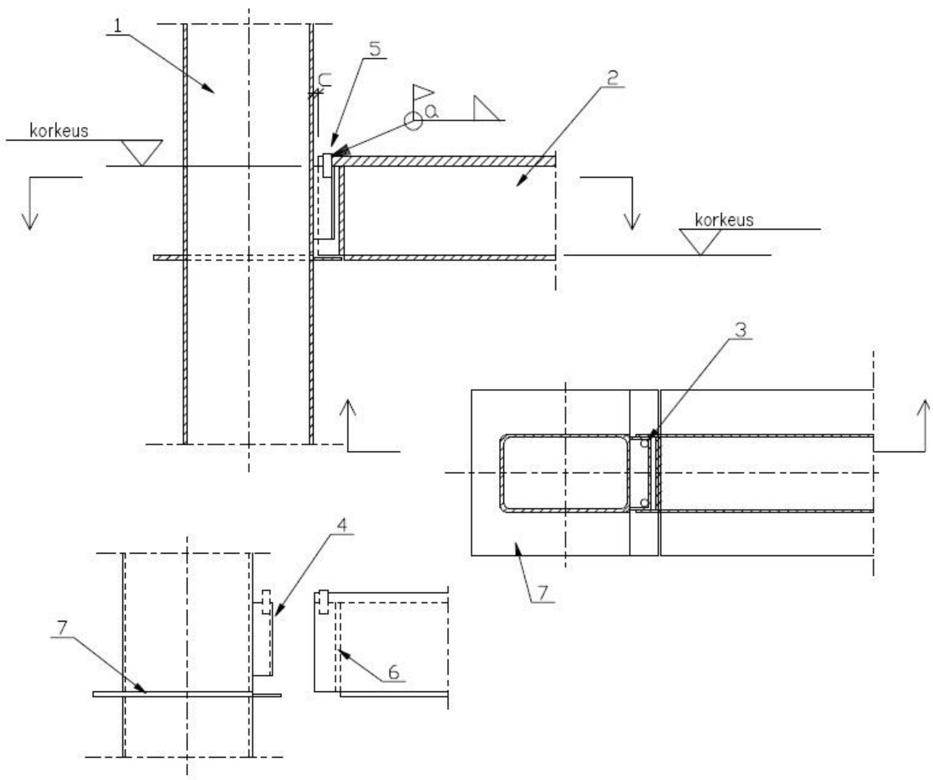
where:
Mj.Rd is the design moment resistance of the joint, assuming no axial force;
Nj.Rd is the axial design resistance of the joint, assuming no applied moment.

3.3.3 WQ-hollow section connection

Connecting technology plays an important role in the performance of hollow section structures. A distinction has to be made between CHS and RHS connected members, because the behaviour of joints, e.g. local behaviour of members is different. A particular case is represented by beam-to-column joints in building frames with Concrete Filled Hollow Section (CFHS) columns. Both welded and/or bolted connections can be used in such a case.

For beam-to-column joints of hollow section frames (e.g. RHS columns and beams or hollow section columns and I or H section beams), blind bolting technology is available. This section summarises the main aspects concerning the behaviour and design of hollow section connections loaded predominantly statically. This means they can also be used for seismic resistant buildings, since seismic motions are not considered as generating fatigue phenomena. (http://people.fsv.cvut.cz/~wald/CESTRUCO/Texts_of_lessons/10_GB_Hollow_Section_Joints.pdf)

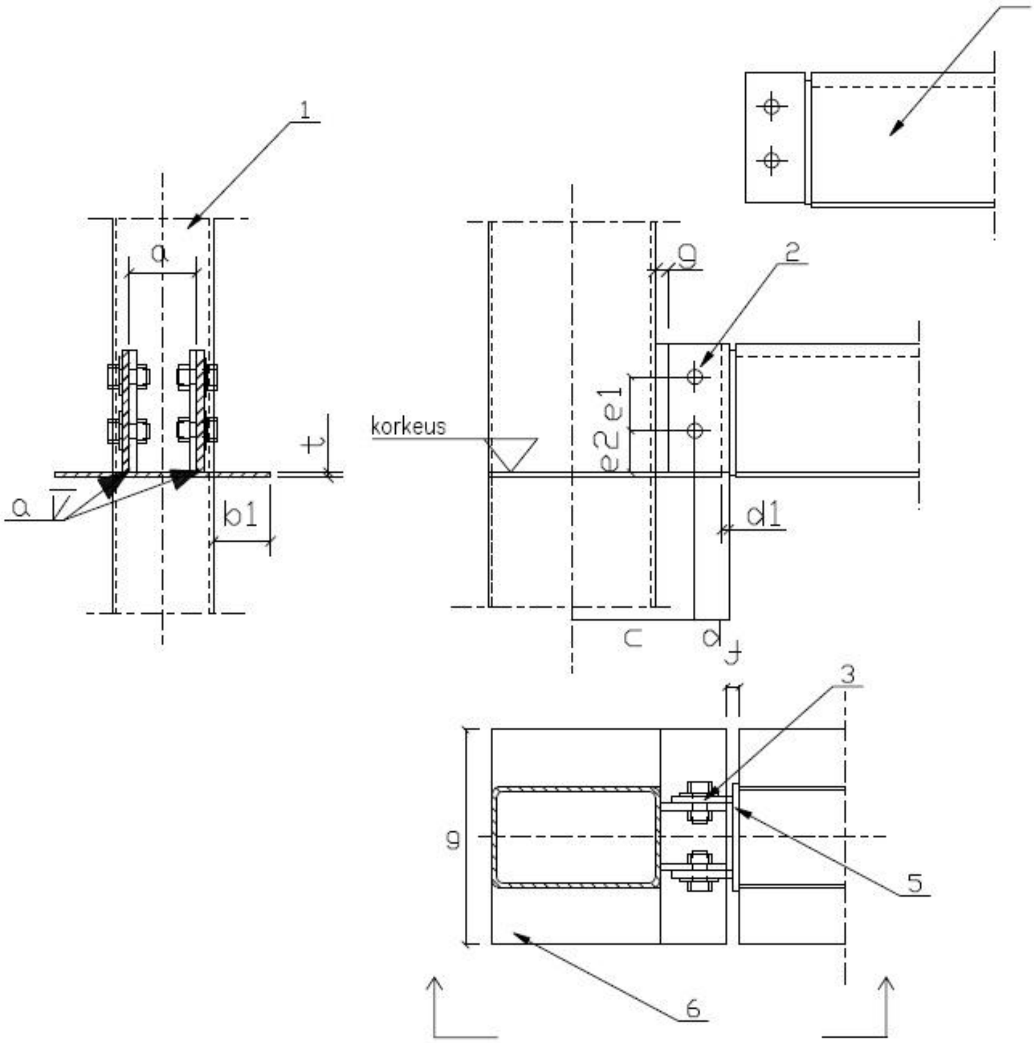
Connection 1. (Finnmap Consulting Oy design)



1 Hollow section profile, concrete filling and reinforcement;
2 WQ-beam;
3 Connection pin 2 D25, L=100+50, welded to U-shape steel box;
4 U-shape steel box, welded to column;
5 Connection pin, welded to beam after placing the beam;
6 Stiffening plate inside the WQ-beam;
7 Supporting plate in external columns for hollow core slabs.

The steel beam is supported during the erection of the slabs.
The connection is cast inside concrete.
If necessary the column is fire protected with paint in the connection area.

Connection 2. (Finnmap Consulting Oy design)



1 Hollow section profile, concrete filling and reinforcement;
2 Bolts 4 M24 8.8, SFS EN 24014;
3 Connection plates;
4 WQ-beam;
5 End plate;
6 Supporting plate in external columns for hollow core slabs.

The steel beam is supported during the erection of the slabs.
The connection is cast inside concrete.
If necessary the column is fire protected with paint in the connection area.

3.4. Engineering of the connection

3.4.1 Basics about the engineering

Structural steel design of framework can be categorized into three areas: main members, secondary members and connections.

The structural engineering of main members may include beams, column, trusses, and girders. Main members are the skeleton of the framework, bracing and the primary members that carry the loads imparted on the structure. Simply, it is the part of the structure that holds things up. The structural engineering of secondary members may include stairs, and decking. Secondary structural elements are designed to carry specific loads. For example, a brace is added to provide extra support in the area of a load thereby reducing the size of a member or the moment at a connection. Connections are joints or nodes of structural elements used to transfer forces between structural elements or members. The structural engineering of connections ensures that at the point (node) where the structural members are meet (connect), sufficient steel area exists to resist the cumulative stresses at that node – axial loads (compression and tension), bending moments, and torsional loadings (torque).

The process of fabrication is the process of cutting, burning, welding, drilling, grinding, punching, bending and generally producing the steel detail pieces shown on the detail drawings.

Later in the process, fitters or a separate welding crew will attach a series of detail pieces together to form the assemblies or shipping pieces. There are many techniques used in welding metal together.

The two most common welding processes in the construction industry fall under the categories of gas welding and arc welding. Gas welding is a process in which heat is produced with an electric arc formed between a metal being welded. An inert gas, usually helium or argon shields the arc from contamination.

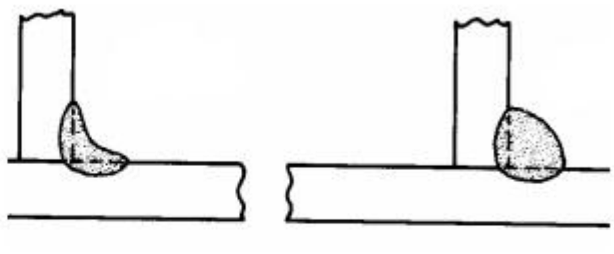


Figure 23. Typical fillet weld details (http://www.greyhawk.com/news/technical/Design_Structural_Steel_Design_and_Construction.pdf)

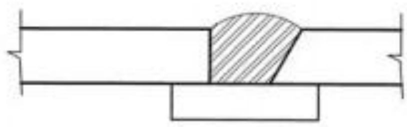


Figure 24. Full penetration joint (http://www.greyhawk.com/news/technical/Design_Structural_Steel_Design_and_Construction.pdf)

Carbon arc welding is a pudding process in which the heat from an electric arc creates a small pool of molten metal that can be added to using metal from a filler rod. This is sometimes referred to as stick welding. (http://www.greyhawk.com/news/technical/Design_Structural_Steel_Design_and_Construction.pdf)

3.4.2 Welding

Types of weld:

- fillet welds
- fillet welds all round
- butt welds
- plug welds
- flare groove welds. (EN 1993-1-8: 2005)

Butt welds may be either full penetration butt welds or partial penetration butt welds. Both fillet welds all round and plug welds may be either in circular holes or in elongated holes.

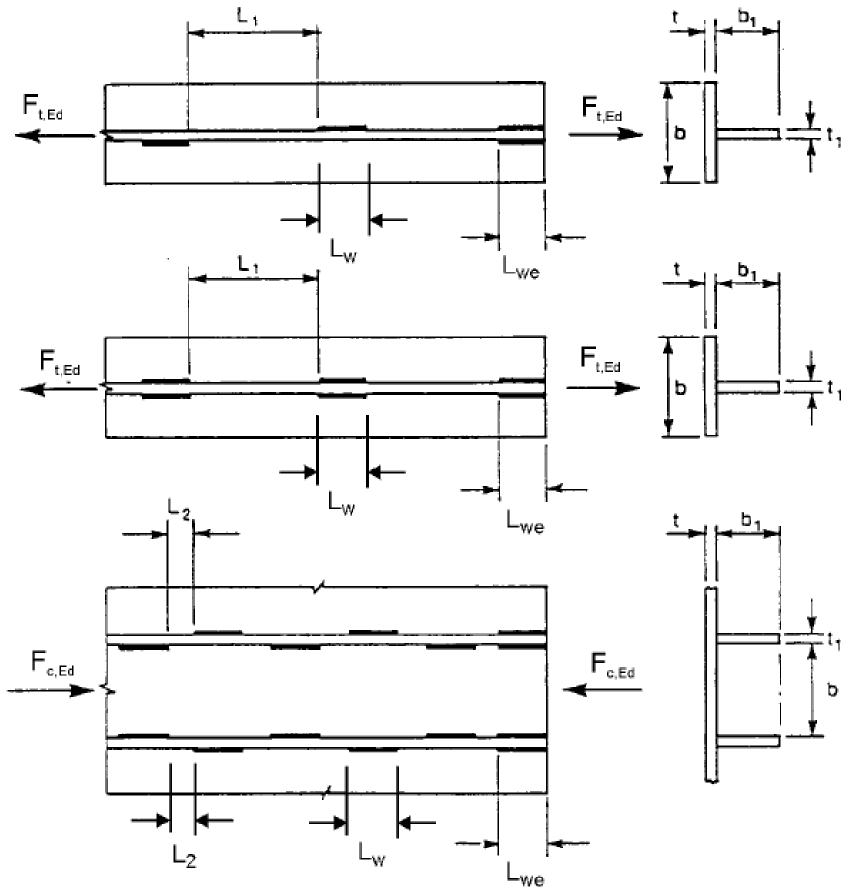
- Fillet welds.

Fillet welds may be used for connecting parts where the fusion faces form an angle of between 60° and 120°. Angles smaller than 60° are also permitted. However, in such cases the weld should be considered to be a partial penetration butt weld. For angles greater than 120° the resistance of fillet welds should be determined by testing.

Fillet welds finishing at the ends or sides of parts should be returned continuously, full size, around the corner for a distance of at least twice the leg length of the weld, unless access or the configuration of the joint renders this impracticable.

Intermittent fillet welds should not be used in corrosive conditions. In an intermittent fillet weld, the gap (L1 or L2) should be taken as the smaller of the distances between the ends of the welds on opposite sides and the distance between the ends of the welds on the same side. In any run of intermittent fillet weld there should always be a length of weld at each end of the part connected.

In a built-up member in which plates are connected by means of intermittent fillet welds, a continuous fillet weld should be provided on each side of the plate for a length at each end equal to at least three-quarters of the width of the narrower plate concerned (see *Figure 25*).



The smaller of $L_{we} \geq 0,75$ b and 0,75 b_1
For build-up members in tension:
The smallest of $L_1 \leq 16$ t and 16 t_1 and 200 mm
For build-up members in tension:
The smallest of $L_1 \leq 16$ t and 16 t_1 and 200 mm

Figure 25. Intermittent fillet welds (EN 1993-1-8: 2005)

- Fillet welds all round.

Fillet welds all round, comprising fillet welds in circular or elongated holes, may be used only to transmit shear or to prevent the buckling or separation of lapped parts. The diameter of a circular hole, or width of an elongated hole, for a fillet weld all round should not be less than four times the thickness of the part containing it.

The ends of elongated holes should be semi-circular, except for those ends which extend to the edge of the part concerned. The centre to centre spacing of fillet welds all round should not exceed the value necessary to prevent local buckling.

- Butt welds.

A full penetration butt weld is defined as a weld that has complete penetration and fusion of weld and parent metal throughout the thickness of the joint. A partial penetration butt weld is defined as a weld that has joint penetration which is less than the full thickness of the parent material. Intermittent butt welds should not be used.

- Plug welds.

Plug welds may be used:
– to transmit shear
– to prevent the buckling or separation of lapped parts
– to inter-connect the components of built-up members
But should not be used to resist externally applied tension.

The diameter of a circular hole, or width of an elongated hole, for a plug weld should be at least 8 mm more than the thickness of the part containing it.

The ends of elongated holes should either be semi-circular or else should have corners which are rounded to a radius of not less than the thickness of the part containing the slot, except for those ends which extend to the edge of the part concerned.

The thickness of a plug weld in parent material up to 16 mm thick should be equal to the thickness of the parent material. The thickness of a plug weld in parent material over 16 mm thick should be at least half the thickness of the parent material and not less than 16 mm.

The centre to centre spacing of plug welds should not exceed the value necessary to prevent local buckling.

- Flare groove welds.

For solid bars the design effective throat thickness of flare groove welds, when fitted flush to the surface of the solid section of the bars, is defined in ***Figure 25***. (EN 1993-1-8: 2005)

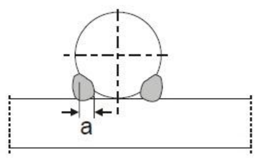


*Figure 26**.** Effective throat thickness of flare groove welds in solid sections (EN 1993-1-8: 2005)*

3.4.3 Bolted connections

Bolted connections are mostly detachable. They are selected for the on site assembly in order to avoid site welding, which may cause welding errors due to environmental difficulties. Site welding is also more costly than site bolting. The main types of bolted connections for hollow section structures are: Bolted knee joints, Flange connections, Splice joints, Joints with fork ends, Screwed tensioner, Through bolting, Bolted connections with flattened ends, Hinged support, Column bases, Fish plate connections, Bolted subassemblies, and Fixing bolts through hand access holes.

These connections are realised using intermediate connecting steel devices, which are welded on the hollow section members, the bolted connections themselves being designed as normal connections. For this reason, design of hollow section connections does not imply specific requirements. (http://people.fsv.cvut.cz/~wald/CESTRUCO/Texts_of_lessons/10_GB_Hollow_Section_Joints.pdf)

Bolt classes

Table 3. Nominal values of the yield strength f_{yb} and the ultimate tensile strength f_{ub} for bolts.
(EN 1993-1-8: 2005)

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

Categories of bolted connections

Bolted connections loaded in shear should be designed as one of the following:

a) Category A: Bearing type

In this category bolts from class 4.6 up to and including class 10.9 should be used. No preloading and special provisions for contact surfaces are required. The design ultimate shear load should not exceed the design shear resistance, nor the design bearing resistance.

b) Category B: Slip-resistant at serviceability limit state

In this category preloaded bolts (of classes 8.8 and 10.9) should be used. Slip should not occur at the serviceability limit state. The design serviceability shear load should not exceed the design slip resistance. The design ultimate shear load should not exceed the design shear resistance, nor the design bearing resistance.

c) Category C: Slip-resistant at ultimate limit state

In this category preloaded bolts (of classes 8.8 and 10.9) should be used. Slip should not occur at the ultimate limit state. The design ultimate shear load should not exceed the design slip resistance, nor the design bearing resistance. In addition for a connection in tension, the design plastic resistance of the net cross-section at bolt holes $N_{net,Rd}$ should be checked, at the ultimate limit state. (EN 1993-1-8: 2005)

Bolted connection loaded in tension should be designed as one of the following:

d) Category D: non-preloaded

In this category bolts from class 4.6 up to and including class 10.9 should be used. No preloading is required. This category should not be used where the connections are frequently subjected to variations of tensile loading. However, they may be used in connections designed to resist normal wind loads.

e) Category E: preloaded

In this category preloaded 8.8 and 10.9 bolts with controlled tightening should be used. (EN 1993-1-8: 2005)

The bolt classes for these connections are summarized in *Table 3*.

A beam-to-column joint in which the design moment resistance of the joint $M_{j,Rd}$ is governed by the design resistance of the column web panel in shear, may be assumed to have adequate rotation capacity for plastic global analysis, provided that $d/t_w \leq 69\varepsilon$. (EN 1993-1-8: 2005)

A joint with either a bolted end-plate or angle flange cleat connection may be assumed to have sufficient rotation capacity for plastic analysis, provided that both of the following conditions are satisfied:

a) the design moment resistance of the joint is governed by the design resistance of either:
   - the column flange in bending or
   - the beam end-plate or tension flange cleat in bending.
b) the thickness t of either the column flange or the beam end-plate or tension flange cleat (not necessarily the same basic component as in (a)) satisfies:

$$t \leq 0{,}36d\sqrt{f_{ub}/f_y} \text{ , (23)}$$

where: f_y is the yield strength of the relevant basic component. (EN 1993-1-8: 2005)

A joint with a bolted connection in which the design moment resistance $M_{j,Rd}$ is governed by the design resistance of its bolts in shear, should not be assumed to have sufficient rotation capacity for plastic global analysis. (EN 1993-1-8: 2005)

3.4.4 Plates in connection

Column bases should be of sufficient size, stiffness and strength to transmit the axial forces, bending moments and shear forces in columns to their foundations or other supports without exceeding the load carrying capacity of these supports. (EN 1993-1-8: 2005)

The design bearing strength between the base plate and its support may be determined on the basis of a uniform distribution of compressive force over the bearing area. For concrete foundations the bearing stress should not exceed the design bearing strength, f_{jd}. (EN 1993-1-8: 2005)

For a column base subject to combined axial force and bending the forces between the base plate and its support can take one of the following distribution depending on the relative magnitude of the applied axial force and bending moment:

- In the case of a dominant compressive axial force, full compression may develop under both column flanges as shown in *Figure 27(a)*.

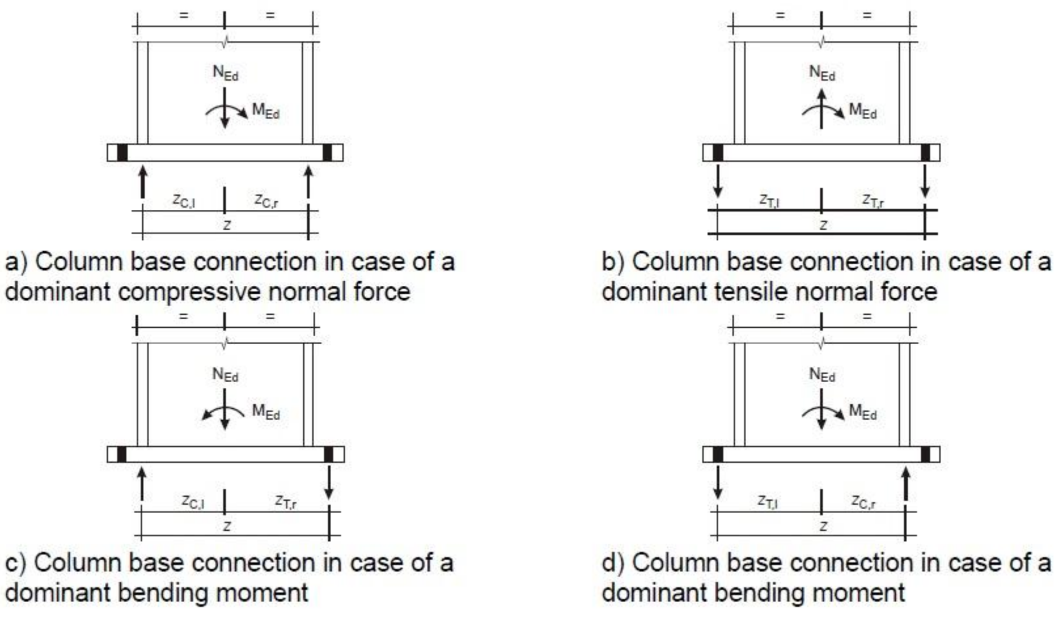
- In the case of a dominant tensile force, full tension may develop under both flanges as shown in *Figure 27(b).*
- In the case of a dominant bending moment compression may develop under one column flange and tension under the other as shown in *Figure 27(c)* and *Figure 27(d).*

One of the following methods should be used to resist the shear force between the base plate and its support:

- Frictional design resistance at the joint between the base plate and its support.
- The design shear resistance of the anchor bolts.
- The design shear resistance of the surrounding part of the foundation.

If anchor bolts are used to resist the shear forces between the base plate and its support, rupture of the concrete in bearing should also be checked, according to EN 1992.

Where the above methods are inadequate special elements such as blocks or bar shear connectors should be used to transfer the shear forces between the base plate and its support. (EN 1993-1-8: 2005)



a) Column base connection in case of a dominant compressive normal force

b) Column base connection in case of a dominant tensile normal force

c) Column base connection in case of a dominant bending moment

d) Column base connection in case of a dominant bending moment

Figure 27. Determination of the lever arm z for column base connections (EN 1993-1-8: 2005)

For a bolted end-plate joint with more than one row of bolts in tension, the stiffness coefficients k_i for the related basic components should be combined.

In a bolted end plate joint with more than one bolt-row in tension, as a simplification the contribution of any bolt-row may be neglected, provided that the contributions of all other bolt-rows closer to the centre of compression are also neglected. The number of bolt-rows retained need not necessarily be the same as for the determination of the design moment resistance. (EN 1993-1-8: 2005)

The basic components that should be taken into account when calculating the stiffness of a welded beam-to-column joint and a joint with bolted angle flange cleats are given in *Table 4*. Similarly, the basic components for a bolted end-plate connection and a base plate are given in *Table 5*. In both of these tables the stiffness coefficients, ki ,for the basic components are defined in *Table 6*. (EN 1993-1-8: 2005)

Table 4. Joints with welded or bolted angle flange cleat connections (EN 1993-1-8: 2005)

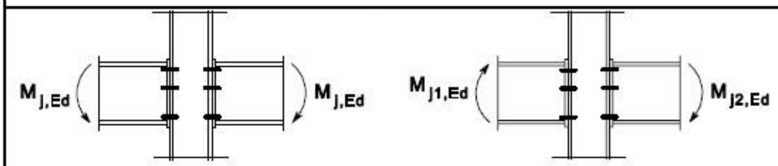
Beam-to-column joint with welded connections	Stiffness coefficients k_i to be taken into account
Single-sided	k_1; k_2; k_3
Double-sided – Moments equal and opposite	k_2; k_3
Double-sided – Moments unequal	k_1; k_2; k_3
Beam-to-column joint with Bolted angle flange cleat connections	Stiffness coefficients k_i to be taken into account
Single-sided	k_1; k_2; k_3; k_4; k_6; k_{10}; k_{11} *); k_{12} **)
Double-sided – Moments equal and opposite	k_2; k_3; k_4; k_6; k_{10}; k_{11} *); k_{12} **)
Double-sided – Moments unequal	k_1; k_2; k_3; k_4; k_6; k_{10}; k_{11} *); k_{12} **)
 Moments equal and opposite Moments unequal	*) Two k_{11} coefficients, one for each flange; **) Four k_{12} coefficients, one for each flange and one for each cleat.

Table 5. Joints with bolted end-plate and base plate connections (EN 1993-1-8: 2005)

Beam-to-column joint with bolted end-plate connections	Number of bolt-rows in tension	Stiffness coefficients k_i to be taken into account
Single-sided	One	k_1; k_2; k_3; k_4; k_5; k_{10}
	Two or more	k_1; k_2; k_{eq}
Double sided – Moments equal and opposite	One	k_2; k_3; k_4; k_5; k_{10}
	Two or more	k_2; k_{eq}
Double sided – Moments unequal	One	k_1; k_2; k_3; k_4; k_5; k_{10}
	Two or more	k_1; k_2; k_{eq}
Beam splice with bolted end-plates	**Number of bolt-rows in tension**	**Stiffness coefficients k_i to be taken into account**
Double sided - Moments equal and opposite	One	k_5[left]; k_5[right]; k_{10}
	Two or more	k_{eq}
Base plate connections	**Number of bolt-rows in tension**	**Stiffness coefficients k_i to be taken into account**
Base plate connections	One	k_{13}; k_{15}; k_{16}
	Two or more	k_{13}; k_{15} and k_{16} for each bolt row

Table 6. Stiffness coefficients for basic joint components.
(EN 1993-1-8: 2005)

Component	Stiffness coefficient k_i	
Column web panel in shear	Unstiffened, single-sided joint, or a double-sided joint in which the beam depths are similar	stiffened
	$k_1 = \dfrac{0{,}38 A_{VC}}{\beta z}$	$k_1 = \infty$
	z is the lever arm from Figure 6.15; β is the transformation parameter from 5.3(7).	
Column web in compression	unstiffened	stiffened
	$k_2 = \dfrac{0{,}7 b_{eff,c,wc} t_{wc}}{d_c}$	$k_2 = \infty$
	$b_{eff,c,wc}$ is the effective width from 6.2.6.2	
Column web in tension	stiffened or unstiffened bolted connection with a single bolt-row in tension or unstiffened welded connection	stiffened welded connection
	$k_3 = \dfrac{0{,}7 b_{eff,t,wc} t_{wc}}{d_c}$	$k_3 = \infty$
	$b_{eff,t,wc}$ is the effective width of the column web in tension from 6.2.6.3. For a joint with a single bolt-row in tension, $b_{eff,t,wc}$ should be taken as equal to the smallest of the effective lengths ℓ_{eff} (individually or as part of a group of bolt-rows) given for this bolt-row in Table 6.4 (for an unstiffened column flange) or Table 6.5 (for a stiffened column flange).	
Column flange in bending (for a single bolt-row in tension)	$k_4 = \dfrac{0{,}9 \ell_{eff} t_{fc}^3}{m^3}$ ℓ_{eff} is the smallest of the effective lengths (individually or as part of a bolt group) for this bolt-row given in Table 6.4 for an unstiffened column flange or Table 6.5 for a stiffened column flange; m is as defined in Figure 6.8.	
End-plate in bending (for a single bolt-row in tension)	$k_5 = \dfrac{0{,}9 \ell_{eff} t_p^3}{m^3}$ ℓ_{eff} is the smallest of the effective lengths (individually or as part of a group of bolt-rows) given for this bolt-row in Table 6.6; m is generally as defined in Figure 6.11, but for a bolt-row located in the extended part of an extended end-plate $m = m_x$, where m_x is as defined in Figure 6.10.	
Flange cleat in bending	$k_6 = \dfrac{0{,}9 \ell_{eff} t_a^3}{m^3}$ ℓ_{eff} is the effective length of the flange cleat from Figure 6.12; m is as defined in Figure 6.13.	

Component	Stiffness coefficient k_i
Bolts in tension (for a single bolt-row)	$k_{10} = 1{,}6 A_s / L_b$ preloaded or non-preloaded L_b is the bolt elongation length, taken as equal to the grip length (total thickness of material and washers), plus half the sum of the height of the bolt head and the height of the nut.
Bolts in shear	non-preloaded: $k_{11} \text{ (or } k_{17}) = \dfrac{16 n_b d^2 f_{ub}}{E d_{M16}}$ preloaded [*]: $k_{11} = \infty$ d_{M16} is the nominal diameter of an M16 bolt; n_b is the number of bolt-rows in shear.
Bolts in bearing (for each component j on which the bolts bear)	non-preloaded: $k_{12} \text{ (or } k_{18}) = \dfrac{24 n_b k_b k_t d f_u}{E}$ preloaded [*]: $k_{12} = \infty$ $k_b = k_{b1}$ but $k_b \le k_{b2}$ $k_{b1} = 0{,}25\, e_b / d + 0{,}5$ but $k_{b1} \le 1{,}25$ $k_{b2} = 0{,}25\, p_b / d + 0{,}375$ but $k_{b2} \le 1{,}25$ $k_t = 1{,}5\, t_j / d_{M16}$ but $k_t \le 2{,}5$ e_b is the distance from the bolt-row to the free edge of the plate in the direction of load transfer; f_u is the ultimate tensile strength of the steel on which the bolt bears; p_b is the spacing of the bolt-rows in the direction of load transfer; t_j is the thickness of that component.
Concrete in compression (including grout)	$k_{13} = \dfrac{E_c \sqrt{b_{eff} l_{eff}}}{1{,}275 E}$ b_{eff} is the effective width of the T-stub flange, see 6.2.5(3); l_{eff} is the effective length of the T-stub flange, see 6.2.5(3).
Plate in bending under compression	$k_{14} = \infty$ This coefficient is already taken into consideration in the calculation of the stiffness coefficient k_{13}
Base plate in bending under tension (for a single bolt row in tension)	with prying forces [**]: $k_{15} = \dfrac{0{,}85 \ell_{eff} t_p^3}{m^3}$ without prying forces [**]: $k_{15} = \dfrac{0{,}425 \ell_{eff} t_p^3}{m^3}$ l_{eff} is the effective length of the T-stub flange, see 6.2.5(3); t_p is the thickness of the base plate; m is the distance according to Figure 6.8.
Anchor bolts in tension	with prying forces [**]: $k_{16} = 1{,}6 A_s / L_b$ without prying forces [**]: $k_{16} = 2{,}0 A_s / L_b$ L_b is the anchor bolt elongation length, taken as equal to the sum of 8 times the nominal bolt diameter, the grout layer, the plate thickness, the washer and half of the height of the nut.

*) provided that the bolts have been designed not to slip into bearing at the load level concerned

**) prying forces may develop, if $L_b \le \dfrac{8{,}8 m^3 A_s}{l_{eff} t^3}$

For end-plate joints with two or more bolt-rows in tension, the basic components related to all of these bolt-rows should be represented by a single equivalent stiffness coefficient k_{eq} determined from:

$$k_{eq} = \frac{\sum_r k_{eff,r} h_r}{z_{eq}} \text{, (24)}$$

where:
h_r is the distance between bolt-row r and the centre of compression;
$k_{eff,r}$ is the effective stiffness coefficient for bolt-row r taking into account the stiffness coefficients k_i for the basic components;
z_{eq} is the equivalent lever arm.

The effective stiffness coefficient keff,r for bolt-row r should be determined from:

$$k_{eff,r} = \frac{1}{\sum_i \frac{1}{k_{i,r}}} \text{, (25)}$$

where:
$k_{i,r}$ is the stiffness coefficient representing component *i* relative to bolt-row *r* .

The equivalent lever arm z_{eq} should be determined from:

$$z_{eq} = \frac{\sum_r k_{eff,r} h_r^2}{\sum_r k_{eff,r} h_r} \text{, (26)}$$

In the case of a beam-to-column joint with an end-plate connection, k_{eq} should be based upon (and replace) the stiffness coefficients k_i for:

- the column web in tension (k_3);
- the column flange in bending (k_4);
- the end-plate in bending (k_5);
- the bolts in tension (k_{10}).

In the case of a beam splice with bolted end-plates, k_{eq} should be based upon (and replace) the stiffness coefficients k_i for:

- the end-plates in bending (k_5);
- the bolts in tension (k_{10}).

For extended end-plate connections with two bolt-rows in tension, (one in the extended part of the end-plate and one between the flanges of the beam, see *Figure 28*), a set of modified values may be used for the stiffness coefficients of the related basic components to allow for the combined contribution of both bolt-rows. Each of these modified values should be taken as twice the corresponding value for a single bolt-row in the extended part of the end-plate.

When using this simplified method, the lever arm z should be taken as equal to the distance from the centre of compression to a point midway between the two bolt-rows in tension, see *Figure 28*. (EN 1993-1-8: 2005)

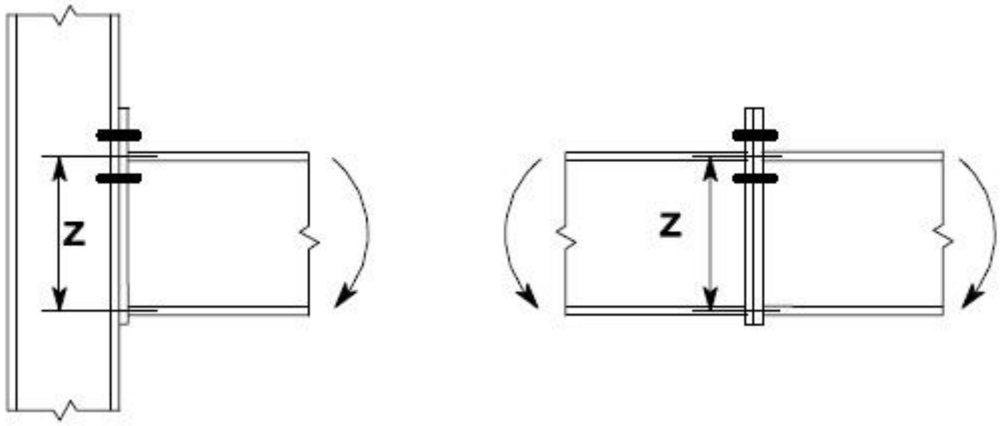


Figure 28. Lever arm z for simplified method
(EN 1993-1-8: 2005)

4 THE DESIGN AND CALCULATION

For welds the strength is based on ultimate strength (rather than yield strength). A partial safety factor is also included. (LUT Steel Structures Design – Gary B. Marquis)

The weld must full these requirements. The normal stress related to ultimate strength (LUT Steel Structures Design – Gary B. Marquis):

$$\sigma_{\perp} \leq \frac{f_u}{\gamma_{M2}}, \quad (27)$$

For combined stresses (LUT Steel Structures Design – Gary B. Marquis):

$$\sqrt{\sigma_{\perp}^2 + 3\tau_{\perp}^2 + 3\tau_{II}^2} \leq \frac{f_u}{\beta_w \cdot \gamma_{M2}}, \quad (28)$$

Partial safety factor based on ultimate limit state, γ_{M2} = 1.25. (LUT Steel Structures Design – Gary B. Marquis)

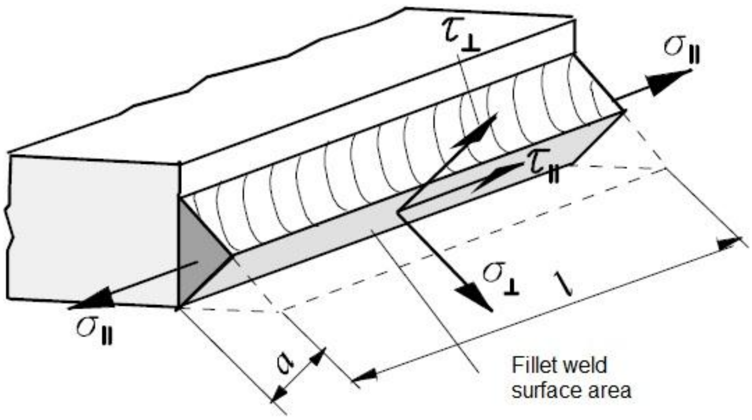


Figure 29. Stresses on the throat section of a fillet weld. (LUT Steel Structures Design – Gary B. Marquis)

Design principle

- Weld stress is computed as force divided by total area (simple and conservative):

$$\frac{F}{a \cdot l} \leq f_{vw,d}, \quad (29)$$

- Stress should be less than some allowed value based on material:

$$\sigma_{eq} \leq f_{vw,d}, \quad (30)$$

- Both *a* and *l* should be within an acceptable range. (LUT Steel Structures Design – Gary B. Marquis)

Consider a beam to column weld. Often it is assumed that the flanges support the full moment and the web supports the shear. (LUT Steel Structures Design – Gary B. Marquis)

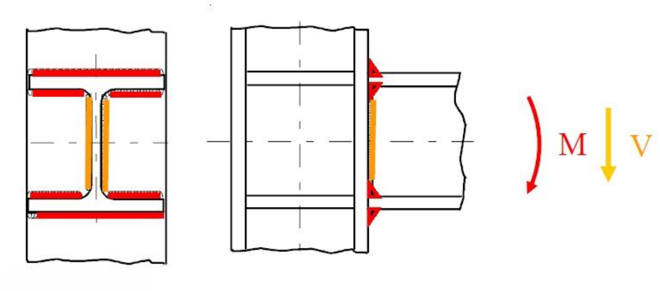


Figure 30. Moment and shear force.
(LUT Steel Structures Design – Gary B. Marquis)

Stresses in plate

$$F_{\perp} = \frac{\sigma_x \cdot t}{2} \text{ , (31)} \qquad F_{II} = \frac{\tau_{xy} \cdot t}{2} \text{ , (32)}$$

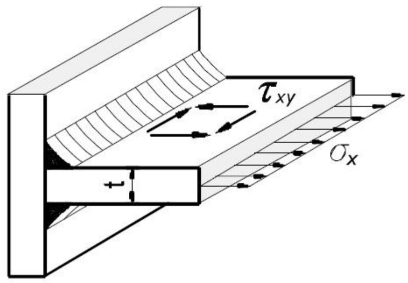


Figure 31. Stresses in plate.
(LUT Steel Structures Design – Gary B. Marquis)

Stresses in weld

$$\sigma_{\perp} = \tau_{\perp} = \frac{\sigma_x \cdot t}{2\sqrt{2} \cdot a} \text{ , (33)} \qquad \tau_{II} = \frac{\tau_{xy} \cdot t}{2 \cdot a} \text{ , (34)}$$

$$\sigma_{eq} = \sqrt{\sigma_{\perp}^2 + 3\tau_{\perp}^2 + 3\tau_{II}^2} = \frac{t}{2 \cdot a}\sqrt{2\sigma_x^2 + 3\tau_{xy}^2} \text{ , (35)}$$

The design shear strength $f_{vw,d}$ of the weld should be determined from:

$$f_{vw,d} = \frac{f_u / \sqrt{3}}{\beta_w \cdot \gamma_{M2}} \text{ , (36)}$$

where:
f_u is the nominal ultimate tensile strength
β_w is the appropriate correlation factor
γ_{M2} partial factor

Design requirement

$$f_{vw,d} \leq \sigma_{eq}, \quad (37)$$

$$f_{vw,d} \geq \frac{t}{2 \cdot a}\sqrt{2\sigma_x^2 + 3\tau_{xy}^2}, \quad (38)$$

Chart 1. Stress in the consol and in the end plate. a_{con}≥5mm, $a_{e.pl}$≥4mm

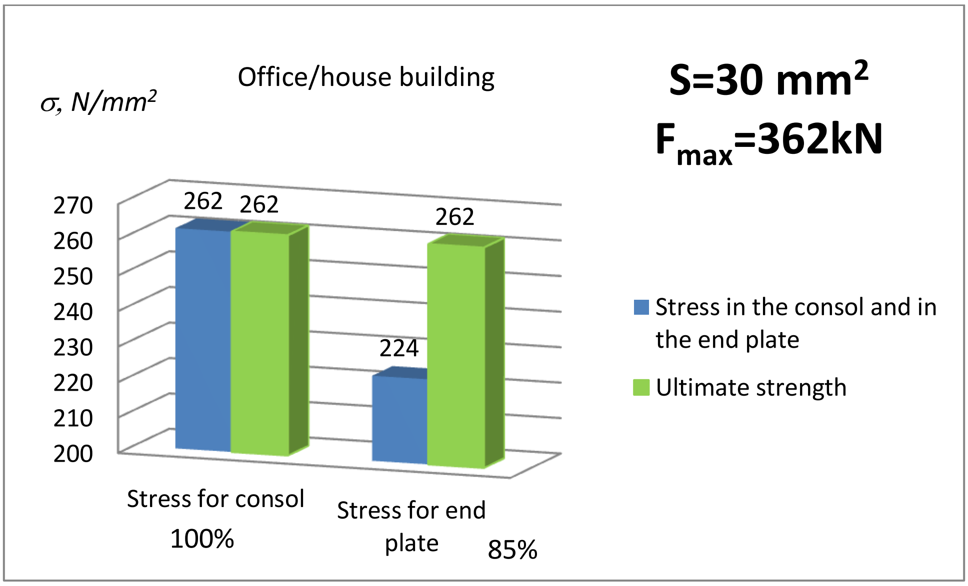


Chart 2. Stress in the consol and in the end plate. a_{con}≥7mm, $a_{e.pl}$≥5mm

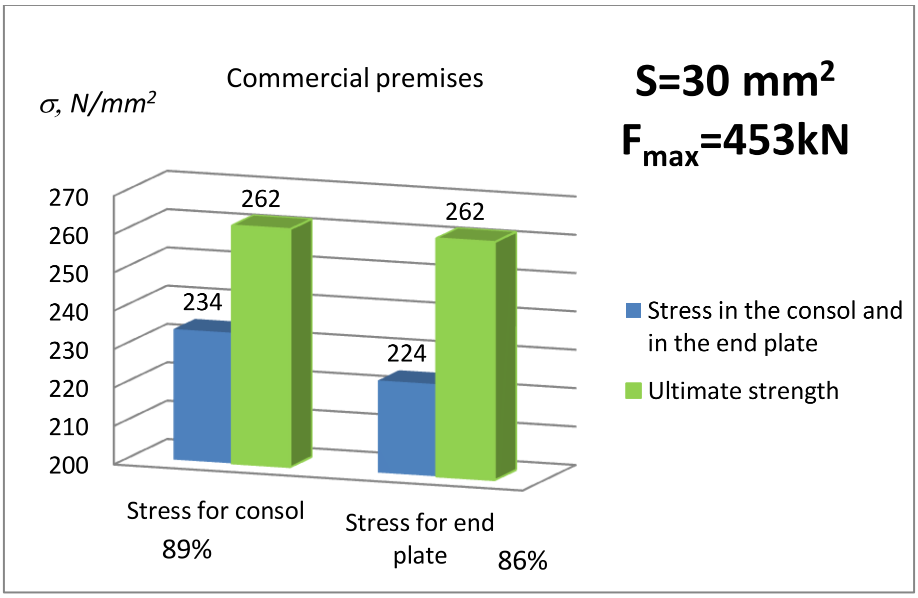


Chart 3. Stress in the consol and in the end plate. a_{con}≥6mm, $a_{e.pl}$≥4mm

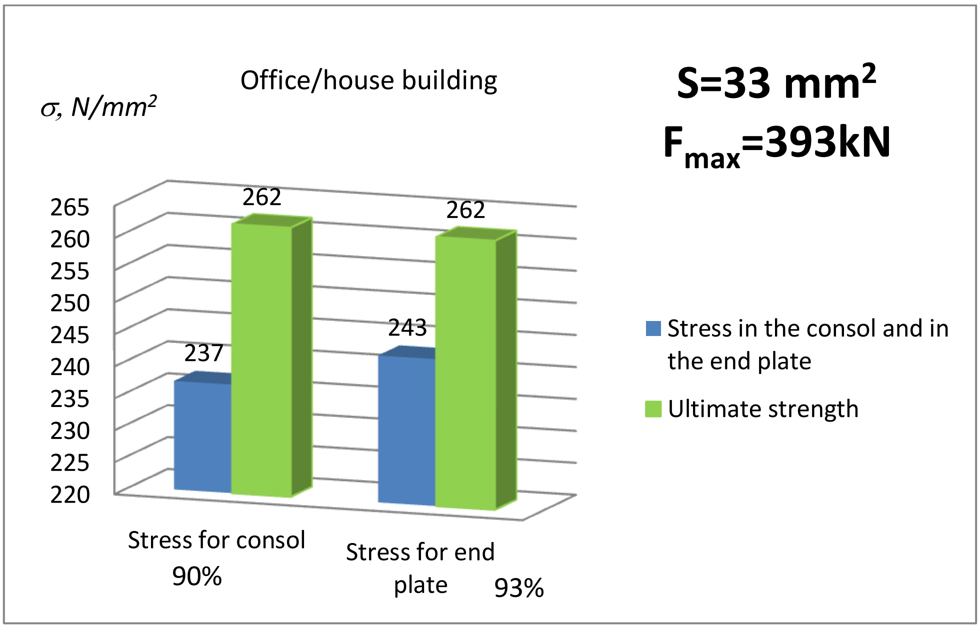
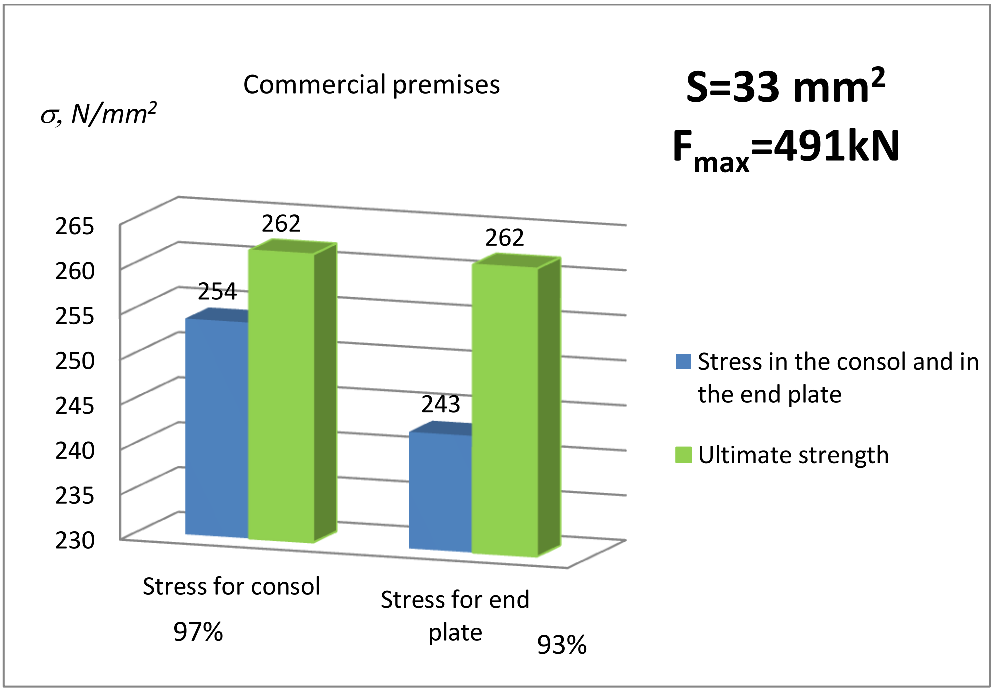


Chart 4. Stress in the consol and in the end plate. a_{con}≥7mm, $a_{e.pl}$≥5mm



The equivalent stress has to be less or equal to the design shear strength, according to EN 1993-1-8: 2005, p. 44. The formula is based on the mean stress method which considers the weld strength as being equal to the shear strength. On these charts can be seen the dependence stress on welding: the more welding is the less stress in the member.

Chart 5. Stress in the consol and in the end plate. a_{con}≥6mm, $a_{e.pl}$≥4mm

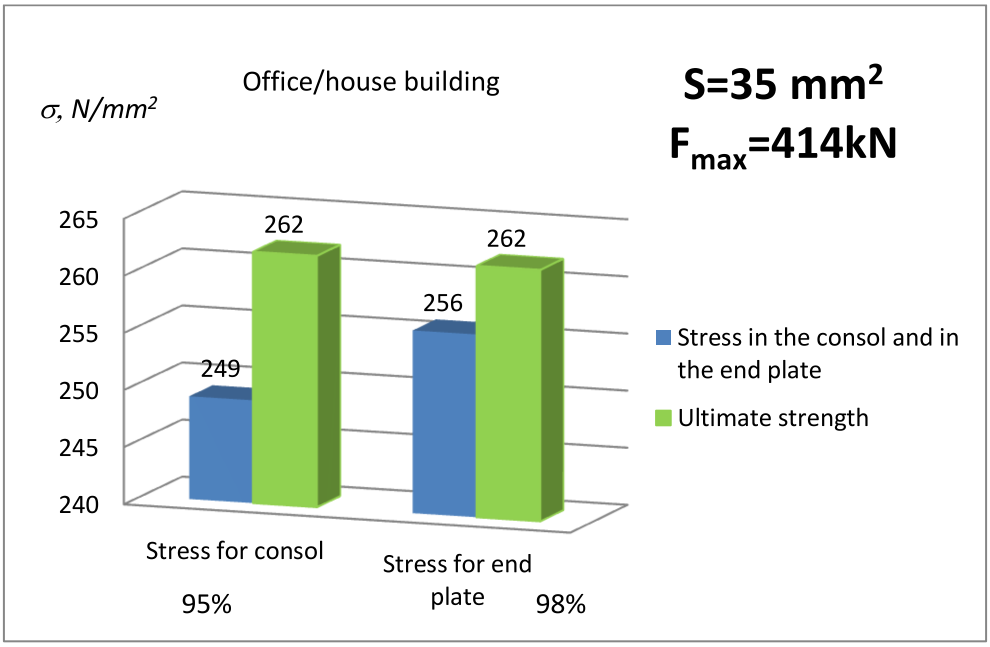
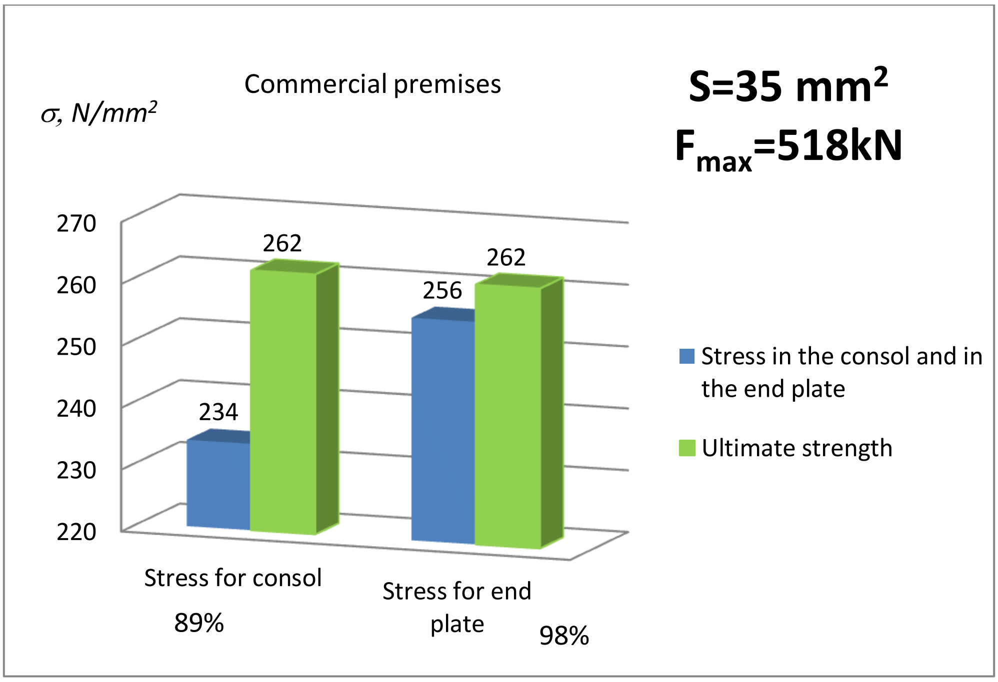


Chart 6. Stress in the consol and in the end plate. a_{con}≥8mm, $a_{e.pl}$≥5mm



Based on these charts, if in the calculations the equivalent stress is higher than the design shear strength then it needs to rise up the welding and to make calculations again until it satisfy the demands. On the other hand the welding should not be very big. It is more wiser then to change some other dimensions data in steel parts.

The required weld size

$$a \geq \frac{t}{2 \cdot f_{vw,d}} \sqrt{2\sigma_x^2 + 3\tau_{xy}^2}, \quad (39)$$

Chart 7. Stress chart with live load 3 kN/mm²

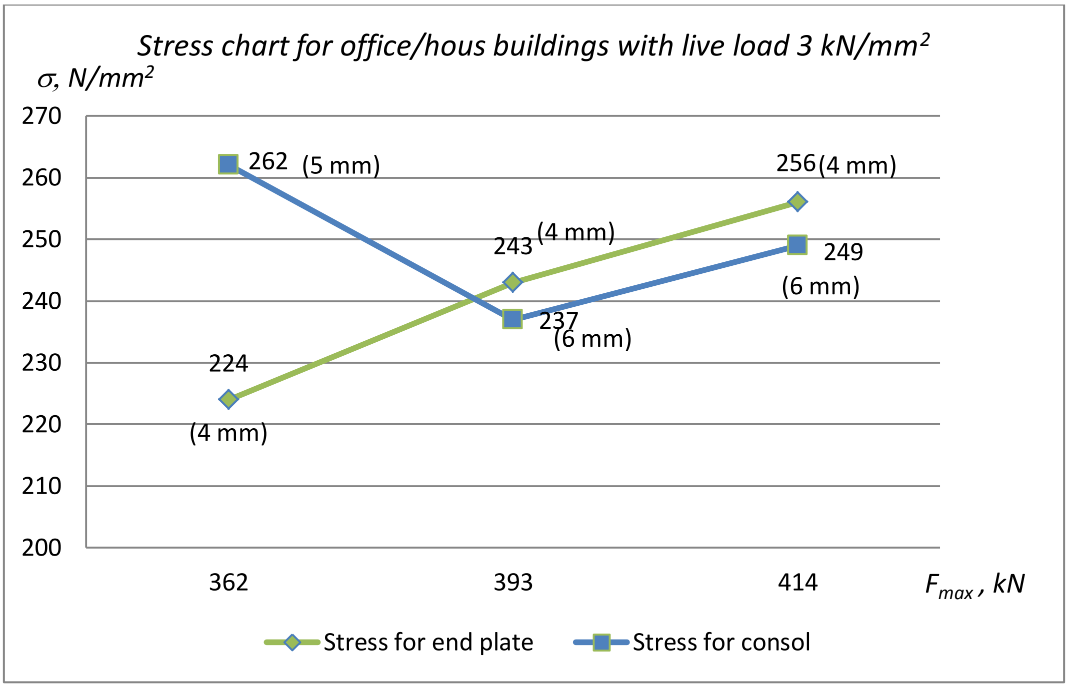
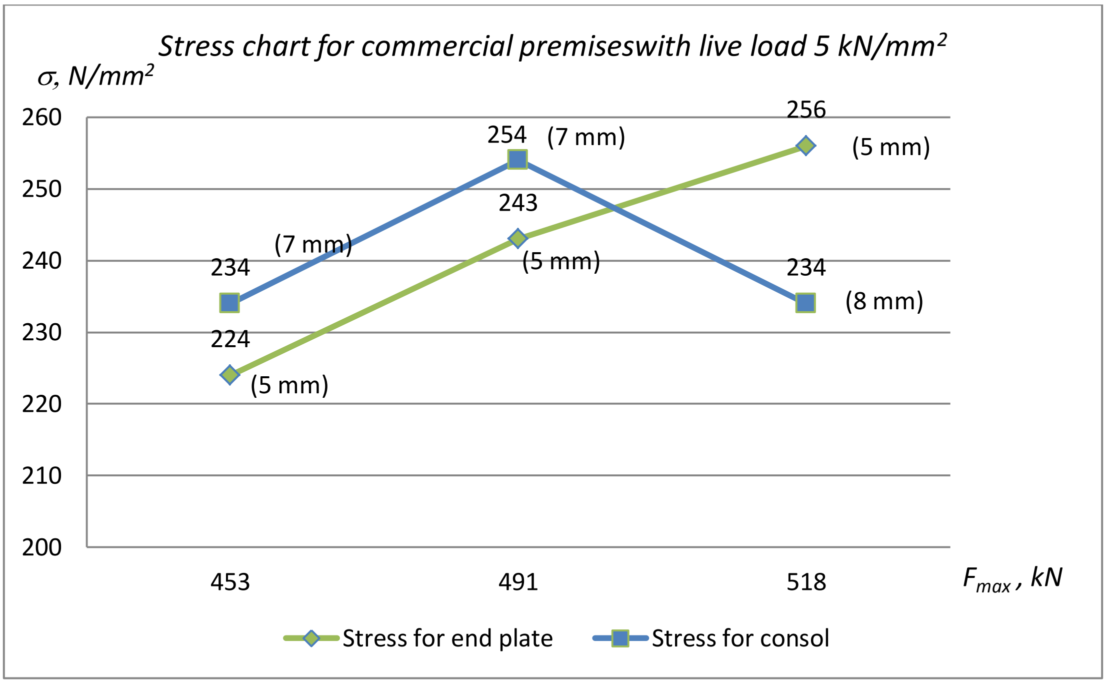


Chart 8. Stress chart with live load 5 kN/mm²



For Office/house buildings and for Commercial premises it is seen that stress in the end plate is always growing up and the welding all the way the same. In the consol stress changes because the welding has to be bigger, otherwise the equivalent stress is bigger than the design shear strength.

The design shear resistance

$$\frac{V_{Ed}}{V_{Rd}} \leq 1{,}0, \quad (40)$$

$$V_{Rd} = A \cdot \frac{f_y / \sqrt{3}}{\gamma_{M2}}, \quad (41)$$

Chart 9. Shear force in members. LL=5 kN/m², S_{load}=33m²

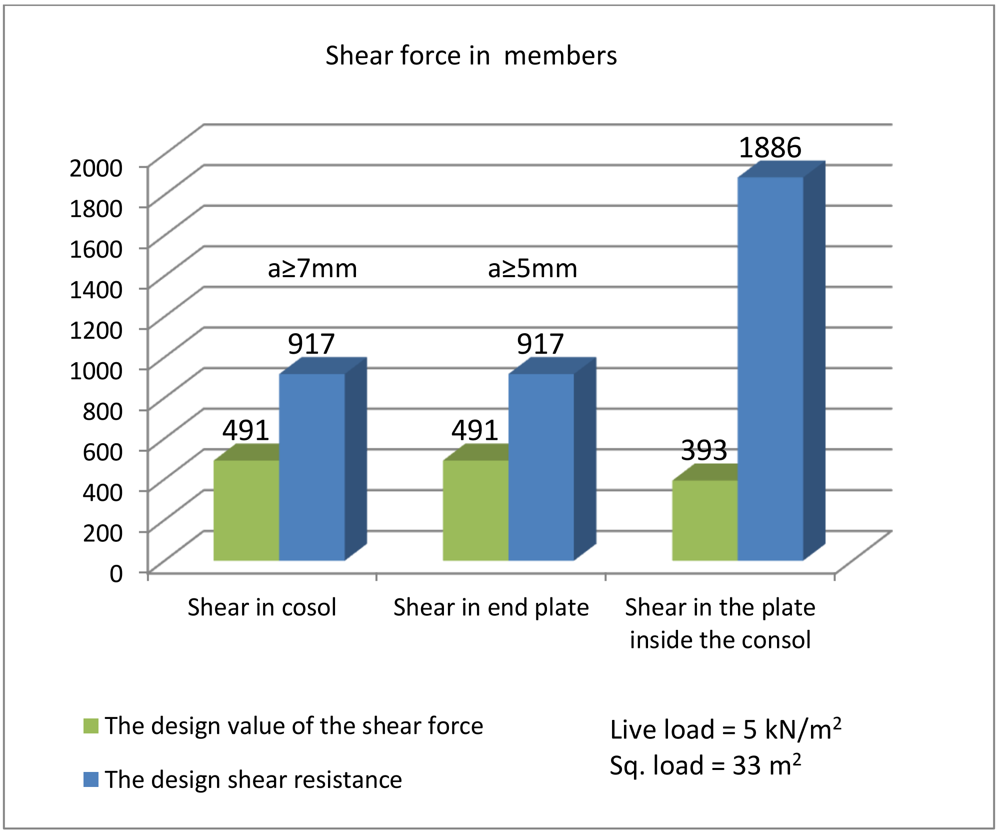


Chart 10. Shear force in members. LL=3 kN/m², S_{load}=33m²

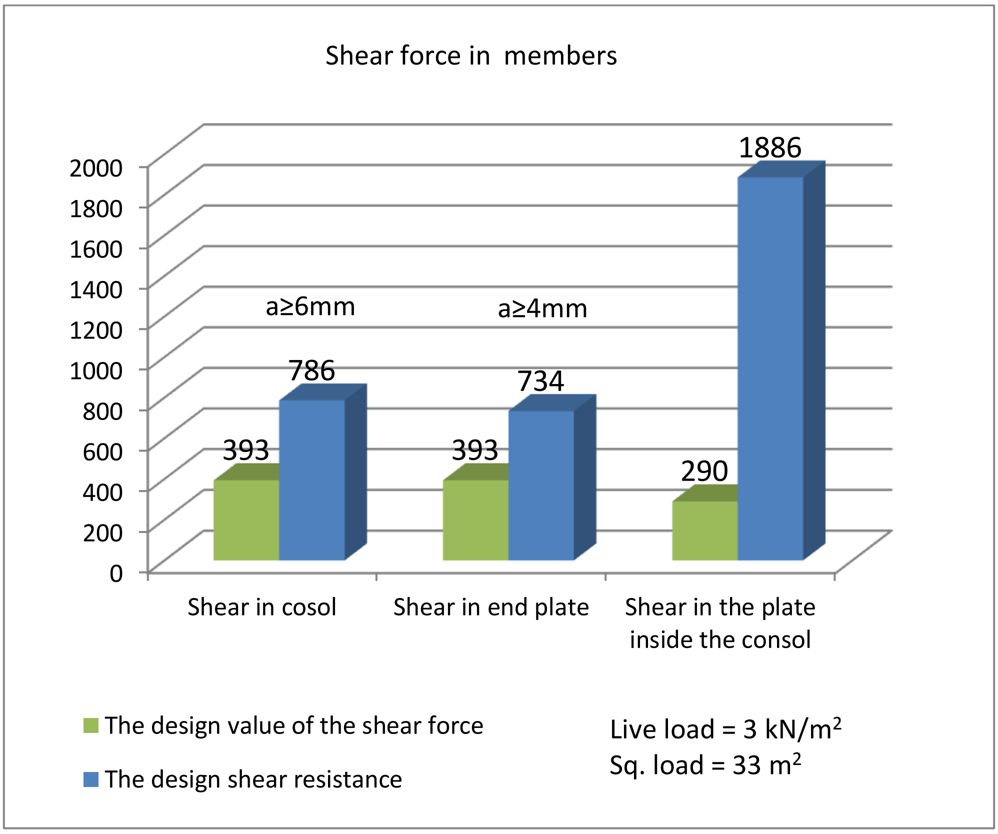
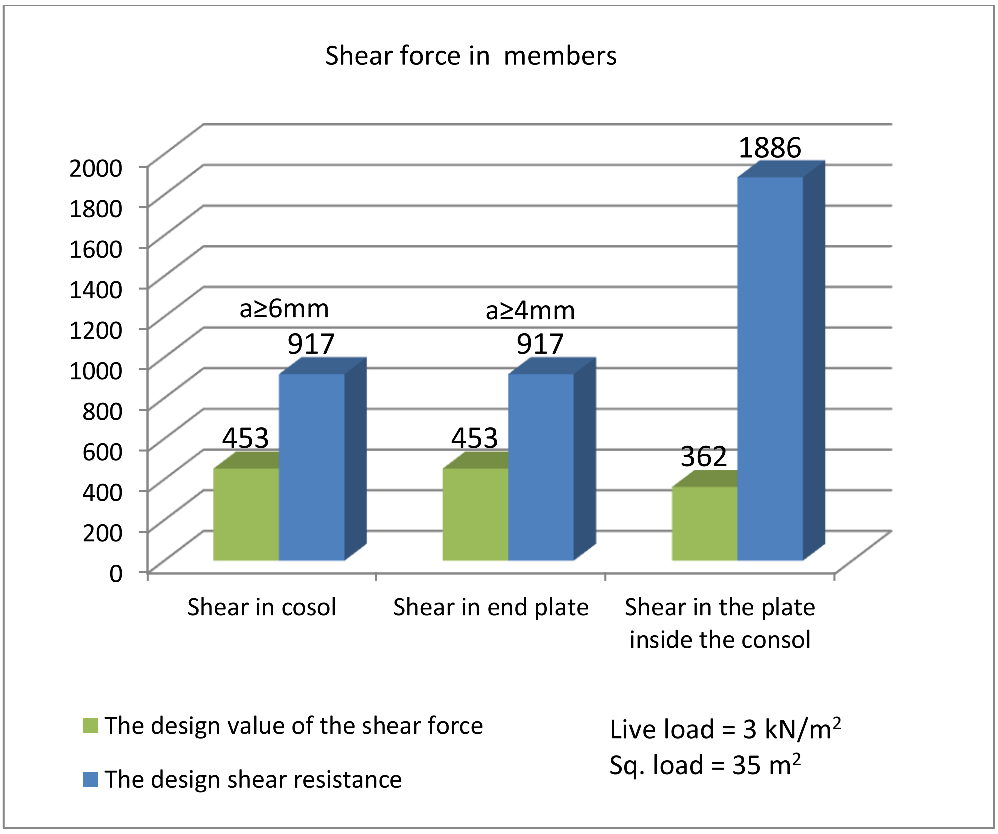


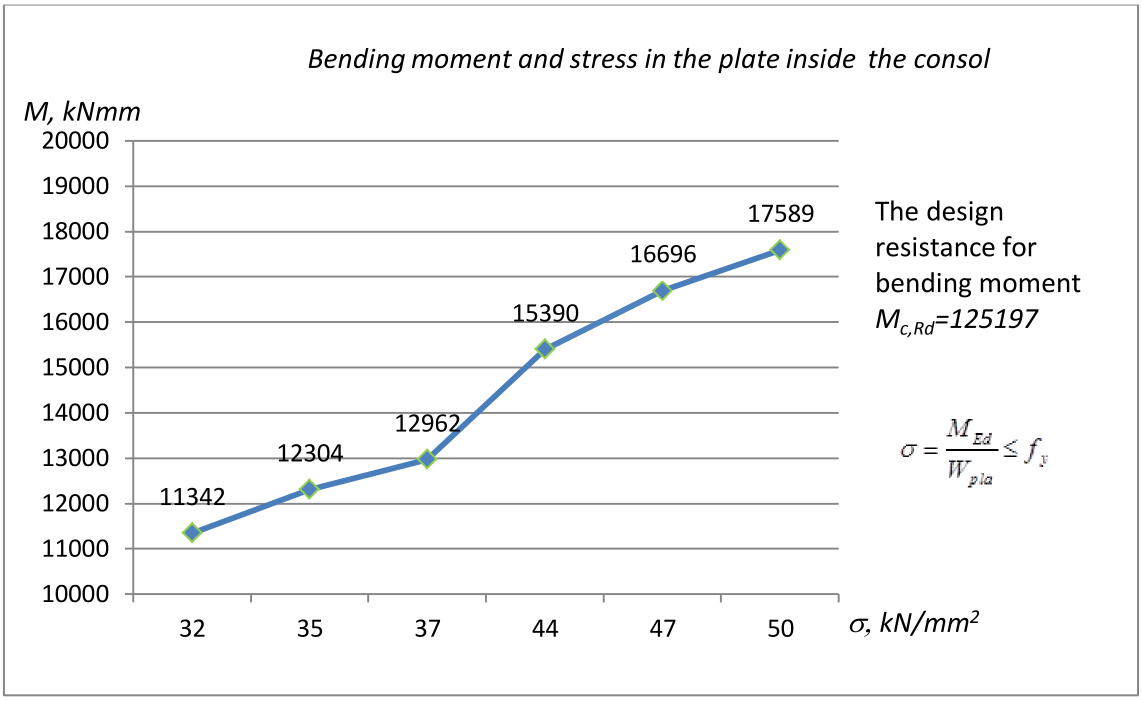
Chart 11. Shear force in members. LL=3 kN/m², S_{load}=35m²



The design shear resistance in the consol and end plate are the same(Chart 9 and Chart 11), because consider to *Formula 38* even though the stress in weld changes in both cases the welding is the same.

$$\sigma = \frac{M}{W} \leq f_y \text{, (42)}$$

Chart 12. Bending moment and stress in the plate inside the consol



The difference between the design resistance for bending moment and design bending moment is very big and the utilization is very small. It means that there is no significant bending moment that could effect on the connection.

In welded connections, and in bolted connections with end-plates, the welds connecting the beam web should be designed to transfer the shear force from the connected beam to the joint, without any assistance from the welds connecting the beam flanges. (EN 1993-1-8: 2005)

In all joints, the sizes of the welds should be such that the design moment resistance of the joint $M_{j,Rd}$ is always limited by the design resistance of its other basic components, and not by the design resistance of the welds. (EN 1993-1-8: 2005)

5 CALCULATION EXAMPLE

The calculation part consists of two examples of live load for two different types of buildings. The idea of this part is to calculate the connection between WQ-beam for hollow core slabs and hollow section column. The work is focused on making the welding between the end plate and WQ-beam, and between column and consol.

The purpose of the calculation is to make the connection between the hollow section column and WQ-beam. For this connection is needed to make the welding for end plate with WQ-beam and welding for consol. In the consol there is a plate, which works in fire case.

The calculation starts from collecting the loads: dead load, in both cases is the same, and live load, which depends on type of the building. The safety factors to calculate the maximum load are used according to Eurocode's numbers.

WQ-beam/column connection /WQ 320-6-t_2x552-30x340/

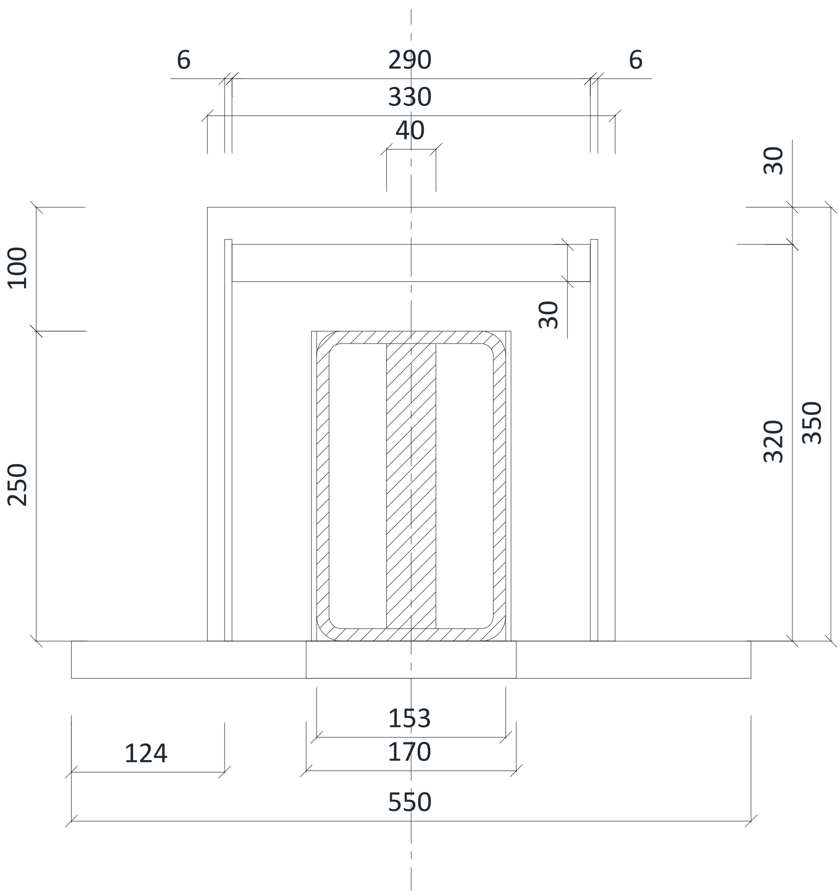


Figure 32. Cross section of the connection.

Square 8,1x8,1m.

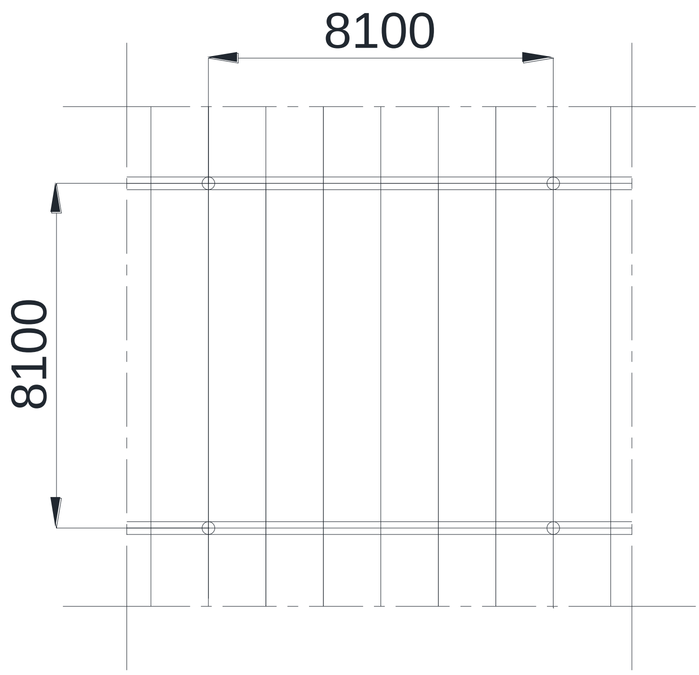
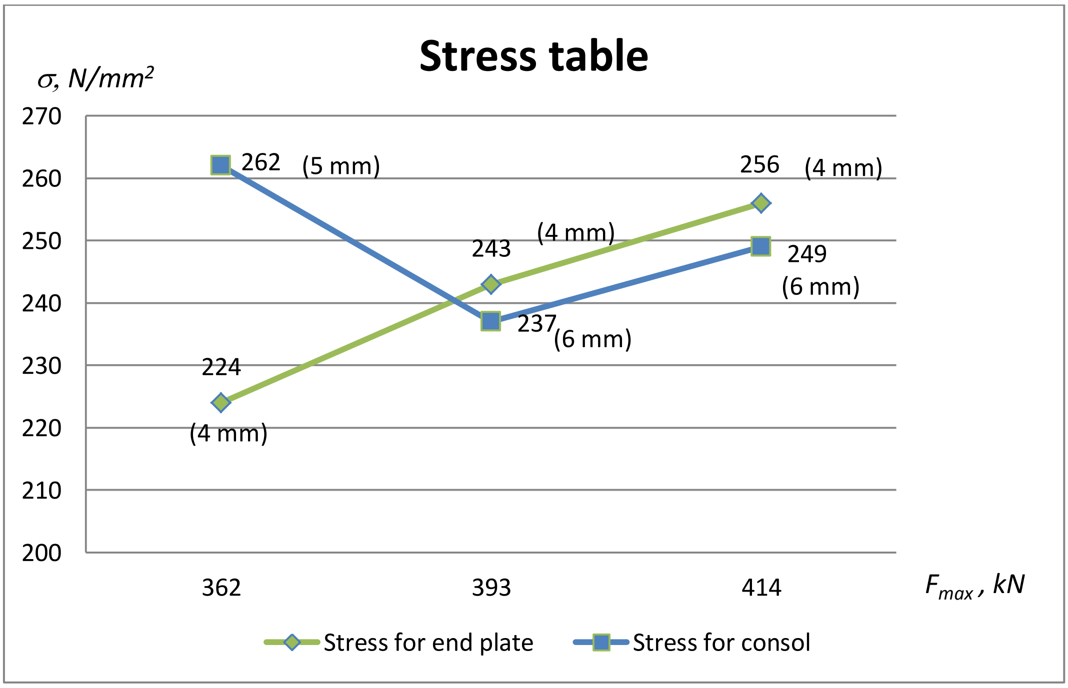


Figure 33. Plan of the building.

5.1 The first case: Calculation for office/house buildings

Chart 13. The stress in the consol and in the end plate depending on the maximum value of shear force. (In office/house buildings)



Materials:

- hollow sections:S355J2H
- plates and WQ-beams: S355J2+N

5.1.1 Initial data

Dead loads:

- Surface of cast (h=60mm) 0,6x25:	1,5 kN/m^2
- Hollow cor slab weight of sealed (h=320mm):	4,2 kN/m^2
- Walls:	0,5 kN/m^2
- Suspension:	0,2 kN/m^2
- Floor covering:	0,1 kN/m^2
Live loads:	
- For office/house building EN1991-8-1:	3 kN/m^2
Partial factors:	γ_{M0}=1,00; γ_{M2}=1,25 (Annex 1, Table 2)
The ultimate limit f_u:	510 N/mm^2 (Annex 1, Table 1)
Yield strength fy:	355 N/mm^2 (Annex 1, Table 1)
Factor ψ_2:	0,3 (Annex 1, Table 3)
Correlation factor β_w:	0,9 (Annex 1, Table 4)

5.1.2 Collecting loads:

Dead Load:

$$\sum DL' = 6{,}5kN/m^2$$

$$DL = \sum DL' \cdot S_{load} = 6{,}5kN/m^2 \cdot 33m^2 = 213kN$$

Live Load:

$$LL' = 3kN/m^2$$

$$LL = LL' \cdot S_{load} = 3kN/m^2 \cdot 33m^2 = 98kN$$

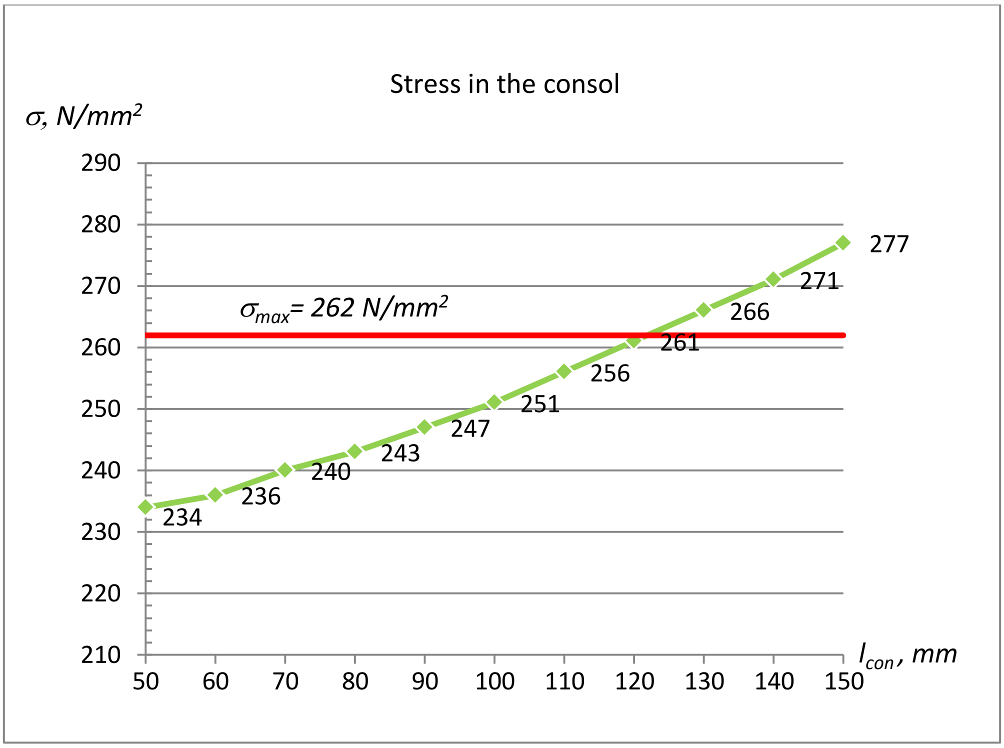
For the calculation of the maximum load it is needed to use the safety factors:

- for the dead load it is 1,15;
- for the live load it is 1,5.

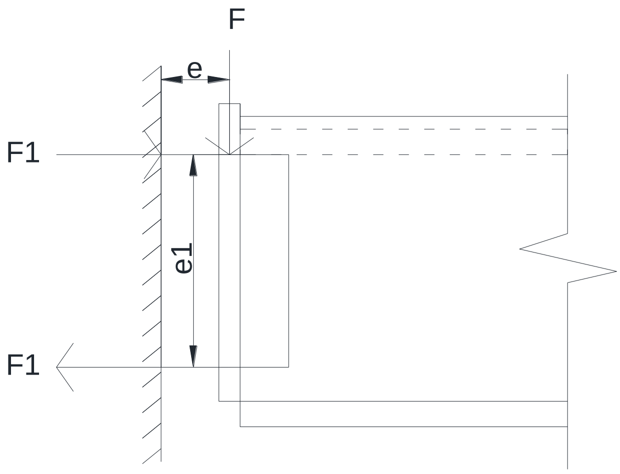
$$F_{Ed} = DL \cdot 1{,}15 + LL \cdot 1{,}5 = 245kN + 148kN = 393kN$$

5.1.3 Consol calculations

Chart 14. The stress in the consol, depending on the length of the consol. $\sigma_{vw,d}$=262N/mm^2. (In office/house buildings)

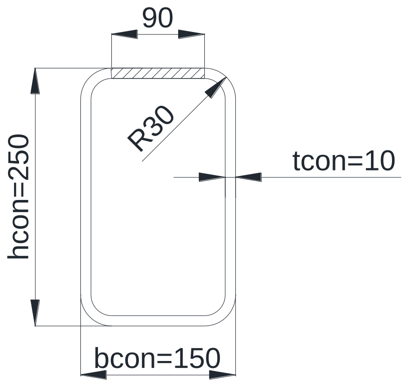


The buckling force should be calculated:



$$F_1 = \frac{e \cdot F_{Ed}}{e_1} = \frac{33mm \cdot 393kN}{250mm} = 52kN$$

Stress square in the consol:



$$A_{con} = (b_{con} - 60) \cdot t_{con} = (150 - 60) \cdot 10 = 900mm^2$$

Calculation of stresses in weld:

$$\sigma = \frac{F_1 \cdot 1000}{A_{con}} = \frac{52kN \cdot 1000}{900mm^2} = 58\frac{N}{mm^2}$$

$$\tau = \frac{F_{Ed} \cdot 1000}{h_{con} \cdot t_{con}} = \frac{393kN \cdot 1000}{250mm \cdot 10mm} = 157\frac{N}{mm^2}$$

The design shear strength of the weld (see part 4, formula 36):

$$f_{vw,d} = \frac{510/\sqrt{3}}{0{,}9 \cdot 1{,}25} = 262\frac{N}{mm^2}$$

The welding for flange (see part 4, formula 39):

$$a \geq \frac{10mm}{2 \cdot 262\frac{N}{mm^2}}\sqrt{2 \cdot 58^2 \frac{N}{mm^2}} = 1{,}6mm$$

The welding for web (see part 4, formula 39):

$$a \geq \frac{10mm}{2 \cdot 262\frac{N}{mm^2}}\sqrt{3 \cdot 157^2 \frac{N}{mm^2}} = 5{,}2mm$$

The final welding should take the maximum of both and round upward.
Thy the final welding is 6 mm for consol.

Stress in the consol:

$$\sigma_{eq} = \frac{t}{2 \cdot a} \cdot \sqrt{2\sigma_x^2 + 3\tau_{xy}^2}$$

$$\sigma_{eq} = 237N/mm^2 \leq f_{vw,d} = 262N/mm^2$$

Utilization: 90%

Shear force in the consol:

The design resistance per unit length $F_{w,Rd}$ should be determined from:

$$F_{w,Rd} = f_{vw,d} \cdot a = 786kN$$

See formula 14, part 2.10

$$F_{w,Ed} = 393kN \leq F_{w,Rd} = 786kN$$

Utilization: 50%

Bending moment in the consol:

See part 2.9.

$$M_{Ed} = F \cdot e = 393kN \cdot 33mm = 12964kN \cdot mm$$

$$M_{c,Rd} = \frac{150mm \cdot 250^2 mm^2 \cdot 355N/mm}{6 \cdot 1000} = 554688kN \cdot mm$$

$$M_{Ed} = 12964kN \cdot mm \leq M_{c,Rd} = 554688kN \cdot mm$$

Utilization: 2%

5.1.3 End plate calculations

Calculation of stress in weld:

$$\tau = \frac{F_{Ed} \cdot 1000}{h_{bep} \cdot b_{bep}} = \frac{393kN \cdot 1000}{350mm \cdot 25mm} = 45\frac{N}{mm^2}$$

The welding for web (see part 4, formula 39):

$$a \geq \frac{25mm}{2 \cdot 262\frac{N}{mm^2}} \sqrt{3 \cdot 45^2 \frac{N}{mm^2}} = 3{,}7mm$$

The final welding for end plate is 4 mm.

Stress in the end plate:

$$\sigma_{eq} = \frac{t}{2 \cdot a} \cdot \sqrt{2\sigma_x^2 + 3\tau_{xy}^2}$$

$$\sigma_{eq} = 243 N/mm^2 \leq f_{vw,d} = 262 N/mm^2$$

Utilization: 93%

Shear force in the end plate:

The design resistance per unit length $F_{w,Rd}$ should be determined from:

$$F_{w,Rd} = f_{vw,d} \cdot a = 734 kN$$

See formula 14, part 2.10

$$F_{w,Ed} = 393 kN \leq F_{w,Rd} = 734 kN$$

Utilization: 54%

5.1.3 Plate calculations

The design value of shear force should be determined from:

$$V_{Ed} = DL + LL \cdot \psi_2 = 245 kN + 148 kN \cdot 0{,}3 = 290 kN$$

The design share resistance in the plate should be determined from formula 15 in part 2.10:

$$V_{Rd} = \frac{b_{pl} \cdot h_{pl} \cdot f_y / \sqrt{3}}{1000 \cdot \gamma_{M0}} = \frac{40mm \cdot 230mm \cdot 355 N/mm^2}{1{,}73 \cdot 1000 \cdot 1} = 1886 kN$$

See part 2.10, formula 14:

$$V_{Ed} = 290 kN \leq V_{Rd} = 1886 kN$$

Utilization: 15%

The design value of the bending moment should be determined:

$$M_{pl} = V_{Ed} \cdot (e + t_{col}) = 290 kN \cdot (33mm + 10mm) = 12449 kN \cdot mm$$

The design resistance for bending moment should be determined form formula 10, part 2.9:

$$M_{c,Rd} = \frac{40mm \cdot 230^2 mm^2 \cdot 355 N/mm}{6 \cdot 1000} = 125197 kN \cdot mm$$

See part 2.9, formula 9:

$$M_{Ed} = 12449 kN \cdot mm \le M_{c,Rd} = 125197 kN \cdot mm$$

Utilization: 10%

Stress in the plate should be determined from formula 42, part 4:

$$\sigma = \frac{M_{pl} \cdot 1000}{W_{pl}} = \frac{12449(kN \cdot mm) \cdot 1000}{352667 mm^3} = 35 \frac{N}{mm^2} \le f_y = 355 \frac{N}{mm^2}$$

$$W_{pl} = \frac{b_{pl} \cdot {h_{pl}}^2}{6} = \frac{40mm \cdot 230^2 mm^2}{6} = 352667 mm^3$$

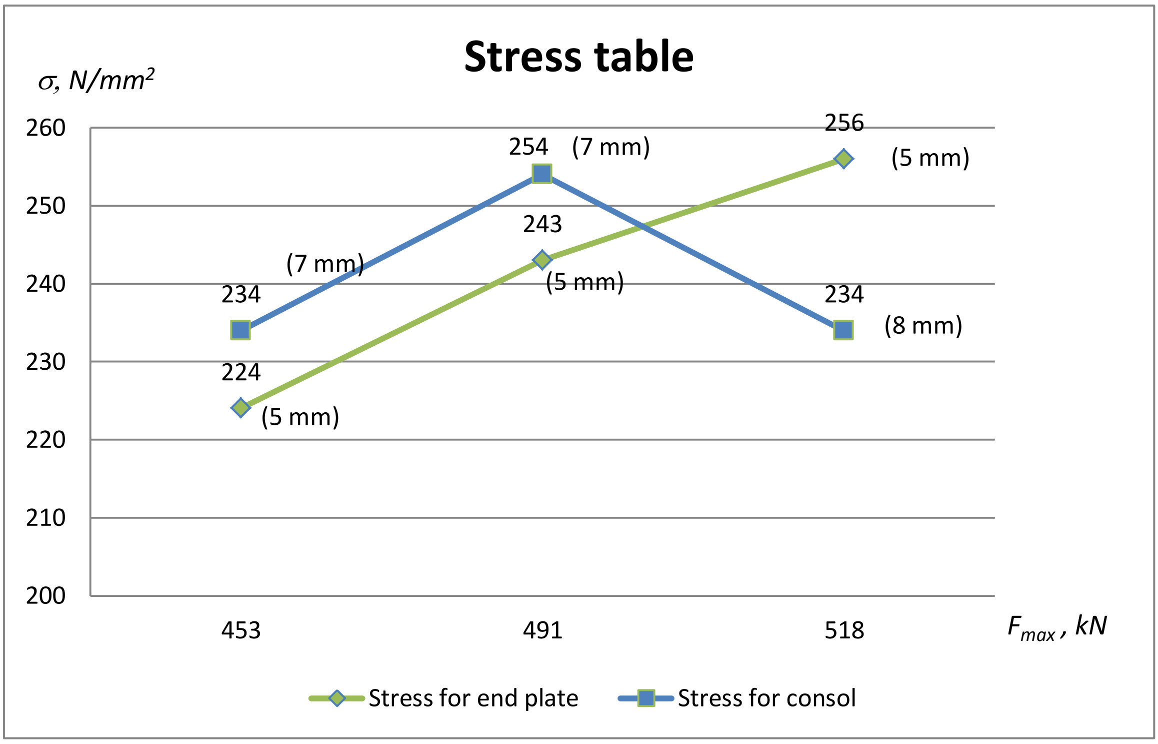
Utilization: 10%

5.1.4 The results

Stress in the consol	237	N/mm^2	90%
Shear force in the consol	786	kN	50%
Bending moment in the consol	554688	kNmm	2%
Stress in the end plate	243	N/mm^2	93%
Shear force in the end plate	734	kN	54%
Shear force in the plate	1886	kN	15%
Bending moment in the plate	125197	kNmm	10%
Stress in the plate	35	N/mm^2	10%

5.2 The second case: Calculation for commercial premises

Chart 15. The stress in the consol and in the end plate
depending on the maximum value of shear force. (In commercial premises)



Materials:

- hollow sections:S355J2H
- plates and WQ-beams: S355J2+N

<u>5.2.1 Initial data</u>

<u>Dead loads:</u>

- Surface of cast (h=60mm) 0,6x25:	1,5 kN/m^2
- Hollow cor slab weight of sealed (h=320mm):	4,2 kN/m^2
- Walls:	0,5 kN/m^2
- Suspension:	0,2 kN/m^2
- Floor covering:	0,1 kN/m^2

<u>Live loads:</u>

- For commercial premises EN1991-8-1:	5 kN/m^2
Partial factors:	γ_{M0}=1,00; γ_{M2}=1,25 (Annex 1, Table 2)
The ultimate limit f_u:	510 N/mm^2 (Annex 1, Table 1)
Yield strength fy:	355 N/mm^2 (Annex 1, Table 1)
Factor ψ_2:	0,6 (Annex 1, Table 3)
Correlation factor β_w:	0,9 (Annex 1, Table 4)

5.2.2 Collecting loads

Dead Load:

$$\sum DL' = 6{,}5kN/m^2$$

$$DL = \sum DL' \cdot S_{load} = 6{,}5kN/m^2 \cdot 33m^2 = 213kN$$

Live Load:

$$LL' = 5kN/m^2$$

$$LL = LL' \cdot S_{load} = 5kN/m^2 \cdot 33m^2 = 98kN$$

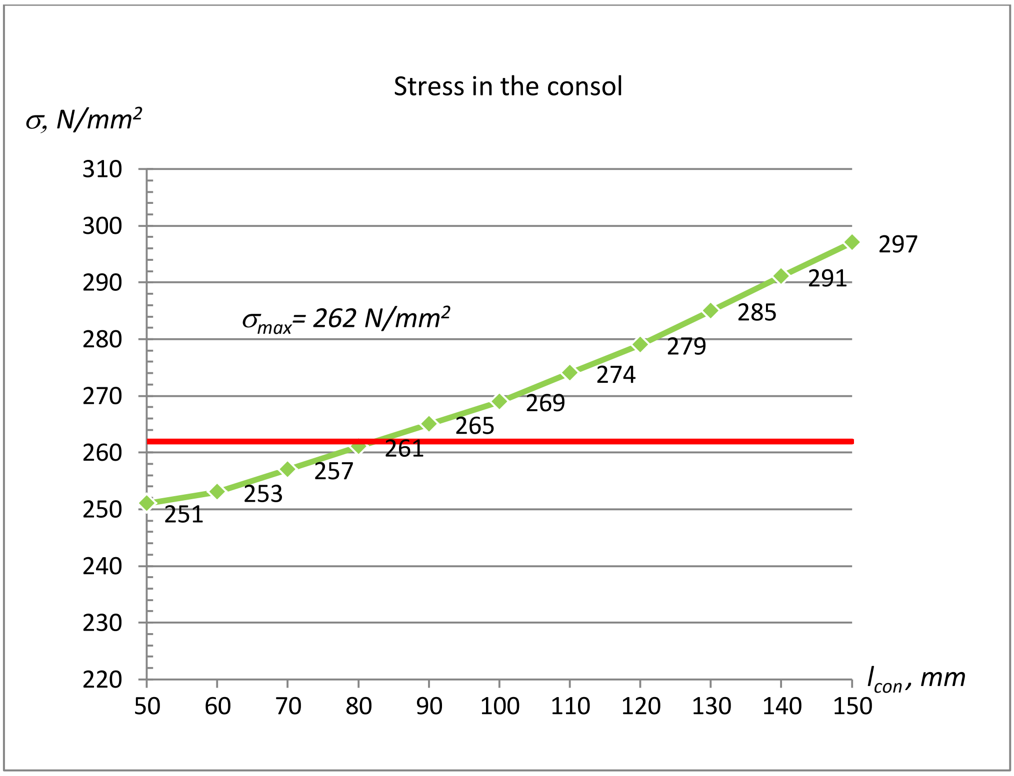
For the calculation of the maximum load it is needed to use the safety factors:

- for the dead load it is 1,15;
- for the live load it is 1,5.

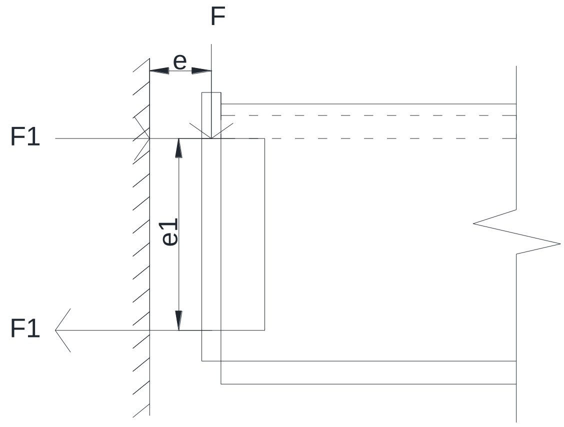
$$F_{Ed} = DL \cdot 1{,}15 + LL \cdot 1{,}5 = 245kN + 246kN = 491kN$$

5.2.3 Consol calculations

Chart 16. The stress in the consol, depending on the length of the consol. $\sigma_{vw,d}$=262N/mm². (In office/house buildings)

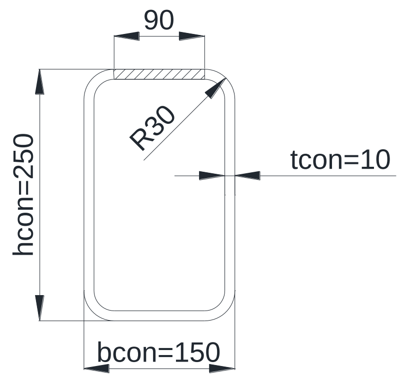


The buckling force should be calculated:



$$F_1 = \frac{e \cdot F_{Ed}}{e_1} = \frac{33mm \cdot 491kN}{250mm} = 65kN$$

Stress square in the consol:



$$A_{con} = (b_{con} - 60) \cdot t_{con} = (150 - 60) \cdot 10 = 900mm^2$$

Calculation of stresses in weld:

$$\sigma = \frac{F_1 \cdot 1000}{A_{con}} = \frac{65kN \cdot 1000}{900mm^2} = 72\frac{N}{mm^2}$$

$$\tau = \frac{F_{Ed} \cdot 1000}{h_{con} \cdot t_{con}} = \frac{491kN \cdot 1000}{250mm \cdot 10mm} = 197\frac{N}{mm^2}$$

The design shear strength of the weld (see part 4, formula 36):

$$f_{vw,d} = \frac{510/\sqrt{3}}{0,9 \cdot 1,25} = 262\frac{N}{mm^2}$$

The welding for flange (see part 4, formula 39):

$$a \geq \frac{10mm}{2 \cdot 262 \frac{N}{mm^2}} \sqrt{2 \cdot 72^2 \frac{N}{mm^2}} = 1{,}9mm$$

The welding for web (see part 4, formula 39):

$$a \geq \frac{10mm}{2 \cdot 262 \frac{N}{mm^2}} \sqrt{3 \cdot 197^2 \frac{N}{mm^2}} = 6{,}5mm$$

The final welding should take the maximum of both and round upward.
Thy the final welding is 7 mm for consol.

Stress in the consol:

$$\sigma_{eq} = \frac{t}{2 \cdot a} \cdot \sqrt{2\sigma_x^2 + 3\tau_{xy}^2}$$

$$\sigma_{eq} = 254N/mm^2 \leq f_{vw,d} = 262N/mm^2$$

Utilization: 97%

Shear force in the consol:

The design resistance per unit length $F_{w,Rd}$ should be determined from:

$$F_{w,Rd} = f_{vw,d} \cdot a = 917kN$$

See formula 14, part 2.10

$$F_{w,Ed} = 491kN \leq F_{w,Rd} = 917kN$$

Utilization: 54%

Bending moment in the consol:

See part 2.9.

$$M_{Ed} = F \cdot e = 491kN \cdot 33mm = 16211kN \cdot mm$$

$$M_{c,Rd} = \frac{150mm \cdot 250^2 mm^2 \cdot 355N/mm}{6 \cdot 1000} = 554688kN \cdot mm$$

$$M_{Ed} = 16211kN \cdot mm \leq M_{c,Rd} = 554688kN \cdot mm$$

Utilization: 3%

5.2.3 End plate calculations

Calculation of stress in weld:

$$\tau = \frac{F_{Ed} \cdot 1000}{h_{bep} \cdot b_{bep}} = \frac{491kN \cdot 1000}{350mm \cdot 25mm} = 56\frac{N}{mm^2}$$

The welding for web (see part 4, formula 39):

$$a \geq \frac{25mm}{2 \cdot 262\frac{N}{mm^2}} \sqrt{3 \cdot 56^2 \frac{N}{mm^2}} = 4{,}6mm$$

The final welding for end plate is 5 mm.

Stress in the end plate:

$$\sigma_{eq} = \frac{t}{2 \cdot a} \cdot \sqrt{2\sigma_x^2 + 3\tau_{xy}^2}$$

$$\sigma_{eq} = 243N/mm^2 \leq f_{vw,d} = 262N/mm^2$$

Utilization: 93%

Shear force in the end plate:

The design resistance per unit length $F_{w,Rd}$ should be determined from:

$$F_{w,Rd} = f_{vw,d} \cdot a = 917kN$$

See formula 14, part 2.10

$$F_{w,Ed} = 491kN \leq F_{w,Rd} = 917kN$$

Utilization: 54%

5.2.3 Plate calculations

The design value of shear force should be determined from:

$$V_{Ed} = DL + LL \cdot \psi_2 = 245kN + 246kN \cdot 0{,}3 = 393kN$$

The design share resistance in the plate should be determined from formula 15 in part 2.10:

$$V_{Rd} = \frac{b_{pl} \cdot h_{pl} \cdot f_y / \sqrt{3}}{1000 \cdot \gamma_{M0}} = \frac{40mm \cdot 230mm \cdot 355N/mm^2}{1{,}73 \cdot 1000 \cdot 1} = 1886kN$$

See part 2.10, formula 14:

$$V_{Ed} = 393kN \leq V_{Rd} = 1886kN$$

Utilization: 21%

The design value of the bending moment should be determined:

$$M_{pl} = V_{Ed} \cdot (e + t_{col}) = 393kN \cdot (33mm + 10mm) = 16892kN \cdot mm$$

The design resistance for bending moment should be determined form formula 10, part 2.9:

$$M_{c,Rd} = \frac{40mm \cdot 230^2 mm^2 \cdot 355N/mm}{6 \cdot 1000} = 125197kN \cdot mm$$

See part 2.9, formula 9:

$$M_{Ed} = 16892kN \cdot mm \leq M_{c,Rd} = 125197kN \cdot mm$$

Utilization: 13%

Stress in the plate should be determined from formula 42, part 4:

$$\sigma = \frac{M_{pl} \cdot 1000}{W_{pl}} = \frac{16892(kN \cdot mm) \cdot 1000}{352667mm^3} = 48\frac{N}{mm^2} \leq f_y = 355\frac{N}{mm^2}$$

$$W_{pl} = \frac{b_{pl} \cdot h_{pl}^{\ 2}}{6} = \frac{40mm \cdot 230^2 mm^2}{6} = 352667mm^3$$

Utilization: 13%

5.2.4 The results

Stress in the consol	254	N/mm^2	97%
Shear force in the consol	917	kN	54%
Bending moment in the consol	554688	kNmm	3%
Stress in the end plate	243	N/mm^2	93%
Shear force in the end plate	917	kN	54%
Shear force in the plate	1886	kN	21%
Bending moment in the plate	125197	kNmm	13%
Stress in the plate	48	N/mm^2	13%

6 CONCLUSION

Connection can be the most complex part of the structural engineering of a project, besides figuring out how to physically weld or bolt the various stiffening plates and bracing.

Steel columns and other framing components make up the structural skeleton of homes and businesses. These columns support walls and ceilings, and give the building its basic shape. Engineers and builders must choose from different types of steel columns based on budget, durability and aesthetic appeal. In many applications, the column design also reflects local building code requirements aimed at creating a safe and stable structure.

Steel beams can be connected to each other in the traditional way, with plates and rivets welded into the faces of the linked beams. They can also be linked by heavy clamps, which are attached with plates and a set of heavy bolts.

Even if bolted connections to hollow sections are utilised to assemble prefabricated elements or space structures, the most used method to assemble CHS members is welding.

In this thesis I dealt with the connection between WQ-beam and column. At first I went through the history of the Eurocodes, I presented the basis of design and requirements of steel structures. Also I described the beam to column connection and the engineering overall. And at the same time I considered the WQ-beam to column connection.

I was concerned to do the welding for the end plate to WQ-beam, the welding between the column and consol. And I had to make the calculations for the plate inside the consol for the fire case.

The connection includes the simplicity of design and further use. As well I made the drawings such as dimensions of the connection.

The purpose was to do the standard connections (see Appendix 3) and make the connections clear in accordance with the design of the Eurocode. In the annexed model of the standard card the numerical values are hidden. At the same time I learned better how to read the standards and to search for information in them. All this information I had to apply in my work.

I delivered to Aaro Kohonen Oy the Excel calculations of connection design. Another design card is attached to my thesis without numeric values (see Appendix 2).

In addition, even if the Eurocodes are very difficult to follow in some places and the necessary information can be distributed among a number of different sections of the standard, the Eurocode does not look very difficult in the details and the information is required to integrate and apply.

FIGURES

TABLES

CHARTS

REFERENCES

Eurocode help desk. http://www.eurocodes.fi (Read 02/07/2011)

SFS-EN 1090-2. Steel structures and aluminum structures. Part 2: Steel structures, the technical requirements.

SFS-EN 1993-1-1. Eurocode 3: Design of steel structures. Part 1-1: General rules and rules for buildings

SFS-EN 1993-1-5. Eurocode 3: Design of steel structures. Part 1-5: Plated structural elements.

SFS-EN 1993-1-8. Eurocode 3: Design of steel structures. Part 1-8: Design of joints

SFS-EN 1990 Eurocode: Basis of structural design

Gary B. Marquis - Design of Welded Structures (LUT Steel Structures Design)

http://people.fsv.cvut.cz/~wald/CESTRUCO/Texts_of_lessons/10_GB_Hollow_Section_Joints.pdf

http://ec.europa.eu/enterprise/sectors/construction/eurocodes/index_en.htm

http://www.irccbuildingregulations.org/pdf/4-04.pdf

http://www.eurocodes-online.com/index.php/en_US/en/about-the-eurocodes/national-annexes

http://www.ehow.com/about_5701590_types-steel-beams-connections.html

http://www.efunda.com

http://units.civil.uwa.edu.au/teaching/CIVL4111?f=275134

http://www.ruukki.com/Products-and-solutions/Building-solutions/Steel-frame-structures/CWQ-beam-system

http://nptel.iitm.ac.in/courses/IIT-MADRAS/Design_Steel_Structures_I/5_compression/8_types_of_column_sections.pdf

http://www.indiantubecompany.com/metallic-hollow-section.html

http://people.fsv.cvut.cz/~wald/CESTRUCO/Texts_of_lessons/10_GB_Hollow_Section_Joints.pdf

http://www.greyhawk.com/news/technical/Design_Structural_Steel_Design_and_Construction.pdf

Table 1. *Nominal values of yield strength f_y and ultimate tensile strength f_u for hot rolled structural steel. (SFS-EN 1993-1-3)*

Standard and steel grade	Nominal thickness of the element t [mm]			
	$t \leq 40$ mm		$40 \text{ mm} < t \leq 80$ mm	
	f_y [N/mm²]	f_u [N/mm²]	f_y [N/mm²]	f_u [N/mm²]
EN 10025-2				
S 235	235	360	215	360
S 275	275	430	255	410
S 355	355	510	335	470
S 450	440	550	410	550
EN 10025-3				
S 275 N/NL	275	390	255	370
S 355 N/NL	355	490	335	470
S 420 N/NL	420	520	390	520
S 460 N/NL	460	540	430	540
EN 10025-4				
S 275 M/ML	275	370	255	360
S 355 M/ML	355	470	335	450
S 420 M/ML	420	520	390	500
S 460 M/ML	460	540	430	530
EN 10025-5				
S 235 W	235	360	215	340
S 355 W	355	510	335	490
EN 10025-6				
S 460 Q/QL/QL1	460	570	440	550

Table 2. *Partial factors γ_{Mi} for buildings. (SFS-EN 1993-1-3)*

$$\gamma_{M0} = 1{,}00$$

$$\gamma_{M1} = 1{,}00$$

$$\gamma_{M2} = 1{,}25.$$

Table 3. *Recommended values of ψ factors for buildings. (SFS-EN 1990)*

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

Table 4. *Correlation factor β_w for fillet welds. (SFS-EN 1993-1-8)*

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

Company		Project number		Page
Aaro Kohonen FMC GROUP		01-0300-		1(6)
		Date	Factor	
		23.05.2012	LiSe	
Designer		Contents		
Lidia Sergeeva		Connection between WQ-beam and consol		
Welding: $a_{con} \geq$ *mm*				
$a_{e.pla} \geq$ *mm*	$F_{max} =$ *kN*			

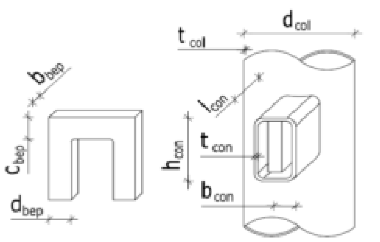
1. Data

1.1 Column

Column diameter	d_{col}=		mm
Column thickness	t_{col}=		mm

1.2 Consol

Consol length	l_{con}=		mm
Consol high	h_{con}=		mm
Consol breadth	b_{con}=		mm
Consol thickness	t_{con}=		mm



1.3 Plate

Plate length	l_{pla}=		mm
Plate high	h_{pla}=		mm
Plate breadth	b_{pla}=		mm

1.4 End plate

End plate breadth	b_{bep}=		mm
Height of the console on top	c_{bep}=		mm
End plate high	h_{bep}=		mm
Width of the console side	d_{bep}=		mm

2. Nominal values of strength

Yield strength	f_y=	355	N/mm^2	S355
Ultimate tensile strength	f_u=	510	N/mm^2	

EN 1993-1-8 : 2005, p. 44:

The design shear strength $f_{vw.d}$ of the weld should be determined from:

$$f_{vw,d} = \frac{f_u / \sqrt{3}}{\beta_w \cdot \gamma_{M2}}$$

where:

f_u is the nominal ultimate tensile strength

β_w is the appropriate correlation factor

γ_{M2} partial factor (1,25)

β_w =	0,9	
$f_{vw,d}$=	262	N/mm^2

Company	Project number		Page
Aaro Kohonen FMC GROUP	01-0300-		2(6)
	Date	**Factor**	
	23.05.2012	LiSe	
Designer	**Contents**		
Lidia Sergeeva	Connection between WQ-beam and consol		

Welding: $a_{con} \geq$ *mm*

$a_{e.pla} \geq$ *mm* $F_{max} =$ *kN*

3. Collecting loads

3.1 Dead loads

Surface of cast h= mm x2 kN/m^2

Hollow cor slab weight of sealed h= mm kN/m^2

Walls kN/m^2

Suspension kN/m^2

Floor covering kN/m^2

ΣDL= kN/m^2

3.2 Live loads EN1991-8-1

Office/house building LL= 3,0 kN/m^2

Factor for quasi-permanent value of a variable action ψ_2= 0,3

ΣDL+LL= kN/m^2

Column space 8,1x8,1 A= 8100 mm

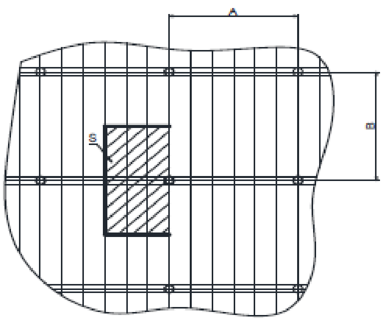
Span B= 8100 mm

Square load on one consol S= 33 m^2

EN 1990:2001:

Safety factors: for Dead Load use 1,15

for Live Load use 1,5



DL= kN

LL= kN

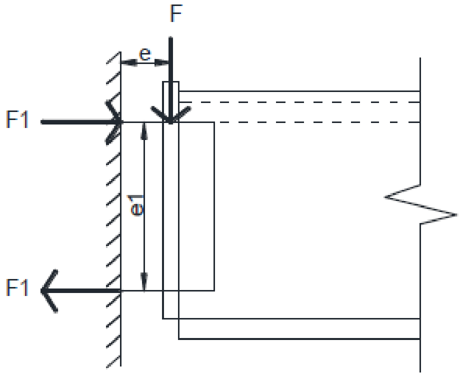
ΣDL+LL= kN

Company	Project number	Page
Aaro Kohonen FMC GROUP	01-0300-	3(6)
	Date: 23.05.2012 / Factor: LiSe	
Designer	**Contents**	
Lidia Sergeeva *Welding:* $a_{con} \geq$ mm $a_{e.pla} \geq$ mm $F_{max} =$ kN	Connection between WQ-beam and consol	

4. Calculations

4.1. Consol calculations

4.1.1. Welding calculation for the Consol



F= (kN)

e= (mm)

e_1= (mm)

F_1= (kN)

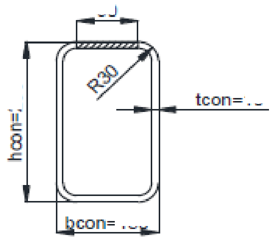
Stress square

$A_{con} = (b_{con} - 60) \cdot t_{con} =$ mm^2

Srtess in weld

$\sigma = F_1 / A_{con} =$ (N/mm^2)

$\tau = F / (h_{con} \cdot t_{con}) =$ (N/mm^2)



EN 1993-1-8 : 2005, p. 44:

Welding $f_{vw,d} \geq \sigma_{eq}$

Design requirement $f_{vw,d} \geq \frac{t}{2 \cdot a} \cdot \sqrt{2\sigma_x^2 + 3\tau_{xy}^2}$ $a \geq \frac{t}{2 \cdot f_{vw,d}} \cdot \sqrt{2\sigma_x^2 + 3\tau_{xy}^2}$

for flange

$a \geq \frac{t_{con}}{2 \cdot f_{vw,d}} \cdot \sqrt{2\sigma_x^2} \geq$ mm

for web

$a \geq \frac{t_{con}}{2 \cdot f_{vw,d}} \cdot \sqrt{3\tau_{xy}^2} \geq$ mm

The final welding for consol mm

4.1.2 Stress in the consol

$$\sigma_{eq} = \frac{t}{2 \cdot a} \cdot \sqrt{2\sigma_x^2 + 3\tau_{xy}^2}$$

$$\sigma_{eq} \leq f_{vw,d}$$

$\leq$

Utilizaition: %

Company		Project number		Page
Aaro Kohonen FMC GROUP		01-0300-		4(6)
		Date	Factor	
		23.05.2012	LiSe	
Designer		Contents		
Lidia Sergeeva		Connection between WQ-beam and consol		
Welding:	$a_{con} \geq$ mm			
	$a_{e.pla} \geq$ mm $F_{max} =$ kN			

4.1.3 Shear force in the consol

EN 1993-1-8 : 2005, p. 44:

The design resistance of a fillet weld may be assumed to be adequate if, at every point along its length, the resultant of all the forces per unit length transmitted by the weld

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

The design resistance per unit length $F_{w,Rd}$ should be determined from:

$$F_{w,Rd} = f_{vw,d} \cdot a$$

The design value of shear force

$$F_{w,Ed} = \quad \text{kN}$$

The design resistance for shear force

$$F_{w,Rd} = \quad \text{kN}$$

Utilizaition: %

4.1.4 Bending moment in the consol

EN 1993-1-1: 2005, p. 50:

The design value of the bending moment M_{Ed} at each cross-section should satisfy:

$$\frac{M_{Ed}}{M_{c,Rd}} \leq 1,0$$

The design value of the bending moment

$$M_{Ed} = F \cdot e = \quad \text{kNmm}$$

The design resistance for bending moment

$$M_{c,Rd} = W_{pl} \cdot f_y / \gamma_{M0} = \quad \text{kNmm}$$

Utilizaition: %

Company	Project number		Page
Aaro Kohonen FMC GROUP	01-0300-		5(6)
	Date	Factor	
	23.05.2012	LiSe	
Designer	Contents		
Lidia Sergeeva	Connection between WQ-beam and consol		
Welding: $a_{con} \geq$ *mm*; $a_{e.pla} \geq$ *mm* $F_{max} =$ *kN*			

4.2 End plate calculations

4.2.1 Welding calculation for the end plate

F= kN

Srtess in weld

$\tau = F / (h_{bep} \cdot b_{bep}) =$ N/mm²

Welding for plate

$$a \geq \frac{t_{con}}{2 \cdot f_{vw,d}} \cdot \sqrt{3\tau_{xy}^2} \geq$$ mm

The final welding for end plate mm

4.2.2 Stress in the end plate

$$\sigma_{eq} = \frac{t}{2 \cdot a} \cdot \sqrt{2\sigma_x^2 + 3\tau_{xy}^2}$$

$$\sigma_{eq} \leq f_{vwd}$$

≤ 262

Utilizaition: %

4.2.3 Shear force in the end plate

EN 1993-1-8 : 2005, p. 44:

The design resistance of a fillet weld may be assumed to be adequate if, at every point along its length, the resultant of all the forces per unit length transmitted by the weld

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

The design resistance per unit length $F_{w,Rd}$ should be determined from:

$$F_{w,Rd} = f_{vw,d} \cdot a$$

The design value of shear force

$F_{w,Ed} =$ kN

The design resistance for shear force

$F_{w,Rd} =$ kN

Utilizaition: %

Company	Project number	Page
Aaro Kohonen FMC GROUP	01-0300-	6(6)
	Date / Factor: 23.05.2012 / LiSe	
Designer	**Contents**	
Lidia Sergeeva *Welding:* $a_{con} \geq$ ___ mm $a_{e.pla} \geq$ ___ mm $F_{max} =$ ___ kN	Connection between WQ-beam and consol	

4.3 Plate in the consol

4.3.1 Shear force in the plate

EN 1993-1-1: 2005,p. 50:

The design value of the shear force V_{Ed} at each cross section should satisfy:

$$\frac{V_{Ed}}{V_{pl,Rd}} \leq 1{,}0$$

The design value of the shear force:

$$V_{Ed} = DL + LL \cdot \psi_2 = \quad \text{kN}$$

The design plastic shear resistance

$$V_{Rd} = A \cdot \frac{f_y / \sqrt{3}}{\gamma_{M0}} = \quad \text{kN}$$

Utilizaition: %

4.3.2 Bending moment in the plate

EN 1993-1-1: 2005, p. 50:

The design value of the bending moment M_{Ed} at each cross-section should satisfy:

$$\frac{M_{Ed}}{M_{c,Rd}} \leq 1{,}0$$

The design value of the bending moment

$$M_{Ed} = F_{pla} \cdot (e + t_{col}) = \quad \text{kNmm}$$

The design resistance for bending

$$M_{c,Rd} = W_{pl} \cdot f_y / \gamma_{M0} = \quad \text{kNmm}$$

Utilizaition: %

4.3.3 Stress in the plate:

$$\sigma = \frac{M_{Ed}}{W_{pla}} \leq f_y \quad \text{N/mm}^2$$

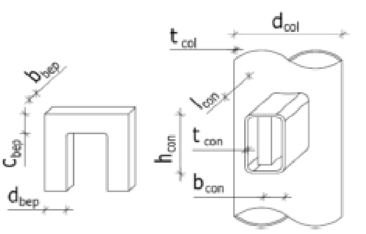
Utilizaition: %

$$W_{pla} = b_{pla} \cdot h_{pla}^2 / 6 = \quad \text{mm}^3$$

APPENDIX 2
Calculation sheet

Company		Project number		Page
Aaro Kohonen FMC GROUP		01-0300-		1(1)
		Date	Factor	
		23.05.2012	LiSe	
Designer		Contents		
Lidia Sergeeva		Connection between WQ-beam and consol		
Welding:	$a_{con} \geq$ _ mm			
	$a_{e.pla} \geq$ _ mm F_{max} = _ kN			

Column	Column diameter	d_{col}=		mm
	Column thickness	t_{col}=		mm
Consol	Consol length	l_{con}=		mm
	Consol high	h_{con}=		mm
	Consol breadth	b_{con}=		mm
	Consol thickness	t_{con}=		mm
Plate	Plate length	l_{pla}=		mm
	Plate high	h_{pla}=		mm
	Plate breadth	b_{pla}=		mm
End plate	End plate breadth	b_{bep}=		mm
	Height of the console on top	c_{bep}=		mm
	End plate high	h_{bep}=		mm
	Width of the console side	d_{bep}=		mm
	Yield strength	f_y=	355	N/mm²
	Ultimate tensile strength	f_u=	510	N/mm²
		$f_{vw,d}$=	262	N/mm²



S355

1. Capacity of the consol

1.1. Stress in the consol		σ=	N/mm²
Utilizaition:	%		
1. 2. Shear force in the consol		$F_{w,Rd}$=	kN
Utilizaition:	%		
1. 3. Bending moment in the consol		$M_{c,Rd}$=	kNmm
Utilizaition:	%		

2. Capacity of the end plate

2.1. Stress in the end plate		σ=	N/mm²
Utilizaition:	%		
2. 2. Shear force in the end plate		$F_{w,Rd}$=	kN
Utilizaition:	%		

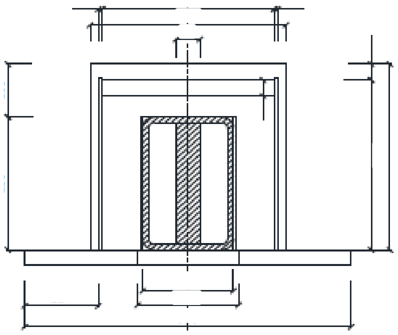
3. Capacity of the plate inside the consol

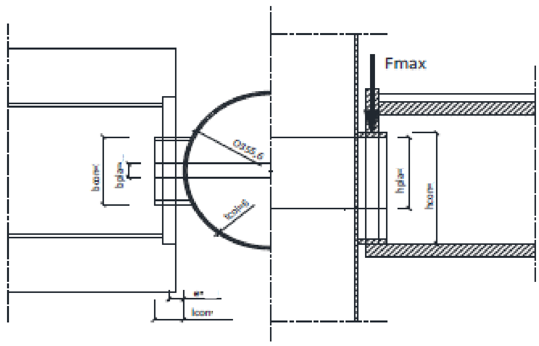
3. 1. Shear force in the plate		$V_{pl,Rd}$=	kN
Utilizaition:	%		
3. 2. Bending moment in the plate		$M_{c,Rd}$=	kNmm
Utilizaition:	%		
3. 3. Stress in the plate		σ=	N/mm²
Utilizaition:	%		

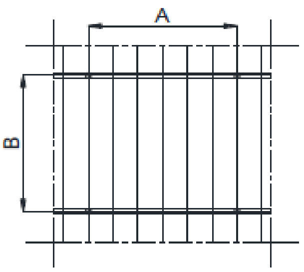
Aaro Kohonen FMC GROUP	Document name STANDARD	Project number 01-0300-	
Subject Column/WQ-beam	Date 21.05.2012	Prepared/Appr. by	Page 1(1)

Title

WQ-BEAM/COLUMN CONNECTION /WQ- 000-0-00 x 000-00 x 000 /







Materials:		Dimentions:
Hollow section S355J2H:		
Consol		RHS 000 x 000 x 00
Plates S355J2+N:		
Plate		00 x 000 x 00
End Plate		000 x 000 x 00

Welding:

Class C SFS-EN ISO 5817

Dead load:

Surface of cast	h= mm	x2	kN/m^2
Hollow cor slab weight of sealed	h= mm		kN/m^2
Walls			kN/m^2
Suspension			kN/m^2
Floor covering			kN/m^2
		ΣDL=	kN/m^2

Live loads:

For office/house building	LL=	3,0	kN/m^2	ΣDL+LL=	kN/m^2
For commercial premises	LL=	5,0	kN/m^2	ΣDL+LL=	kN/m^2

Maximum normal forces for Office/house building:

(AxB)/2 [mxm]			
F_{max} [kN/m]			
a_{cons} [mm]			
$a_{e.pl}$ [mm]			

Maximum normal forces for Commercial premises:

(AxB)/2 [mxm]			
F_{max} [kN/m]			
a_{cons} [mm]			
$a_{e.pl}$ [mm]			