VSB - Technical University of Ostrava

Faculty of Civil Engineering

Department of Structures

STATIC DESIGN OF COMPOSITE STEEL AND CONCRETE BUILDING STRUCTURE

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Degree in: Universidad de Oviedo

Ostrava 2017

VŠB - Technical University of Ostrava
Faculty of Civil Engineering,

Bachelor Thesis Assignment

Student: **Alejandro VALLE RIERA**

Study Program: B3607 Civil Engineering

Study Branch: Erasmus+ Exchange Study Program

Title: **Static design of composite steel and concrete building structure.**
Statické posouzení spřažené ocelobetonové konstrukce patrové budovy.

The thesis language: English

Table of Content:

1. Typical arrangement steel and composite steel and concrete building structures – basic type of structures, static behavior, principles of design, problematic details and its solution.
2. Actions on the chosen building structure, combination of loads, calculation of internal forces – 2D and/or 3D numerical model.
3. Static design of main parts of the structure, economical comparison of possible solutions (weight, difficulty, stiffness, etc.).
4. Detailed design of chosen (difficult, special) parts of the building structure, design of all important details and connections.
5. Drawing documentation – plan, main sections, details, manufacture drawing of selected part of the structure.

References:

[1] EN 1990: Basis of structural design.

[2] EN 1991: Actions on structures. (all needed parts)

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

[4] Vičan, J., Odrobiňák, J.: Steel structures, Žilina 2008, ISBN: 978-80-554-0053-2

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[7] Journals: Structural Engineering, Stahlbau, etc.

[8] Internet

Extent and terms of a thesis are specified in directions for its elaboration that are opened to the public on the web sites of the faculty.

Date of issue: 20. 02. 2017

Date of submission: 31. 05. 2017

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ANNOTATION OF THE THESIS

The work that has been carried out to realize this thesis is based, mainly, in the development and calculation of the main structural of an office building.

The first part of the project documents the basis of the composite structures and the possible types of joints in the structure.

In the second part the calculations are developed to concretely design the ideal dimensions of the main elements of the building. This includes main beams, cross beams, columns and stiffening system mainly.

In addition, a plan has been made with the shape of the building and some important details.

To carry out this work, basic knowledge about structures and construction has been applied and has been carried out in accordance with European regulations, Eurocode.

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1. COMPOSITE STRUCTURES

1.1. OBJECTIVE

The most important and most frequently encountered combination of construction materials is that of steel and concrete, with applications in multi-storey commercial buildings and factories, as well as in bridges. These materials can be used in mixed structural systems, for example concrete cores encircled by steel tubes, as well as in composite structures where members consisting of steel and concrete act together compositely.

These essentially different materials are completely compatible and complementary to each other; they have almost the same thermal expansion; they have an ideal combination of strengths with the concrete efficient in compression and the steel in tension; concrete also gives corrosion protection and thermal insulation to the steel at elevated temperatures and additionally can restrain slender steel sections from local or lateral-torsional buckling.

The composite sections using steel encased with concrete are economic, cost and time effective solution in major civil structures such as bridges and high rise buildings. [1]

The composite structures marked the initial phase between (1850–1900). This was followed by the constitution phase (1900–1925) with its constructional separation of the elements of the cross-section. During the establishment phase (1925–1950) it was gradually realized that the elements of the cross-section had to be connected structurally, initially as positional restraint, later as mechanical shear connector. The quantified connection of the elements of the cross-section through standardized testing and the formation of theories in the classical phase (1950–1975) enabled the realization of multiple forms of steel-concrete composite construction for industrial buildings and bridges. [2]

In due consideration of the above fact, this project has been envisaged which consists of analysis and design of a high-rise building using Steel-Concrete composites.

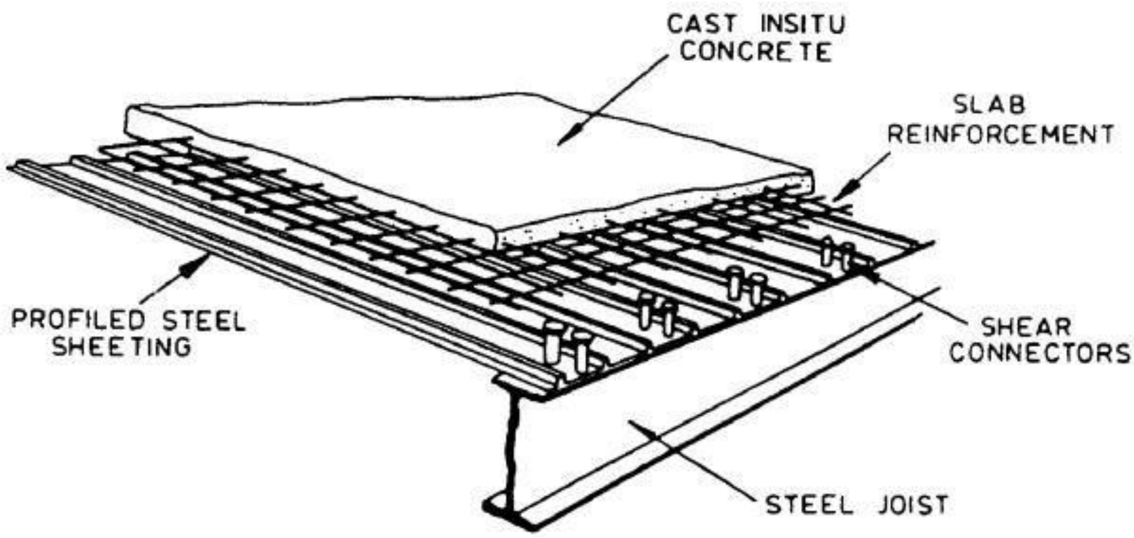


Fig. 1.1. Basic scheme of the appearance and parts of a composite structure.

1.2. DEFINITION

What is meant by a composite element is one that consists of a rolled or a built-up structural steel encased by reinforced concrete or structurally connected to a reinforced concrete slab. Composite members are constructed such that the structural steel shape and the concrete act together to resist axial compression and / or bending.

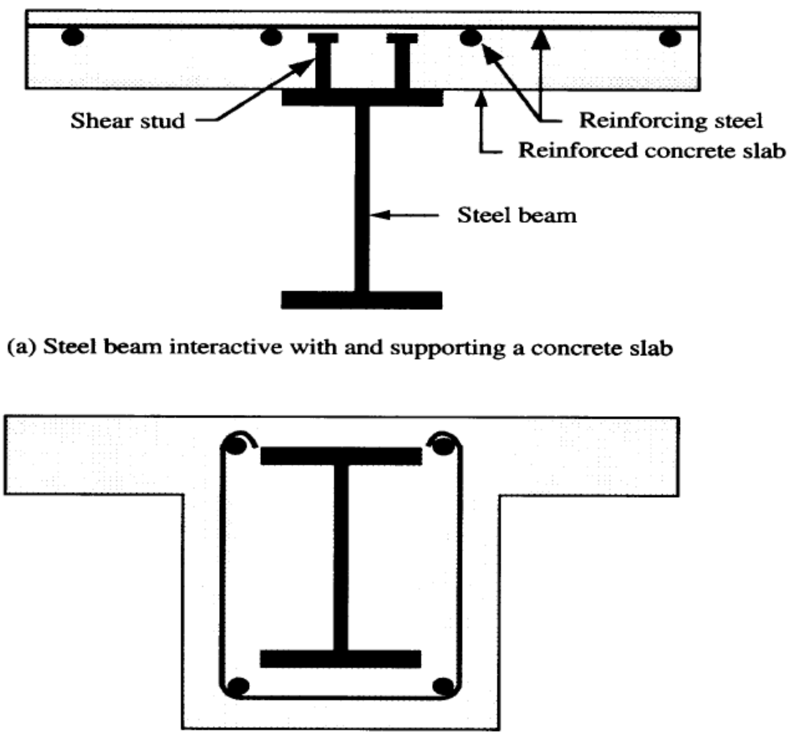


Fig. 1.2. Appearance of a typical composite beam.

In such a composite member, the comparatively high strength of the concrete in compression complements the high strength of the steel in tension. The fact that each material is used to the fullest advantage makes composite Steel-Concrete construction very efficient and economical. However, the real attraction of such construction is based on having an efficient connection of the steel to the concrete, and it is this connection that allows a transfer of forces and gives composite members their unique behaviour.

1.3. ELEMENTS OF COMPOSITE STRUCTURE

PROFILED DECK:

Composite floors using profiled sheet decking have become very popular for high-rise buildings. Composite deck slabs are generally competitive where the concrete floor should be completed quickly and where medium level of fire protection to steel work is sufficient.

COMPOSITE BEAMS:

Slab and beam type constructions are commonly used in buildings and bridges.

Composite beams, subjected mainly to bending, consist of steel section acting compositely with flange of reinforced concrete. To act together, mechanical shear connectors are provided to transmit the horizontal shear between the steel beam and the concrete slab, ignoring the effect of any bond between the two materials.

This behaves like a T-beam with the slab or part of it acting as a flange in compression. Further, bond between the shear connector and slab is assumed to be perfect, i.e., no slippage between the top flange of the steel beam and slab is permitted.

These also resist uplift force acting at the steel concrete interface.

For determining section properties, it is convenient to transform the concrete slab into an equivalent steel section by dividing concrete area by modular ratio. The rest of the analysis is carried out as if the section were made of a homogeneous material.

Advantages of Construction:

• The most effective utilization of steel and concrete is achieved.

• Keeping the span and loading unaltered, a more economical steel section (in terms of depth and weight) is adequate in composite construction compared with conventional non-composite construction.

• As the depth of beam reduces, the construction depth reduces, resulting in enhanced headroom.

• Because of its larger stiffness, composite beams have less deflection than steel beams.

• Composite construction is amenable to "fast-tract" construction because of using rolled steel and pre-fabricated components, rather than cast-in-situ concrete.

• Encased steel beam sections have improved fire resistance and corrosion.

Disadvantages:

• Additional costs for shear connectors and their installation. For lightly loaded short beams, this extra cost may exceed the cost-reduction on all accounts.

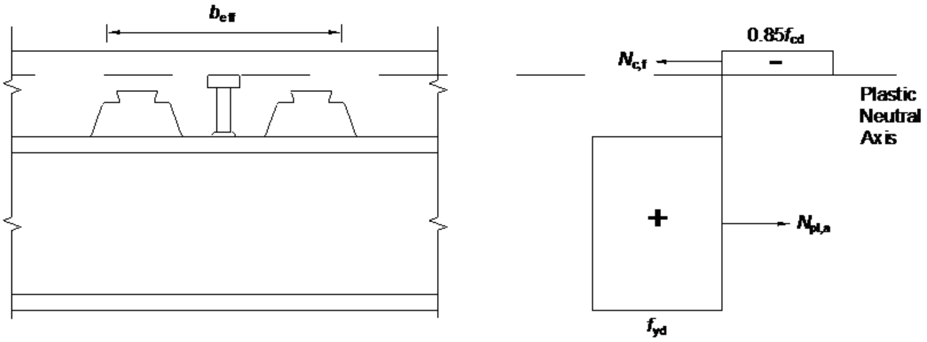


Fig. 1.3. Plastic stress distribution in a composite beam

COMPOSITE COLUMS:

A steel-concrete composite column is a compression member, comprising either a concrete encased hot-rolled steel section or a concrete filled hollow section of hot-rolled steel.

It is generally used as a load-bearing member in a composite framed structure.

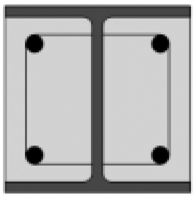
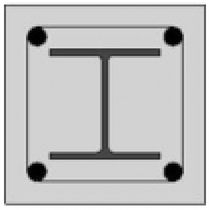
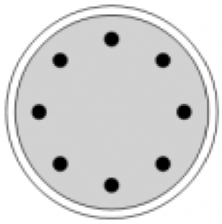
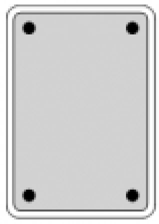
Partially encased HE section	Fully encased HE section	Concrete-filled CHS	Concrete-filled SHS
			

Fig. 1.4. A few examples of composite columns.

Advantages:

• Increased strength for a given cross sectional dimensions.

• Increased stiffness, leading to reduced slenderness and increased buckling resistance.

• Good fire resistance

• Corrosion protection in encased columns.

• Significant economic advantages over either structural steel or R.C.C. alternatives.

• Identical cross sections with different load and moment resistances can be produced by varying steel thickness, the concrete strength or reinforcement. This allows the outer dimensions of a column to be held constant over several floors in a building, thus simplifying the construction and architectural detailing.

• Erection of high rise building in an extremely efficient manner.

• Formwork is not required for concrete filled tubular sections.

COMPOSITE SLABS:

Composite slabs comprise reinforced concrete cast on top of profiled steel decking, which acts as formwork during construction and external reinforcement at the final stage. The decking may be either re-entrant or trapezoidal, as shown below.

Additional reinforcing bars may be placed in the decking troughs, particularly for deep decking. They are sometimes required in shallow decking when heavy loads are combined with high periods of fire resistance. [3]

SHEAR CONNECTORS:

Shear connections are essential for steel concrete construction as they integrate the compression capacity of supported concrete slab with supporting steel beams / girders to improve the load carrying capacity as well as overall rigidity.

Shear connectors are generally classified as rigid or flexible.

RIGID TYPE:

These connectors as the name implies, are designed to be bent proof with little inherent power of deformation. These types of shear connectors could be of various shapes, but the most common types are short length of bars, angles or tees welded on to the steel girder.

FLEXIBLE TYPE:

Flexible type connectors such as studs, channels welded to the structural beams derive their resistance essentially through the bending of the connectors and normally failure occurs when the yield stress in the connector is exceeded resulting in slip between the structural beam and the concrete slab.

There are some examples of connectors:

Fig. 1.5. Headed studs.

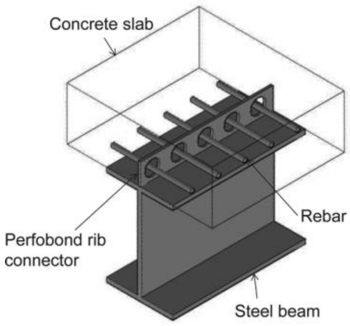


Fig. 1.6. Perfobond ribs.

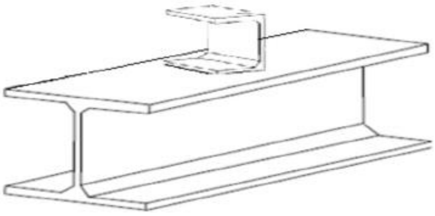


Fig. 1.7. Channel connector.

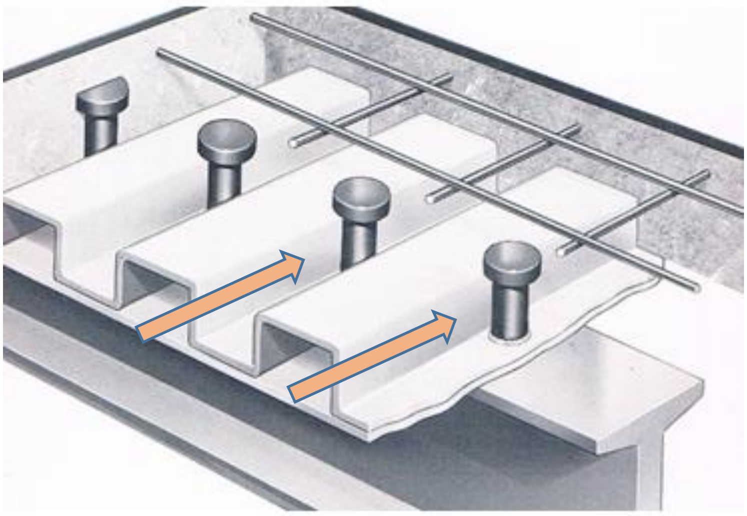


Fig. 1.8. Position of the connectors within the structure.

1.4. ADVANTAGES OF STEEL-CONCRETE COMPOSITE CONSTRUCTION

1. Faster construction for maximum utilization of rolled and/or fabricated components (structural steel members) and hence quick return of the invested capital.

2. Advantages based on life-cycle-cost analysis instead of initial cost only.

3. Quality assurance of the steel material along with availability of proper paint system suiting to different corrosive environment.

4. Ability to cover large column free area in buildings and longer span for bridges/flyovers.

This leads to more usable space.

5. Reinforced cement concrete (RCC) slab is in compression and steel joist is in tension.

Hence, most effective utilization of the materials can be achieved.

6. Better seismic resistance i.e. best suited to resist repeated earthquake loadings, which require a high amount of ductility and hysteretic energy of the material/structural frame.

7. Composite sections have higher stiffness than the corresponding steel sections (in a steel structure) and thus bending stress as well as deflection are lesser.

8. Keeping span and loading unaltered, a lower structural steel section (having lesser depth and weight) can be provided in composite construction, compared to the section required for non-composite construction.

9. Reduced beam depth reduces the story height and consequently the cost of cladding in a building and lowers the cost of embankment in a flyover (due to lower height of embankment).

10. Reduced depth allows provision of lower cost for fire proofing of beam's exposed faces.

11. Cost of formwork is lower compared to RCC construction.

12. Cost of handling and transportation is minimized for using major part of the structure fabricated in the workshop.

13. Easy structural repair, modification and maintenance.

14. Structural steel component has considerable scrap value at the end of useful life.

15. Reductions in overall weight of structure and thereby reduction in foundation costs.

16. More use of a material i.e. steel, which is durable, fully recyclable on replacement and environment friendly.

1.5. COMPARISON COMPOSITE AND R.C.C STRUCTURE

After evaluating the advantages and disadvantages of both composite structures and R.C.C., and considering various experiments with real structures, we conclude that it is preferable to use structures composed of the reasons explained below. [4] [5] [6]

STIFFNESS: Transverse and longitudinal storey stiffness for composite structure is large as compared to RCC structure.

BASE SHEAR: Base shear due to earthquake load, for composite building lower than R.C.C.

LATERAL FORCES: It is clear that the lateral forces acting on a RCC structure are much more than steel and composite structure, hence composite structure is less susceptible against seismic forces action on structure.

STOREY DRIFT: The result shows that the inter storey drift for composite structure is comparatively less than R.C.C. structure in both transverse and longitudinal direction.

DISPLACEMENT: It is observed that composite structure has less displacement compared to R.C.C.

MODAL FRECUENCY: The increased stiffness of composite structure results in increased frequency and reduction in time period than RCC and steel structure.

IN COLUMNS:

- Axial force in composite columns is reduced than RCC columns.
- Shear force in composite column is reduced in transverse and longitudinal directions respectively.
- The twisting moments are found to be negligible and for composite structure these are reduced in transverse and longitudinal directions respectively as compared to RCC structure.
- The bending moment in composite columns is reduced in transverse direction and in longitudinal direction as compared to RCC columns.

WEIGHT: Weight of various types of structures is very important to know because it will affect the cost of foundation as well as the cost of ground improvement.

Weight results from various research papers can be summarized as below.

- Weight of the composite structure is quite low as compared to RCC structure, which helps in reducing foundation cost.
- Dead load of composite is less than RCC and more than steel.

COST: Cost is a major aspect of comparison of steel, RCC and composite buildings. Because costly structures are generally neglected in construction if another cheaper option is available in front of it.

• For multi-storey buildings:

- Cost of composite beams is less than RCC beams because composite beams do not require any formwork.
- As axial forces and reactions are less in composite columns as compared to RCC columns, so cost of composite columns is less.

It concludes that composite buildings are more economical than RCC in this case.

• For low rise buildings:

- Cost of composite buildings is more than RCC and less than steel structures.

CONCLUSIONS:

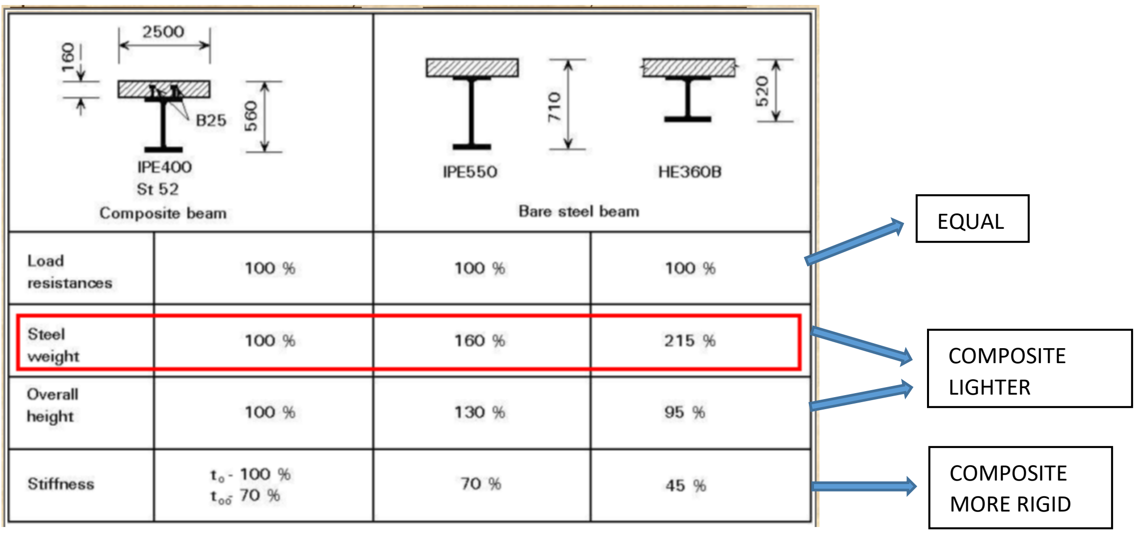
• Overall response of composite structure is better than RCC structure i.e. composite structure produces less displacement and resists more structural forces.

• Composite structures are best solution for high rise buildings and they are resulted in speedy construction.

• Steel option is better than RCC but the composite option for high rise building is best.

• Steel has excellent resistance to tensile loading but prone to buckling and concrete gives more resistance to compressive force. Steel can be used to induce ductility and concrete can be used for corrosion and fire protection.

• Composite structures are resulted into lighter construction than traditional concrete construction as well as speedy construction. So, completion period of composite building is less than RCC building.



	IPE400 St 52 Composite beam	IPE550 Bare steel beam	HE360B Bare steel beam
Load resistances	100 %	100 %	100 %
Steel weight	100 %	160 %	215 %
Overall height	100 %	130 %	95 %
Stiffness	t_0 - 100 % t_∞ 70 %	70 %	45 %

Fig. 1.9. Comparison of a Composite beam with other two that are not (IPE and HEB).

1.6. DESIGN OF A COMPOSITE STRUCTURE

A composite structure or part of it, is considered, unfit for use when it exceeds the limit state, beyond which it infringes one of the criteria governing its performance or use.

The limit states can be classified into the following categories:

• ***Ultimate Limite State***, which corresponds to the maximum load carrying capacity.

• ***Serviceability Limit State***, which are related to the criteria governing normal use and durability.

Ultimate Limit State to be considered in buildings and structures made of steel-concrete composite construction are:

• Collapse due to flexural failure of one or more critical sections.

• Collapse due to horizontal shear failure at the interface between the steel beam and the concrete slab.

• Collapse due to vertical separation of the concrete slab from the steel beam.

The serviceability limit states to be considered are as follows:

• Limit state of deflection.

• Limit state of stresses in concrete and steel.

Design for the limit state of collapse in flexure is based on the following assumptions:

• Plane sections normal to the axis remain plane-after bending.

• The maximum bending strain in concrete at the outermost compression fiber is taken as 0,0035.

• For characteristic compressive strength of concrete *fck*, maximum permissible bending compression in the structure is assumed to be 0.67 *fck*. With a value of 1.5 for the partial safety factor for the strength of concrete material, maximum design stress is 0.446 *fck*.

• The tensile strength of concrete is ignored.

• The stress-strain curve for the steel section is assumed to be bilinear and partial safety factor of the material is 1.15.

2. CONNECTIONS

According to the design of the building and the construction of it, it is necessary to study how the elements will be arranged.

Specifically, in this section will choose which types of joints to be used, both between beams, as between a beam and a column.

To do this, the efforts to which they will be subjected and the advantages and disadvantages of each one of the unions are analysed, to choose one that efficiently provides a secure connection.

It will be used in the building ***simple connections***, which allow the beam end to rotate without a significant restraint. These connections transfer shear out of the beam. [7]

Simple connections are nominally pinned connections that are assumed to transmit end shear only and to have negligible resistance to rotation. Therefore, they do not transfer significant moments at the ultimate limit state.

This definition underlies the design of multi-storey braced designed as 'simple construction', in which the beams are designed as simply-supported and the columns are designed for axial load and the small moments induced by the end reactions from the beams. Stability is provided to the frame by bracing or by the concrete core.

There are two principle ways of simple connection, these being:

• Flexible end-plates.

• Fin plates.

Commonly simple connections include:

• Beam-to-beam and beam-to-column connections using:

- Partial depth end plates
- Full depth end plates
- Fin plates

• Column splices (bolted cover plates or end plates).

• Column bases.

• Bracing connections (Gusset plates).

As for the considerations of the joints, nominally pinned joints should be able to transmit the internal forces, without developing significant moments which might adversely affect the members or the whole structure and be capable to accept the resulting rotations under the design loads.

In addition, the joint must provide the directional restraint to members which has been assumed in the member design and have sufficient robustness to satisfy the structural integrity requirements (tying resistance).

2.1. CONNECTION TYPES

The selection of beam end connections can often be quite involved.

Selection of beams and connections is generally the responsibility of the steelwork contractor who will choose the connection type to suit the fabrication workload, economy and temporary stability during erection.

The relative merits of the three connection types (partial depth end plates, full depth end plates and fin plates) are summarised in the table below.

	Partial depth end plate	Full depth end plate	Fin plate
Design			
Shear resistance - % of beam resistance	Up to 75%	100%	Up to 50% *
Tying resistance	Fair	Good	Good
Special considerations			
Skewed Joints	Fair	Fair	Good
Beams eccentric to columns	Fair	Fair	Good
Connection to column webs	Good	Good	Fair **
Fabrication and treatment			
Fabrication	Good	Good	Good ***
Surface treatment	Good	Good	Good
Erection			
Ease of erection	Fair ****	Fair ****	Good
Site adjustment	Fair	Fair	Fair
Temporary stability	Fair	Good	Fair

Table 2.1. Comparison of the properties of partial & full depth end plate & fin plate connections.
** Up to 75% with two vertical lines of bolts*
*** To facilitate erection, flange stripping may be required. Stiffening may be required for long fin plates*
**** Stiffening may be required for long fin plates*
***** Care needed for two-sided connections*

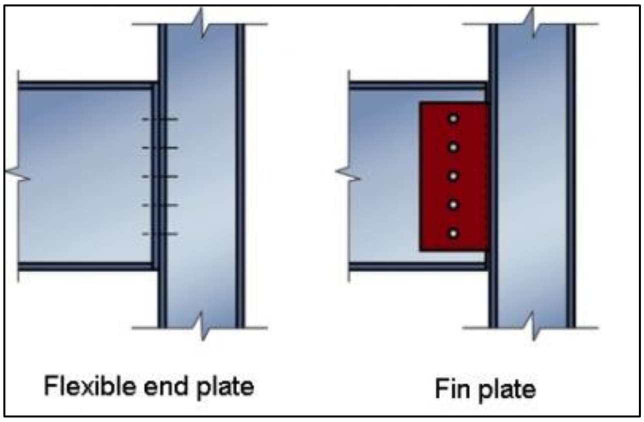


Fig. 2.1. Flexible end plate & fin plate connection.

2.2. BEAM-TO-BEAM AND BEAM-TO-COLUMN CONNECTIONS

The end plate beam to beam connection is similar to the beam to column end plate connection. however, because the top flanges of the beam support floors or roofs structures directly, the top flange of the end of the incoming beam should be notched.

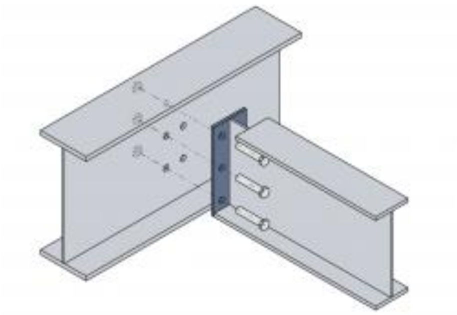


Fig. 2.2. Connection with the top flange of the incoming beam notched.

An alternative detail is to provide a projecting welded bracket or plate on the supporting beam. Adjustment is similar to the beam to column detail.

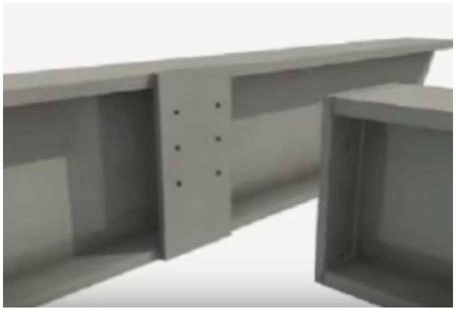


Fig. 2.3. Connection with a welded bracket or plate on the supporting beam.

End plate beam to column connection are common for moment transfer joins.

An end plate is welded to the end of the beam and is bolted trough the flange of the column.

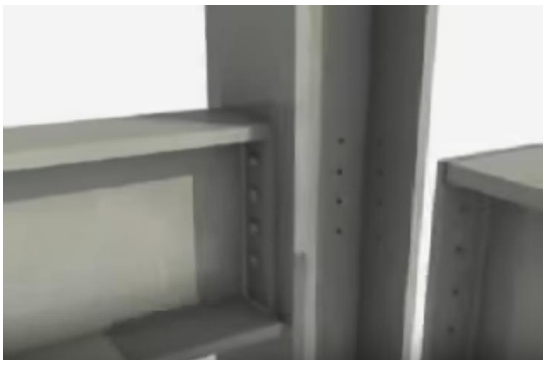


Fig. 2.4. Beam to column connection with the flange and web of the column.

2.3. FLEXIBLE END PLATES CONNECTIONS

In these connections, the end plate, which may be partial depth or full depth, is welded to the supported beam in the workshop. The beam is then bolted to the supporting beam or column on site.

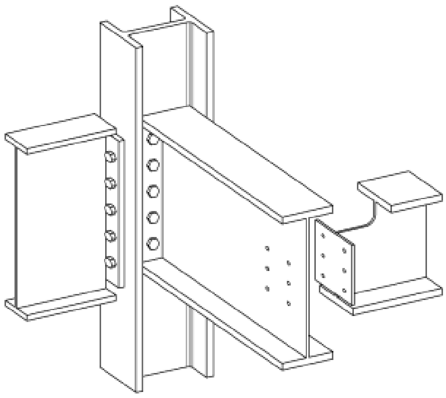


Fig. 2.5. Flexible end plates connections.

This type of connection is relatively inexpensive but has the disadvantage that there is little opportunity for site adjustment. Overall beam lengths need to be fabricated within tight limits, although packs can be used to compensate for fabrication tolerances and erection tolerances.

End plates are probably the most popular of the simple beam connections currently in use. They can be used with skewed beams and can tolerate moderate offsets in beam to column joints.

Flowdrill, Hollo-Bolts, Blind bolts or other special assemblies are used for connections to hollow section columns.

Standard flexible end plate details (full depth and partial depth end plates) are shown in the figure below.

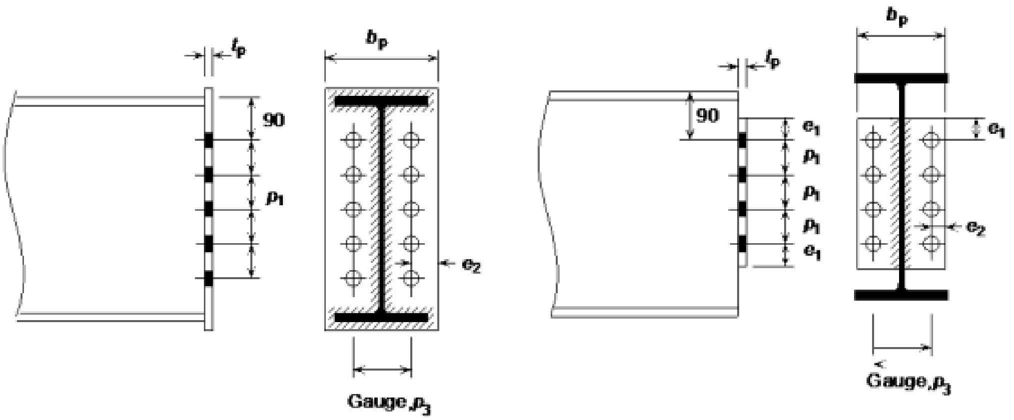


Fig. 2.6. Lateral and transverse view of a flexible end plate connection.

2.4. FIN PLATES CONNECTIONS

Fin plate connections are economical to fabricate and simple to erect. These connections are popular, as they can be the quickest connections to erect and overcome the problem of shared bolts in two-sided connections.

A fin plate connection consists of a length of plate welded in the workshop to the supporting member, to which the supported beam web is bolted on site, as shown in the figure below. There is a small clearance between the end of the supported beam and the supporting column.

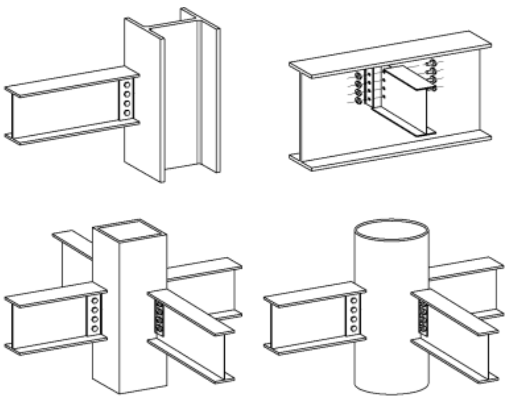


Fig. 2.7. Fin plates connections.

In the design of a fin plate connection it is important to identify the appropriate line of action for the shear. There are two possibilities: either the shear acts at the face of the column or it acts along the centre of the bolt group connecting the fin plate to the beam web.

For this reason, both critical sections should be checked for a minimum moment taken as the product of the vertical shear and the distance between the face of the column and the centre of the bolt group. Both critical sections are then checked for the resulting moment combined with the vertical shear.

Due to the uncertainty of the moment applied to the fin plate, the fin plate welds are sized to be full strength.

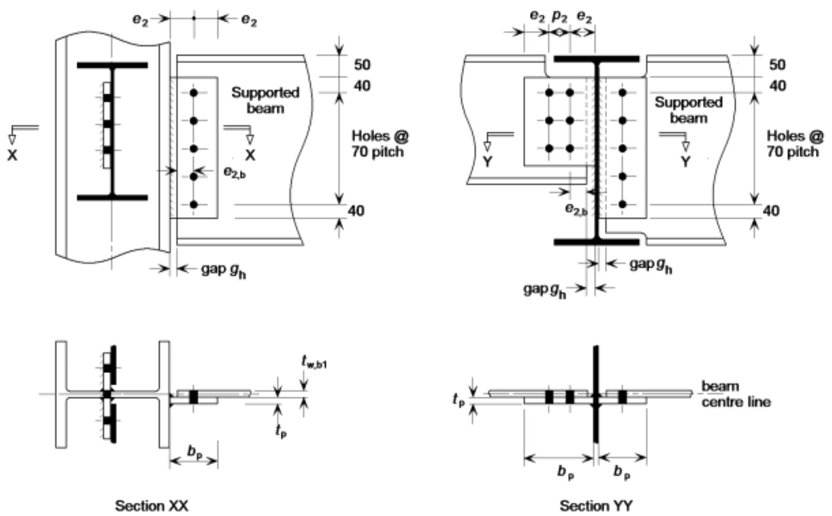


Fig. 2.8. Lateral and transverse view of a fin plate connection.

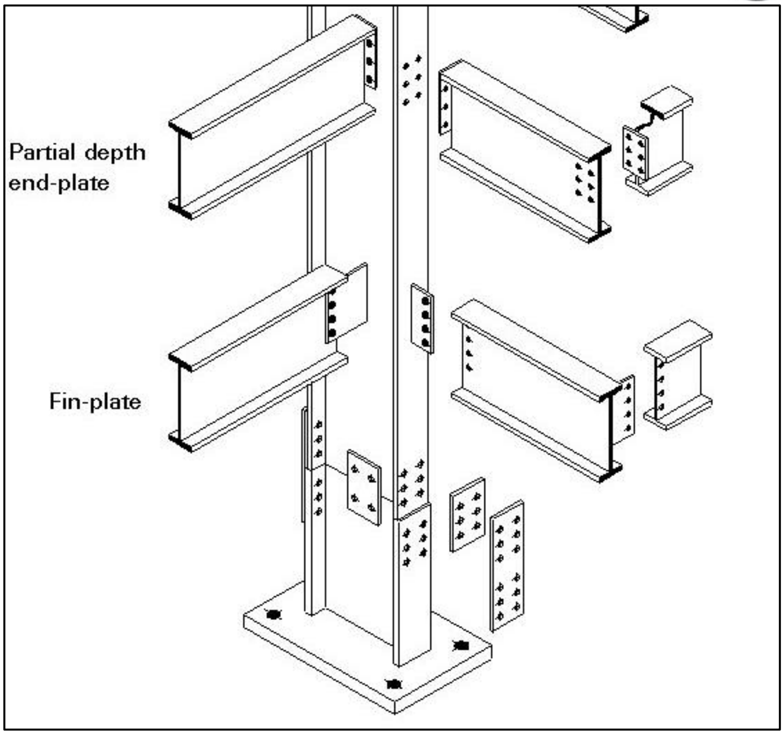


Fig. 2.9. Difference between partial depth end-plate connection and fin-plate connection.

3. MATERIALS TO BE USED IN THE PROJECT

3.1. STEEL

The common characteristics to all types of steel are:

• Modulus of elasticity (E) .... $210.000\ N/mm^2$

• Transverse Modulus of elasticity (G) .... $81.000\ N/mm^2$

• Coefficient of Poisson (υ) 0,3

• Coefficient of thermal expansion (α) .... $1{,}2 \cdot 10^{-5}\ {}^{\text{o}}C^{-1}$

• Density (ρ) .... $7850\ kg/m^3$

The design of structures is based basically on the following properties of steel: [8]

• The mainly reason is the elastic limit.

• Ductility, hardness and other properties that may vary per the application of the structure.

• The availability and the cost: per the plant that manufactures the steel that type of steel has.

• Weldability: The weldability decreases with the amount of carbon.

If the value of Carbon Equivalent (CEV)> 0.5%, the weldability of the material is low.

• Local conditions: exposure environments and standards.

Tabla 4.1 Características mecánicas mínimas de los aceros UNE EN 10025

DESIGNACIÓN	Espesor nominal t (mm) — Tensión de límite elástico f_y (N/mm²) $t \leq 16$	$16 < t \leq 40$	$40 < t \leq 63$	Tensión de rotura f_u (N/mm²) $3 \leq t \leq 100$	Temperatura del ensayo Charpy ºC
S235JR					20
S235J0	235	225	215	360	0
S235J2					-20
S275JR					20
S275J0	275	265	255	410	0
S275J2					-20
S355JR					20
S355J0					0
S355J2	355	345	335	470	-20
S355K2					-20 (1)
S450J0	450	430	410	550	0

Table 3.1. Mechanic characteristics of the different steel types.

3.2. CONCRETE

Concrete is known by its grade which is designated as M15, M20 etc.

Letter M refers to concrete mix.

Number denotes the specified compressive strength (fck) of 150mm cube at 28 days, expressed in N/mm^2.

Thus, concrete is known by its compressive strength.

M20 and M25 are the most common grades of concrete, and higher grades of concrete should be used for severe, very severe and extreme environments. [9]

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

Table 3.2. Properties for each type of concrete.

PROPERTIES: [10]

STRENGTH AND DURABILITY:

Characterized mainly by its strength. Gains strength over time and does not show weakness due to moisture, mold or pests.

Concrete structures can withstand natural disasters such as earthquakes and hurricanes.

VERSATILITY:

It is used in most of the structures we see day by day as for example, buildings, bridges, runways and even roads.

FIRE-RESISTANCE.

LOW MAINTENANCE:

By being inert, compact and non-porous, does not attract mould or lose its key properties over time.

AFFORDABILITY:

Compared to other comparable building materials e.g. steel, concrete is less costly to produce and remains extremely affordable.

THERMAL MASS:

Concrete walls and floors slow the passage of heat moving through, reducing temperature swings.

This reduces energy needs from heating or air-conditioning, offering year-round energy savings over the life-time of the building.

LOCALLY PRODUCED AND USED:

Concrete transportation is relatively expense. That is the reason why very little cement and concrete is traded and transported internationally.

This saves significantly on transport emissions of CO_2 that would otherwise occur.

ALBEDO EFFECT:

The high "albedo" (reflective qualities) of concrete used in pavements and building walls means more light is reflected and less heat is absorbed, resulting in cooler temperatures.

This reduces the "urban heat island" effect prevalent in cities today, and hence reduces energy use for e.g. air-conditioning.

ENERGY EFFICIENCY IN PRODUCTION:

Numerous studies have shown that typically more than 80% of a building's CO_2 emissions do not come from the production of the materials nor the actual construction process; but rather from the use phase, which are mainly from the combustion of fuels in heating systems and the generation of the electricity that the building consumes for air conditioning, lighting etc.

4. TYPE OF PROFILES TO BE USED IN THE STRUCTURES

4.1. BEAMS

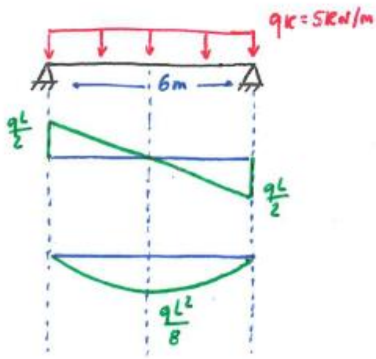
It is very common to see IPE profiles used in beams. In my project beams are going to be made with IPE profiles too.

Then, I am going to explain why it is better to use these profiles and not others with an example.

In fact, I will compare IPE profile with HEB profile.

EXAMPLE:

COMPARISON OF AN IPE PROFILE WITH A HEB IN A BEAM SUBJECTED TO AN UNIFORM LOAD



AS IT IS NOT KNOWN IF IS A VARIABLE OR A PERMANENT LOAD, IT WILL BE USED: $\gamma = 1.4$

$$V_{sd} = \frac{1,4 \cdot 5\,kN/m \cdot 6m}{2} = 21\,kN$$

$$M_{sd} = \frac{1,4 \cdot 5\,kN/m \cdot (6m)^2}{8} = 31,5\,kN\cdot m$$

IPE SUPOSITION IPE-180: $A_v = 1,04 \cdot h \cdot t_w = 1,04 \cdot 180 \cdot 5,3 = 992,16\,mm^2$

$$V_a = A_v \cdot \frac{f_y}{\gamma_a \cdot \sqrt{3}} = 992,16\,mm^2 \cdot 235\,N/mm^2 \cdot \frac{1}{1,0 \cdot \sqrt{3}} = 134,6\,kN \geq 2 \cdot V_{sd} = 42\,kN \checkmark$$

$$M_a = \frac{W_a \cdot f_y}{\gamma_{ma}} = \frac{166 \cdot 10^3\,mm^3 \cdot 235\,N/mm^2}{1,0} = 39,01\,kN\cdot m \geq M_{sd} = 31,5\,kN\cdot m \checkmark$$

$$\delta = \frac{1}{E_a \cdot I_a} \cdot \left(\frac{5}{384} \cdot q \cdot L^4\right) = \frac{1}{210 \cdot 10^3 \cdot 13,17 \cdot 10^3}\left(\frac{5}{384} \cdot 5 \cdot 6000^4\right) = 30,51\,mm \leq L/250 = 24\,mm \times$$

$\boxed{\text{IPE-200}}$: $A_v = 1,04 \cdot 200 \cdot 5,6 = 1164,8\,mm^2$

$V_a = 158,03\,kN \geq 2V_{sd} \checkmark$ $M_a = 51,94\,kN\cdot m \geq M_{sd} \checkmark$

$$\delta = \frac{1}{210 \cdot 10^3 \cdot 19,43 \cdot 10^3} \cdot \left(\frac{5}{384} \cdot 5 \cdot 6000^4\right) = 20,68\,mm \leq L/250 \checkmark$$

HEB

HEB-140 \ $A_v = 1{,}04 \cdot t_w \cdot h = 1{,}04 \cdot 140 \cdot 7 = 1019{,}2\ mm^2$

$$V_a = 1019{,}2\ mm^2 \cdot 235\ N/mm^2 \cdot \frac{1}{1{,}0 \cdot \sqrt{3}} = 138{,}28\ kN \geq 2 \cdot V_{sd}\ ✓$$

$$M_a = \frac{245{,}4 \cdot 10^3 \cdot 235}{1{,}0} = 57{,}67\ kN \cdot m \geq M_{sd}\ ✓$$

$$\delta = \frac{1}{210 \cdot 10^3 \cdot 15{,}09 \cdot 10^3} \cdot \left(\frac{5}{384} \cdot 5 \cdot 6000^4\right) = 26{,}63\ mm \leq L/250\ ✗$$

$\boxed{\text{HEB-160}}$ $V_a = 180{,}61\ kN$ ✓ $M_a = 83{,}19\ kN \cdot m$ ✓ $\delta = 16{,}12\ mm$ ✓

$$\text{IPE-200} \begin{Bmatrix} A = 2850\ mm^2 \\ \text{WEIGHT} = 22{,}4\ kg/m \end{Bmatrix} \Rightarrow \text{TOTAL WEIGHT: } 6m \cdot 22{,}4\ kg/m = \underline{\underline{134{,}4\ kg}}$$

$$\text{HEB-160} \begin{Bmatrix} A = 5430\ mm^2 \\ \text{WEIGHT} = 42{,}6\ kg/m \end{Bmatrix} \Rightarrow \text{TOTAL WEIGHT: } 6m \cdot 42{,}6\ kg/m = \underline{\underline{255{,}6\ kg}}$$

As a conclusion, it can be said that if we want to find an IPE and a HEB profile with the same capacity for bending, HEB will have a worse response in deflection, and HEB will be much heavier, what will be traduced in a much higher price.

PROFILE	CAPACITY FOR BENDING	STIFFNESS	WEIGHT (COST)
IPE-200	=	↑	↓↓
HEB-140	=	↓	↑↑

Table 4.1. Comparison between HEB & IPE profiles that have the same capacity or bending.

After this example, we can understand better why it is said that IPE profile is one of the best to be used in beams.

This profile combines a good resistance to bending and weight ratio which makes it one of the best in relation to behaviour-price.

4.2. COLUMNS

It is going to be explained in the same way as with the beams, the reason why HEB and not IPE profiles are usually used for the columns.

The columns are structural elements that work in compression. It is a known fact that, in compression, the main problem that appears is the buckling effect.

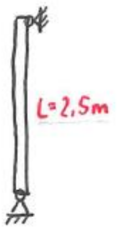
Buckling is a phenomenon of elastic instability that can occur in slender compressed elements, and is manifested by the appearance of important displacements transverse to the main direction of compression.

The maximum load will be realized assuming that the structure is not cross-braced, and therefore the weak plane check will be done, because it is the most restrictive value.

Another consideration that is taken is to suppose the bi-supported columns.

First, it is going to be compared, the resistance of an IPE and HEB profiles for different column´s length.

L=2,5m



BI-SUPPORTED COLUMN $\longrightarrow \beta = 1.0 \Longrightarrow l_{cr} = \beta \cdot l = 2,5\,m$ S-235 : $\lambda_1 = 93,9$

HEB-120 $\left\{ \begin{array}{l} I_z = 3,17 \cdot 10^6\,mm^4 \\ i_z = 30.6\,mm \end{array} \right\}$ IPE-240 $\left\{ \begin{array}{l} I_z = 2,84 \cdot 10^6\,mm^4 \\ i_z = 26,9\,mm \end{array} \right\}$

$\boxed{\text{HEB-120}}$ $\lambda = \frac{l_{cr}}{i_z} = \frac{2500\,mm}{30,6\,mm} = 81,7$

LAMINATED PROFILE $\longrightarrow h/b \leq 1,2 \longrightarrow t \leq 100\,mm \longrightarrow$ $\left. \begin{array}{l} \text{S-235} \\ \text{2-2 AXIS} \end{array} \right\}$ CURVE C $\Longrightarrow \boxed{\alpha = 0,49}$

HEB-120 $\longrightarrow$ CLASS 1 $\longrightarrow \beta_A = 1.0$ $\quad \bar{\lambda} = \left(\frac{\lambda}{\lambda_1}\right) \cdot \beta_A^{0,5} = \left(\frac{81,7}{93,9}\right) \cdot 1 = 0,87$

$$\phi = 0,5 \cdot \left[1 + \alpha \cdot (\bar{\lambda} - 0,2) + \bar{\lambda}^2\right] = 0,5 \cdot \left[1 + 0,49 \cdot (0,87 - 0,2) + 0,87^2\right] = 1,0426$$

$$\chi = \frac{1}{\phi + [\phi^2 - \bar{\lambda}^2]^{0,5}} = \frac{1}{1,0426 + [1,0426^2 - 0,87^2]^{0,5}} = 0,618$$

$$N_{b,Rd} = \chi \cdot \beta_A \cdot \frac{A \cdot f_y}{\gamma_{M1}} = 0,618 \cdot 1,0 \cdot \frac{3400\,mm^2 \cdot 235\,N/mm^2}{1,05} = 470,5\,KN$$

$\boxed{IPE\text{-}240}$ $\lambda = \frac{2500\,mm}{26{,}9\,mm} = 92{,}9$

LAMINATED PROFILE $\rightarrow h/b = 2 > 1{,}2 \rightarrow t \leq 40\,mm \rightarrow \left.\begin{matrix} S\text{-}235 \\ z\text{-}z \text{ AXIS} \end{matrix}\right\}$ CURVE B $\Rightarrow \boxed{\alpha = 0{,}34}$

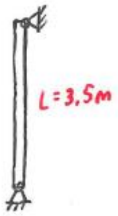
IPE-240 $\rightarrow$ CLASS 1 $\rightarrow \beta_A = 1{,}0$ $\quad \bar{\lambda} = \left(\frac{\lambda}{\lambda_1}\right) \cdot \beta_A^{0,5} = \left(\frac{92{,}9}{93{,}9}\right) \cdot 1 = 0{,}989$

$$\phi = 0{,}5 \cdot \left[1 + 0{,}34 \cdot (0{,}989 - 0{,}2) + 0{,}989^2\right] = 1{,}124$$

$$\chi = \frac{1}{1{,}124 + [1{,}124^2 - 0{,}989^2]^{0,5}} = 0{,}6035$$

$$N_{b,Rd} = 0{,}6035 \cdot 1{,}0 \cdot \frac{3910\,mm^2 \cdot 235\,N/mm^2}{1{,}05} = 528{,}1\,kN$$

L=3,5m



$\boxed{HEB\text{-}120}$

$l_{cr} = 1 \cdot 3{,}5\,m = 3{,}5\,m$

$\lambda = \frac{l_{cr}}{i_z} = \frac{3500\,mm}{30{,}6\,mm} = 114{,}38 \qquad \alpha = 0{,}49$

$\bar{\lambda} = \left(\frac{\lambda}{\lambda_1}\right) \cdot \beta_A^{0,5} = \left(\frac{114{,}38}{93{,}9}\right) \cdot 1 = 1{,}218$

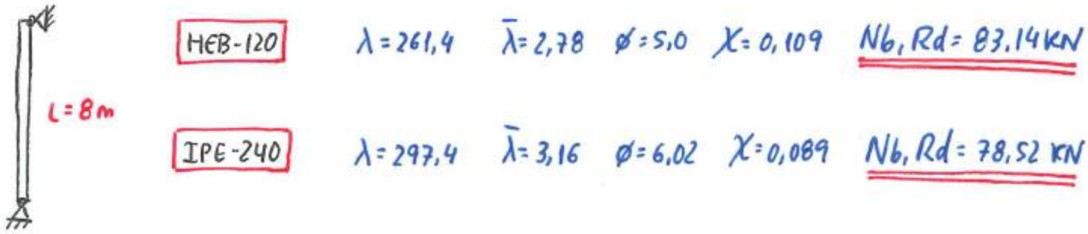
$$\left.\begin{matrix} \phi = 0{,}5 \cdot \left[1 + 0{,}49 \cdot (1{,}218 - 0{,}2) + 1{,}218^2\right] = 1{,}491 \\ \chi = \frac{1}{1{,}491 + [1{,}491^2 - 1{,}218^2]^{0,5}} = 0{,}425 \end{matrix}\right\} \quad N_{b,Rd} = 0{,}425 \cdot 1 \cdot \frac{3400 \cdot 225}{1{,}0} = 323{,}6\,kN$$

$\boxed{IPE\text{-}240}$ $\lambda = \frac{3500\,mm}{26{,}9\,mm} = 130{,}11 \qquad \alpha = 0{,}34$

$\bar{\lambda} = \left(\frac{130{,}11}{93{,}9}\right) \cdot 1 = 1{,}385$

$$\left.\begin{matrix} \phi = 0{,}5 \cdot \left[1 + 0{,}34 \cdot (1{,}385 - 0{,}2) + 1{,}385^2\right] = 1{,}66 \\ \chi = \frac{1}{1{,}66 + [1{,}66^2 - 1{,}385^2]^{0,5}} = 0{,}388 \end{matrix}\right\} \quad N_{b,Rd} = 0{,}388 \cdot 1{,}0 \cdot \frac{3910 \cdot 235}{1{,}05} = 339{,}7\,kN$$

L=8m



HEB-120: 26,7 kg/m

IPE-240: 30,7 kg/m

SIMILAR WEIGHT

• It can be seen how, for low length of 2.5 meters the IPE profile has greater resistance (about 60kN more).

However, as the buckling length increases, that difference is reduced (only 16kN for 3.5 meters) until for 8 meters ends up having better resistance HEB.

• HEB profiles have better buckling response in the strong and weak plane than the IPE profile. The latter does not have a good resistance to buckling in the weak plane.

• On the other hand, it is not convenient to choose a very large IPE profile since its dimensions would be excessively large and that is not profitable in the actual structure.

• To conclude, HEB profile cannot support much bigger loads.

For example, if it is necessary to resist 2500 kN, the maximum load that an HEB profile can resist without being class 4 is, for the example given before, 2150 kN.

So HEB profile is not a possible option when the column has to support large loads because the huge dimensions of the profile and because it will turn into class 4.

5. CALCULATION

Once it is defined the geometry and the design of the structure, as well as the type of steel and concrete to use, it is time to start with the basic calculations of the building.

First, it will be dimensioned the trapezoidal sheeting and from it, the cross beams and the main beams.

5.1. TRAPEZOIDAL SHEETING

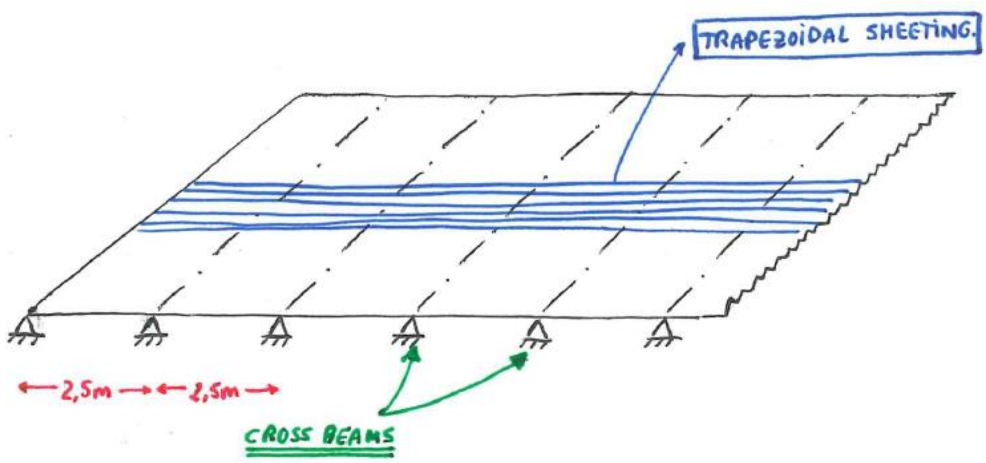


Fig. 5.1. Layout of the trapezoidal sheeting in front of the cross beams.

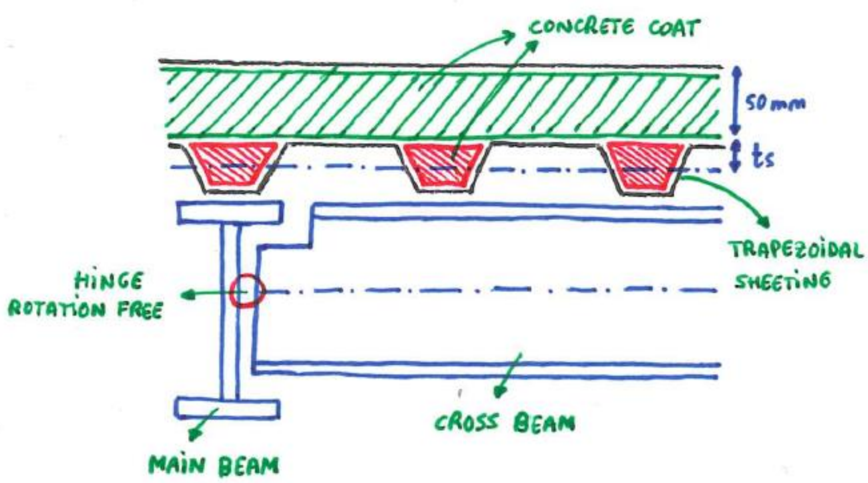


Fig. 5.2. scheme of arrangement between main & cross beam & trapezoidal sheet.

IT WILL BE USED A TRAPEZOIDAL SHEETING TYPE: $\boxed{\text{TR 50/250}}$

THEREFORE, IT IS NECESSARY TO CHOOSE THE NECESSARY THICKNESS OF THE SHEET TO RESIST THE LOADS THAT ACTUATE.

• CONCRETE LOAD:

- DENSITY OF WET CONCRETE: 2600 kg/m^3

IT IS NECESSARY TO CALCULATE THE CONCRETE THAT IS CONTAINED WITHIN THE HOLES OF THE TR.

THE CONCRETE LAYER OVER IT IS DESIGNED WITH A THICKNESS OF: 50 mm.

¿ts?

$$t_s = \frac{4 \cdot A}{1000\,mm} = \left\{ \begin{array}{c} \text{BASED ON THE} \\ \text{GEOMETRY} \\ \text{SHOWN ABOVE} \end{array} \right\} = \frac{4 \cdot 48{,}5\,mm \cdot (54+30{,}5)\,mm}{1000\,mm} = \underline{16{,}4\,mm}$$

$$A = (54 \cdot 48{,}5) + \left(\frac{(30{,}5 \cdot 48{,}5) \cdot 2}{2} \right) = (54+30{,}5) \cdot 48{,}5$$

IT IS NECESSARY TO COMMENT THAT THE NEEDED TRAPEZOIDAL SHEET IS CALCULATED BY METER WIDTH.

WEIGHT: $2600\,kg/m^3 \cdot 1\,m \cdot (50+16{,}4) \cdot 10^{-3}\,m = 172{,}64\,kg/m = \underline{1{,}73\,kN/m}$ (PERMANENT LOAD)

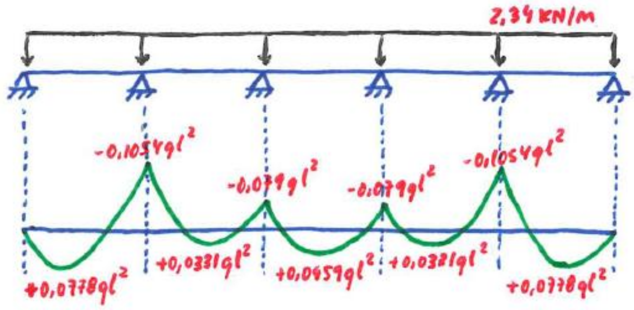
$q_k = 1{,}73\,kN/m \qquad \gamma_G = 1{,}35 \qquad \boxed{q = 1{,}35 \cdot (1{,}73\,kN/m) = 2{,}34\,kN/m}$

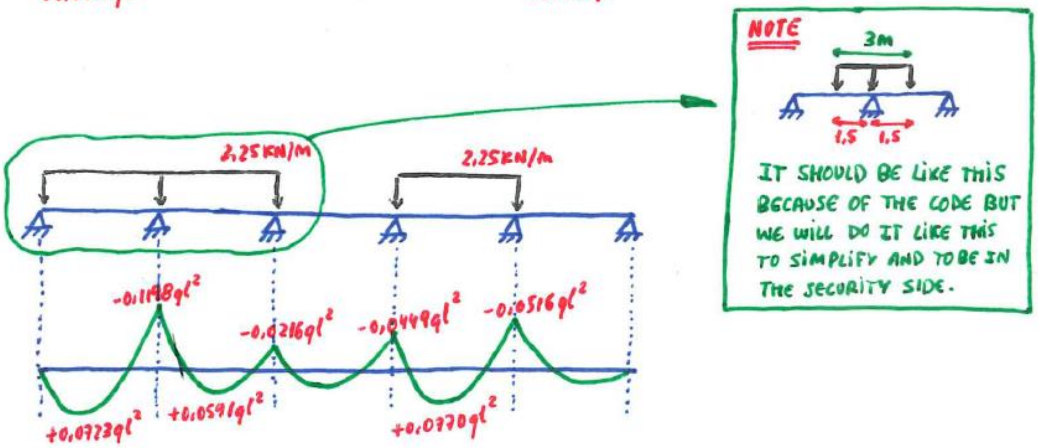
• FABRICATION LOAD: THE CODE SAYS THAT WE HAVE TO CONSIDER:

• $1{,}5\,kN/m^2$ IN 3x3 m AROUND THE UNFAVORABLE POINT.

• $0{,}75\,kN/m^2$ IN THE REMAINING AREA (IT WON'T BE CONSIDERED).

$q_k = 1{,}5\,kN/m^2 \cdot 1\,m = \underline{1{,}5\,kN/m}$ (VARIABLE LOAD). $\gamma_Q = 1{,}5$ $\boxed{q = 1{,}5 \cdot (1{,}5\,kN/m) = 2{,}25\,kN/m}$





NOW, WE SHOULD JOIN BOTH DIAGRAMS BY OVERLAPPING. BUT, AS ONLY THE MAXIMUM MOMENT IS REQUIRED, AND IN BOTH DIAGRAMS IT IS AT THE SAME POINT (IN THE FIRST SUPPORT), IT IS ENOUGH TO MAKE THE SUM OF THE TWO.

- $0,1054\, ql^2 = 0,1054 \cdot 2,34\ \text{KN/m} \cdot (2,5\text{m})^2 = 1,5415\ \text{KN}\cdot\text{m}$
- $0,1198\, ql^2 = 0,1198 \cdot 2,25\ \text{KN/m} \cdot (2,5\text{m})^2 = 1,6847\ \text{KN}\cdot\text{m}$

$$\boxed{3,23\ \text{KN}\cdot\text{m} = M_{sd}}$$

– ***POSITIVE POSITION*** (Filled with concrete narrow webs) – TR 50/250-1mm

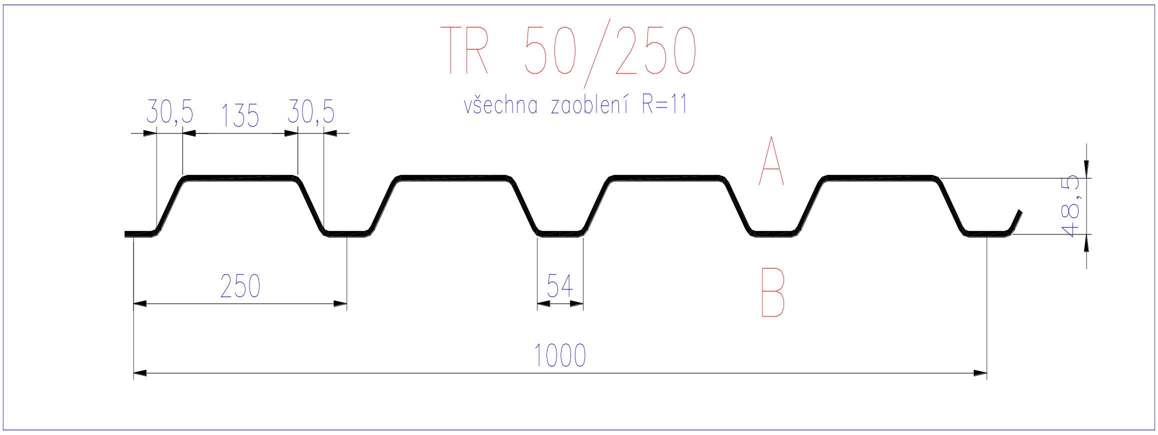


Fig. 5.3. Geometry of the chosen trapezoidal sheet.

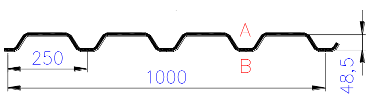
PROFILE	Thickness	Weight	CROSS SECTION		EFFECTIVE CROSS SECTION			
	t	m	A_g	$I_{y,g}$	$W_{y,eff}^{+}$	$W_{y,eff}^{-}$	$I_{y,eff}^{+}$	$I_{y,eff}^{-}$
	[mm]	[kg/m²]	[mm²]	[mm⁴]	[mm³]	[mm³]	[mm⁴]	[mm⁴]
				x10⁶	x10³	x10³	x10⁶	x10⁶
TR 50/250	0,63	6,35	754	0,295	5,90	5,90	0,164	0,208
	0,75	7,55	898	0,352	8,04	8,03	0,212	0,272
	0,88	8,86	1053	0,413	10,24	10,57	0,262	0,347
	1,00	10,07	1197	0,469	12,43	12,83	0,311	0,413
	1,13	11,38	1352	0,530	14,99	15,20	0,365	0,484
	1,25	12,59	1496	0,586	17,05	17,47	0,424	0,550

Table 5.1. Properties of TR 5O/250.

ONCE THE MAXIMUM MOMENT IS CALCULATED, IT IS TIME TO SIZE THE SHEET TO USE. AS IT IS SAID BEFORE, WE ARE GOING TO USE A TR 50/250 SO THE LAST STEP IS TO DETERMINATE THE THICKNESS OF IT WHICH WILL RESIST THE LOADS.

ULS: FIRSTY, A PROFILE WITH A THICKNESS OF 1 mm IS ASUMED.

$$M_a = \frac{W_a \cdot f_y}{\gamma_{ma}} \geq M_{sd} \qquad M_a = \frac{12{,}43 \cdot 10^3 \cdot 235}{1{,}0} = 2{,}92 \text{ kN·m} \geq M_{sd} = 3{,}23 \text{ kN·m} \quad ✗$$

THE CONDITION IS NOT FULFILLED, THEREFORE THE THICKNESS MUST BE INCREASED.

A THICKNESS OF 1.13 mm IS ASUMED NOW.

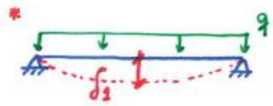
$$M_a = \frac{14{,}99 \cdot 10^3 \cdot 235}{1{,}0} = 3{,}52 \text{ kN·m} \geq M_{sd} = 3{,}23 \text{ kN·m} \quad ✓$$

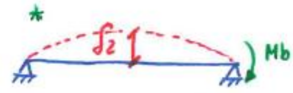
SLS: ONLY PERMANENT LOADS AND WITHOUT BEING WEIGHED.

$$\cdot\ 0{,}1054 \cdot q_k \cdot \ell^2 = 0{,}1054 \cdot (1{,}73 \text{ kN/m}) \cdot (2{,}5 \text{ m})^2 = 1{,}14 \text{ kN·m} = 1{,}14 \cdot 10^6 \text{ N·mm} = M_b$$

$$\delta \leq L/250 \qquad \delta = \frac{1}{E_a \cdot I_a}\left(\frac{5}{384} \cdot \overset{*}{q} \cdot L^4 - \frac{1}{16}\overset{*}{M_b} \cdot L^2\right) =$$

$$= \frac{1}{210 \cdot 10^3 \cdot 365 \cdot 10^3} \cdot \left(\frac{5}{384} \cdot 1{,}73 \cdot 2500^4 - \frac{1}{16} \cdot 1{,}14 \cdot 10^6 \cdot 2500^2\right) = 5{,}67 \text{ mm} \leq L/250 = 2000/250 = 8 \text{ mm} \quad ✓$$





CHECK IN CATALOGUE:

CATALOGUE TR 50/250 POSITIVE ⇒ Multi-spam beam $\left\{ \begin{array}{l} \text{SEPARATION: } 2{,}5 \text{ m} \\ \text{THICKNESS: } 1{,}13 \text{ mm} \end{array} \right\}$

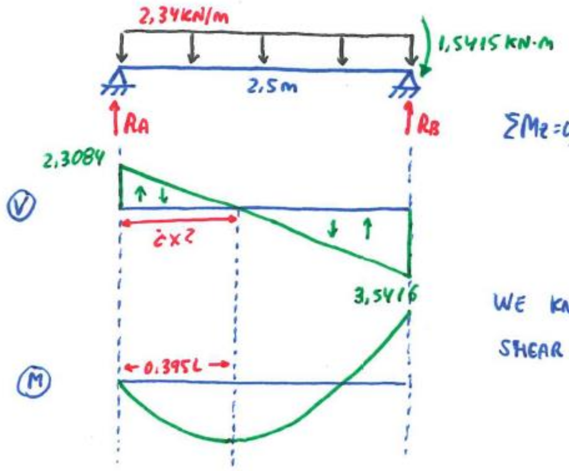
↳ MAXIMUM CAPACITY: 6,06 kN/m

$$M_{sd} = 3{,}23 \text{ kN·m} \ ; \quad M_{sd} = 0{,}1054 \cdot q_{eq} \cdot \ell^2 \Rightarrow q_{eq} = 4{,}90 \text{ kN/m} \leq 6{,}06 \text{ kN/m} \quad ✓$$

$$\boxed{\text{TR } 50/250 - 1{,}13 \text{ mm POSITIVE}}$$

IF THE MAXIMUM MOMENT DOES NOT MATCH IN A SUPPORT AND WE WOULD LIKE TO KNOW THE EXACT POINT WHERE IT IS PRODUCED, WE WILL HAVE TO WORK WITH THE BEAM SECTION WHERE IT WAS PRODUCED AND ADD THE MOMENTS THAT CORRESPOND IN THE SUPPORTS.

I AM GOING TO EXPLAIN IT BY MAKING AN EXAMPLE OF THE FIRST SECTION OF THE CONCRETE LOAD.



$\Sigma F_v = 0: \quad R_A + R_B = 2{,}34\ kN/m \cdot 2{,}5\ m$

$\Sigma (M_z = 0)_A: \quad 2{,}34\ kN/m \cdot 2{,}5\ m \cdot 1{,}25\ m + 1{,}5415\ kN \cdot m = R_B \cdot 2{,}5\ m.$

$\underline{R_A = 2{,}3084\ kN}$ $\quad$ $\underline{R_B = 3{,}5416\ kN}$

WE KNOW BECAUSE OF THE THEORY THAT WHEN THE SHEAR STRESS IS ZERO, THE BENDING MOMENT IS MAXIMUM.

THEN IT IS AS EASY AS FINDING THAT POINT IN THE SHEAR DIAGRAM USING EQUIVALENT TRIANGLES.

$2{,}3084 \longrightarrow x$

$3{,}5416 \longrightarrow 2{,}5 - x$

$\underline{\underline{x = 0{,}9865}}$ $\quad$ $0{,}9865 / 2{,}5\ m = \underline{\underline{0{,}395\ L}}$ ✓

5.2. CROSS BEAMS

5.2.1. DURING FABRICATION

DURING FABRICATION

• TR 50/250-1,13mm WEIGHT:

THICKNESS: 1,13 mm → WEIGHT: 11,38 kg/m^2 = 0,114 KN/m^2

0,114 KN/m^2 x 2,5m (SEPARATION). = 0,285 KN/m (PERMANENT LOAD).

$q_K = 0{,}285$ KN/m $\gamma_G = 1{,}35$ $\boxed{q = 0{,}285\ KN/m \cdot 1{,}35 = 0{,}38\ KN/m}$

• WET CONCRETE LOAD:

WEIGHT: 2600 Kg/m^3 (DENSITY) x 2,5m (SEPARATION) x $(50+16{,}4)\cdot 10^{-3}$m (THICKNESS) = 4,32 KN/m

$q_K = 4{,}32$ KN/m (PERMANENT LOAD) $\gamma_G = 1{,}35$ $\boxed{q = 1{,}35 \cdot (4{,}32\ KN/m) = 5{,}83\ KN/m}$

AS IT WAS SAID BEFORE IN THE PROJECT, BEAMS ARE MADE FROM IPE PROFILES.

WE HAVE TO CONSIDERE THE LOAD OF THE BEAM, BUT WE DON'T KNOW YET WHICH IS THE IPE PROFILE THAT FITS BETTER WITH THE TOTAL LOAD.

AN IPE-200 IS ASUMED.

• IPE-200 LOAD:

WEIGHT: 22,4 kg/m ≃ 0,22 KN/m (PERMANENT LOAD).

$q_K = 0{,}22$ KN/m $\gamma_G = 1{,}35$ $\boxed{q = 1{,}35 \cdot (0{,}22\ KN/m) = 0{,}30\ KN/m}$

• FABRICATION LOAD: THE CODE SAYS THAT WE HAVE TO CONSIDER:

• 1,5 KN/m^2 IN 3x3m AROUND THE UNFAVORABLE POINT.

• 0,75 KN/m^2 IN THE REMAINING AREA.

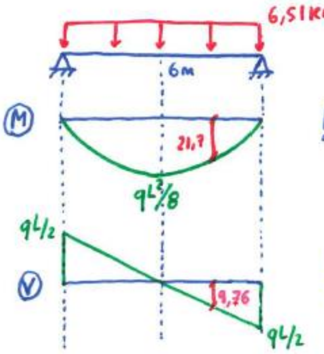
- INSIDE 3x3m: 1,5 KN/m^2 x 2,5m (SEPARATION) = 3,75 KN/m (VARIABLE LOAD).

$q_K = 3{,}75$ KN/m $\gamma_Q = 1{,}5$ $\boxed{q = 1{,}5 \cdot (3{,}75\ KN/m) = 5{,}63\ KN/m}$

- OUTSIDE: 0,75 KN/m^2 x 2,5m = 1,875 KN/m (VARIABLE LOAD).

$q_K = 1{,}875$ KN/m $\gamma_Q = 1{,}5$ $\boxed{q = 1{,}5 \cdot (1{,}875\ KN/m) = 2{,}81\ KN/m}$

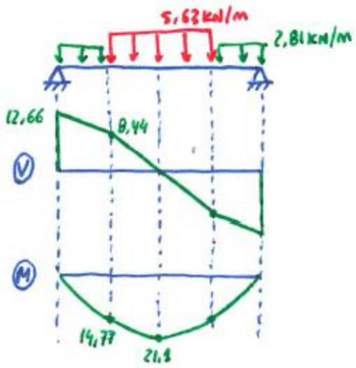
PERMANENT LOADS: 0,38 kN/m + 5,83 kN/m + 0,3 kN/m = 6,51 kN/m



$$M_{max} = \frac{6{,}51\ kN/m \cdot (6m)^2}{8} = 29{,}3\ kN \cdot m$$

$$V_{max} = \frac{6{,}51\ kN/m \cdot 6m}{2} = 19{,}53\ kN$$

VARIABLE LOADS:

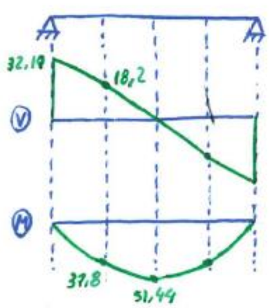


$\Sigma F_v = 0$; $2R_A = 2{,}81\ kN/m \cdot (1{,}5 + 1{,}5)m + 5{,}63\ kN/m \cdot 3m$

$\Sigma M_z = 0)_A$; $R_A = R_B$

$R_A = R_B = 12{,}66\ kN$

JOINING BOTH DIAGRAMS BY OVERLAPPING:



$\boxed{V_{sd} = 32{,}19\ kN}$ $\boxed{M_{sd} = 51{,}44\ kN \cdot m}$

ULS:

$\underline{A_v} = 1{,}04 \cdot h \cdot t_w = 1{,}04 \cdot 200\,mm \cdot 5{,}6\,mm = \underline{1165\,mm^2}$

$V_a = A_v \cdot \frac{f_y}{\gamma_a \cdot \sqrt{3}} = 1165\,mm^2 \cdot \frac{235\,N/mm^2}{1{,}0 \cdot \sqrt{3}} = 158{,}07\,kN \geq 2 \cdot V_{sd} = 2 \cdot 32{,}19 = 64{,}38\,kN$ ✓

$M_a = \frac{W_a \cdot f_y}{\gamma_{Ma}} = \frac{220 \cdot 10^3\,mm^3 \cdot 235\,N/mm^2}{1{,}0} = 51{,}70\,kN \cdot m \geq M_{sd} = 51{,}44\,kN$ ⊗

THE MAXIMUM MOMENT ACCEPTABLE COINCIDE WITH THE MAXIMUM WE HAVE.

WE NEED A BIGGER PROFILE ⟶ IPE-220

THE PERMANENT LOADS DO NOT CHANGE BECAUSE THE WEIGHT DIFFERENCE OF PUTTING A PROFILE OR THE HIGHER ONE IS INAPPRECIABLE.

$\underline{A_v} = 1{,}04 \cdot h \cdot t_w = 1{,}04 \cdot 220\,mm \cdot 5{,}9\,mm = \underline{1350\,mm^2}$

$V_a = A_v \cdot \frac{f_y}{\gamma_a \cdot \sqrt{3}} = 1350\,mm^2 \cdot \frac{235\,N/mm^2}{1{,}0 \cdot \sqrt{3}} = 183{,}16\,kN \geq 2 \cdot V_{sd} = 2 \times 32{,}19 = 64{,}38\,kN$ ✓

$M_a = \frac{W_a \cdot f_y}{\gamma_{Ma}} = \frac{285 \cdot 10^3\,mm^3 \cdot 235\,N/mm^2}{1{,}0} = 67\,kN \cdot m \geq M_{sd} = 51{,}44\,kN$ ✓

SLS:

- $\delta = \frac{1}{E_a \cdot I_a} \cdot \left(\frac{5}{384} \cdot q \cdot L^4 \right) = \frac{1}{210 \cdot 10^3 \cdot 27{,}7 \cdot 10^6} \cdot \left(\frac{5}{384} \cdot 4{,}82 \cdot 6000^4 \right) = 14\,mm \leq \frac{L}{250} = \frac{6000\,mm}{25} = 24\,mm$ ✓
- $\delta = 14\,mm \leq 20\,mm$ ✓

$\boxed{IPE\text{-}220}$

5.2.2. NORMAL USE

VARIABLE LOADS:

Category of use		Subcategory of use		$q_k[kN/m^2]$	$Q_k[kN]$
A	Residential Areas	A1	Housing and room areas in, hospitals and hotels	2	2
		A2	Storerooms	3	2
B	Administrative areas			2	2
C	Areas of public access (except for areas belonging to categories A, B and D)	C1	Zones with tables and chairs	3	4
		C2	Areas with fixed seats	4	4
		C3	Zones without obstacles that prevent the free movement of the people like vestibules of public buildings, administrative, hotels; Exhibition halls in museums; etc.	5	4
		C4	Areas for gymnasium or physical activities	5	7
		C5	Agglomeration areas (concert halls, stadiums, etc.)	5	4
D	Shopping area	D1	Shops	5	4
		D2	Supermarkets, hypermarkets or large surfaces	5	7
E	Traffic and parking areas for light vehicles (total weight <30 kN)			2	20
F	Passable covers only accessible privately			1	2
G	Covers accessible only for conservation	G1	Covers with inclination less than 20º	1	2
			Lightweight covers on straps (without slabs)	0,4	1
		G2	Covers with inclination over 40º	0	2

Table 5.2. Table from the code where the different categories for overload of use are shown.

Provided that a floor allows a lateral distribution of loads, the SELF-WEIGHT OF MOVABLE PARTITIONS may be taken into account by a uniformly distributed load q_k which should be added to the imposed loads of floors obtained from *Table 5.2*.

This uniformly distributed load is dependent on the self-weight of the partitions as follows:

• For movable partitions with a self-weight ≤ 1,0 kN/m wall length: $q_k = 0{,}5\ kN/m^2$

• For movable partitions with a self-weight ≤ 2,0 kN/m wall length: $q_k = 0{,}8\ kN/m^2$

• For movable partitions with a self-weight ≤ 3,0 kN/m wall length: $q_k = 1{,}2\ kN/m^2$

Heavier partitions should be considered in the design taking account of the locations and directions of the partitions and the structural form of the floors.

In the project in will be considered a load of $q_k = 1{,}2\ kN/m^2$ which is the biggest so we are in the safe side which means the heaviest walls can be built into the structure.

• OVERLOADS OF USE: CATEGORY C1 → $q_k = 3\,kN/m^2$; $3\,kN/m^2 \times 2{,}5\,m\,(Sep) = \underline{7{,}5\ kN/m}$

• MOVABLE PARTITIONS: $q_k = 1{,}2\,kN/m^2$; $1{,}2\,kN/m^2 \times 2{,}5\,(Sep) = \underline{3\ kN/m}$

PERMANENT LOADS:

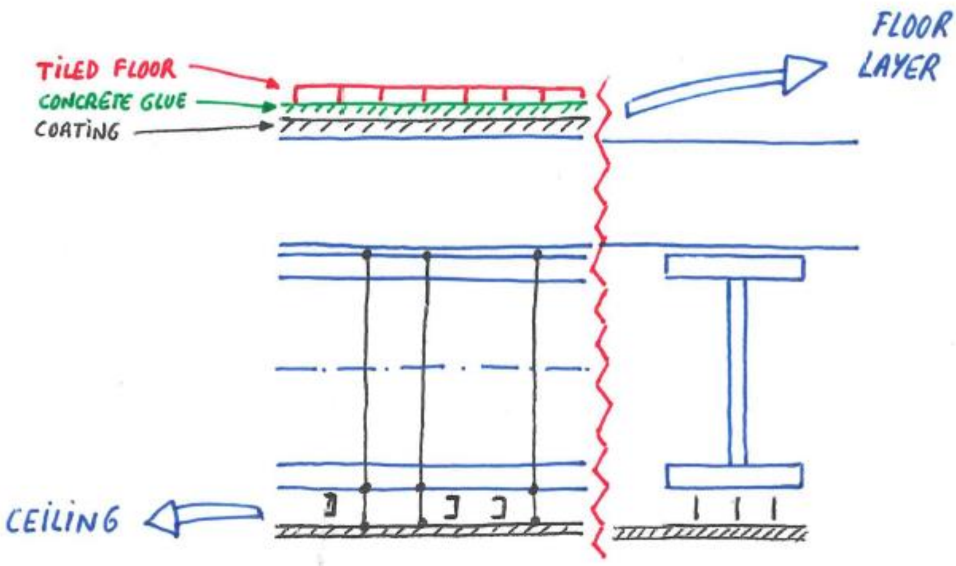


Fig. 5.4. Scheme of the floor layer & ceiling permanent loads acting on the cross beams.

- IPE-220: $26,2\ kg/m = \underline{\underline{0,262\ kN/m}}$

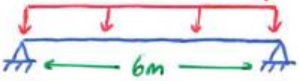
- TR 50/250 (1,13 mm): $11,38\ kg/m^2 \times 2,5\ m\ (Sep) = 28,45\ kg/m = \underline{\underline{0,285\ kN/m}}$

- CONCRETE SLAB: $(50+16,4)\cdot 10^{-3}\ m \times 2500\ kg/m^3\ (\text{DRY CONCRETE}) \times 2,5\ m = 415\ kg/m = \underline{\underline{4,15\ kN/m}}$

- CEILING & FLOOR LAYER:

$$\left.\begin{array}{l} - 2300\ kg/m^3 \times 80\ mm\ (\text{thickness}) \times 2,5\ m\ (Sep) = 460\ kg/m = 4,6\ kN/m \\ - 1400\ kg/m^3 \times 20\ mm\ (\text{thickness}) \times 2,5\ m\ (Sep) = 70\ kg/m = 0,7\ kN/m \end{array}\right\}\ \underline{\underline{5,3\ kN/m}}$$

$$\underline{\underline{q_{TOT}}} = 1,35\cdot(0,262+0,285+4,15+5,3)\ kN/m + 1,5\cdot(7,5+3)\ kN/m = \underline{\underline{29,25\ kN/m}}$$



$$\underline{\underline{Msd}} = \frac{q\cdot l^2}{8} = \frac{29,25\ kN/m\cdot(6\ m)^2}{8} = \underline{\underline{131,6\ kN\cdot m}}$$

$$\underline{\underline{Vsd}} = \frac{q\cdot l}{2} = \frac{29,25\ kN/m\cdot 6\ m}{2} = \underline{\underline{87,75\ kN}}$$

ULS

a) SHEAR

$$V_{rd} = A_v \cdot \frac{f_y}{\gamma_a \cdot \sqrt{3}} = 1350\,mm^2 \cdot \frac{235\,N/mm^2}{1{,}0 \cdot \sqrt{3}} = 183.16\,kN > 2 \cdot V_{sd} = 2 \cdot 87{,}75\,kN = 175{,}5\,kN$$ ✓

b) BENDING

$$b_{eff} = \frac{2 \cdot L}{8} = \frac{2 \cdot 6000\,mm}{8} = 1500\,mm < 2500\,mm \text{ (DISTANCE BETWEEN BEAMS)}$$ ✓

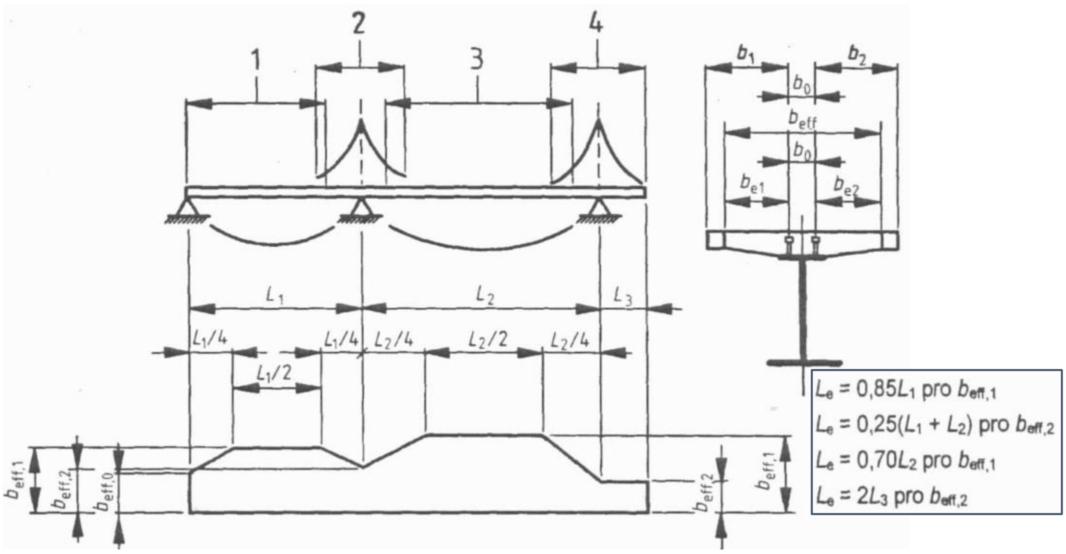


Fig. 5.5. Possible cases of b_{eff} depending on the arrangement in the beam.

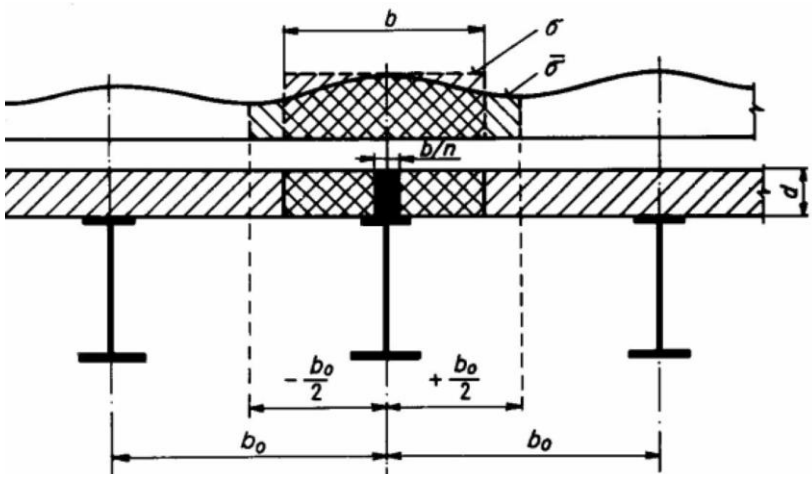
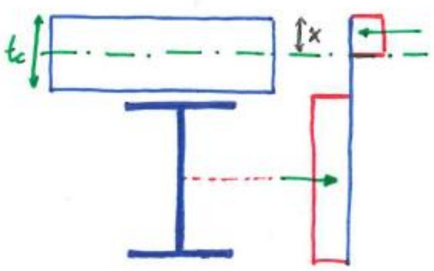


Fig. 5.6. Real stress distribution because of shear leg, and approximation that is used.

ASSUMPTION: The neutral axis passes through the concrete slab.

IT WILL BE DENOTED "X" AS THE DISTANCE FROM THE TOP EDGE OF THE CONCRETE SLAB TO THE NEUTRAL AXIS.

$$\underline{x} = \frac{\frac{A_a \cdot f_y}{\gamma_a}}{\frac{b_{eff} \cdot 0{,}85 \cdot f_{ck}}{\gamma_c}} = \frac{\frac{3340\,mm^2 \cdot 235\,N/mm^2}{1}}{\frac{1500\,mm \cdot 0{,}85 \cdot 20\,N/mm^2}{1{,}5}} = \underline{46{,}17\,mm} \leq 50\,mm \text{ ASSUMPTION}$$



t_c ≡ thickness of the concrete slab = 50 mm

h_p ≡ high of the TR. = 50 mm

$$\underline{\underline{h_a}} = (t_c + h_p) + \frac{h_{IPE\,220}}{2} = 100 + \frac{220}{2} = \underline{\underline{210\,mm}}$$

$$M_{rd} = \frac{A_a \cdot f_y}{\gamma_a} \cdot \left(h_a - \frac{x}{2}\right) = \frac{3340\,mm^2 \cdot 235\,N/mm^2}{1{,}0}\left(210\,mm - \frac{46{,}17\,mm}{2}\right) = 146{,}7\,kN \cdot m$$

$$M_{rd} > M_{sd} = 131{,}6\,kN \cdot m$$

AS WE SEE, THERE IS A LOT OF DIFFERENCE BETWEEN THE RESISTANCE OF STEEL STRUCTURE ($M_{pl,Rd} = 67\,kN \cdot m$) AND THE COMPOSITE STRUCTURE ($M_{rd} = 146{,}7\,kN \cdot m$). THAT HAPPENS BECAUSE THE MOMENT OF INERTIA INCREASES IF THE CONCRETE WORKS FULLY IN COMPRESSION, IN THE RIGHT POSITION.

c) SHEAR CONNECTION

SHEAR STUDS

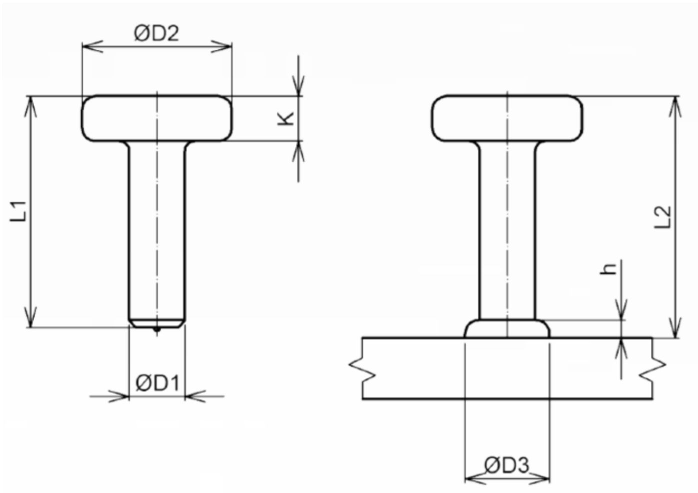


Fig. 5.7. Shear Studs

Tensile strength: R_m	≥ 450 N/mm²
Yield strength: R_{el}	≥ 350 N/mm²
Elongation: A_5	≥ 15%

ISO 13918

Steel St 37-3K

d	l_2+1	D_1	D_2	D_3	h	k
10	50,75,100,125,150,175	10	19	13	2,5	7
13	50,75,100,125,150,175,200	13	25	17	3	8
16	50,75,100,125,150,175,200,225,250	16	32	21	4,5	8
19	50,75,100,125,150,175,200,225,250,275,300,325,350	19	32	23	6	10
22	50,75,100,125,150,175,200,225,250,275,300,325,350	22	35	29	6	10
25	75,100,125,150,175,200,225,250,275,300,325,350	25	40	31	7	12

Table 5.3. Specific dimensions of different types of shear studs.

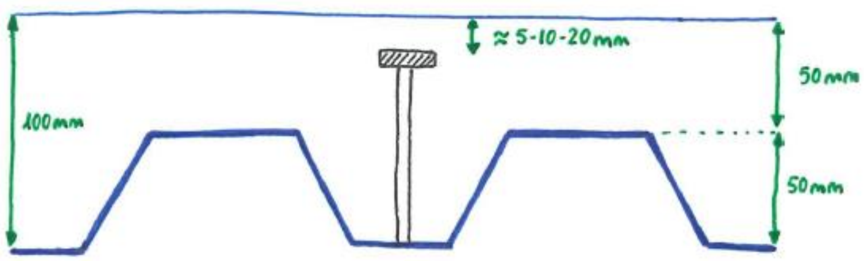


Fig. 5.8. Scheme of the position of the stud compared to the trapezoidal sheet.

$h_{sc} = 90mm$

SHEAR RESISTANCE OF THE STUD

$$P_{RK} \begin{cases} 0,8 \cdot f_u \cdot \frac{\pi \cdot d^2}{4} = 0,8 \cdot 360N/mm^2 \cdot \frac{\pi \cdot (19mm)^2}{4} = 81,65\ kN \\ 0,29 \cdot \alpha \cdot d^2 \cdot \sqrt{f_{ck} \cdot E_{cm}} = 0,29 \cdot 1,0 \cdot (19mm)^2 \cdot \sqrt{20N/mm^2 \cdot 30 \cdot 10^3 N/mm^2} = 81,1\ KN \end{cases}$$

CRACK OF THE CONCRETE

$$\alpha \begin{cases} 0,2 \cdot \left(\frac{h_{sc}}{d} + 1\right) & \text{IF } 3 \leq \frac{h_{sc}}{d} \leq 4 \\ \underline{1,0} & \end{cases} \qquad \frac{h_{sc}}{d} = \frac{90}{19} = 4,73$$

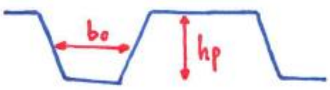
THE MOST RESTRICTIVE OPTION IS CHOSEN $\longrightarrow \underline{P_{RK} = 81,1\ kN}$

$$\underline{P_{Rd}} = \frac{P_{RK}}{\gamma_v} = \frac{81,1 \cdot 10^3 N}{1,25} = \underline{64,88\ KN}$$

REDUCTION DUE TO SHEETING WAVES:

$$k_t = \frac{0,7}{\sqrt{n_r}} \cdot \frac{b_o}{h_p} \cdot \frac{h_{sc} - h_p}{h_p}$$

{ IT IS GOING TO BE ASUMMED : NUMBER OF STUDS IN ONE WAVE : 1. }



$$k_t = \frac{0,7}{\sqrt{1}} \cdot \frac{84,5}{50} \cdot \frac{90-50}{50} = 0,946 < 1,0$$

THERE IS A REDUCTION BUT VERY SMALL.

- PLASTIC SOLUTION :

STEEL IN TENSION

$$F_{cf} \begin{cases} F_{cf,a} = \frac{A_a \cdot f_y}{\gamma_a} = \frac{3340mm^2 \cdot 235N/mm^2}{1,0} = 784,9 \cdot 10^3 N \\ F_{cf,c} = \frac{A_c \cdot 0,85 \cdot f_{ck}}{\gamma_c} + \cancel{\frac{A_s \cdot f_{sk}}{\gamma_s}} = \frac{50 \cdot 1500\ 0,85 \cdot 20}{1,5} = 850 \cdot 10^3 N \end{cases}$$

neglectible

$\underline{A_c = \text{thickness} \times b_{eff}}$

CONCRETE IN COMPRESSION

THE MOST RESTRICTIVE OPTION IS CHOSEN $\longrightarrow \underline{F_{cf,a} = 784,9 \cdot 10^3 N}$

$$N_f = \frac{F_{cf,a}}{P_{Rd,r}} = \frac{784{,}9}{61{,}37} = 12{,}79 \longrightarrow 13 \text{ head studs in each half of the beam.}$$

26 IN TOTAL.

$$P_{Rd,r} = P_{Rd} \times k_t = 64{,}88 \times 0{,}946 = 61{,}37 \text{ KN}$$

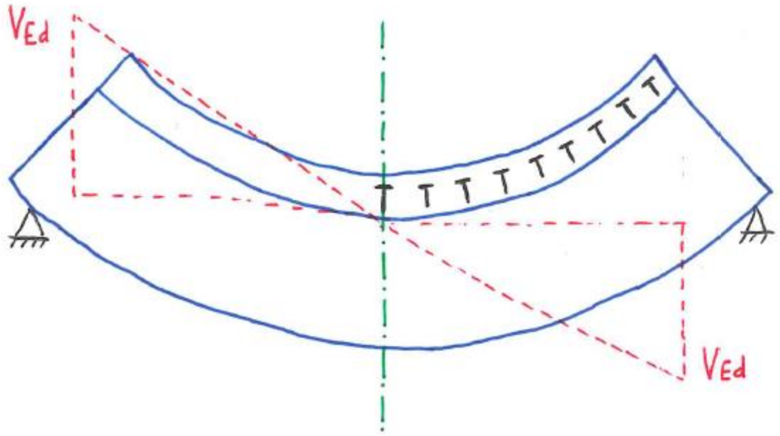


Fig. 5.9. Representation of the shear force acting on the head studs.

$$\frac{3000\,mm}{13} = 230{,}8\,mm$$

IT IS NECESSARY TO PUT A STUD EVERY 230,8 mm, BUT THAT IS NOT POSSIBLE. BECAUSE OF THE GEOMETRY OF THE TRAPEZOIDAL SHEET, THE MINIMUN SEPARATION DISTANCE IS 250 mm.

The total number of studs that can be placed in each row in one half of the beam is:

$$\frac{3000\,mm}{N_{f,max}} = 250\,mm \longrightarrow N_{f,max} = 12\ head\ studs$$

At this point, it is necessary to solve the problem that is found. Therefore, there are two main options to answer it.

The first is to replace the single row of connectors, by two rows, i.e. multiply the number of connectors by two and ensure that we will comply with safety checks.

However, this option is underused because it is not economical.

However, it is shown below the calculations and steps to be followed if this option were to be carried out.

***1º OPTION:* INCREASE THE NUMBER OF STUDS**

$n_r = 2$ $\quad k_t = \frac{0{,}7}{\sqrt{n_r}} \cdot \frac{b_o}{h_p} \cdot \frac{h_{sc} - h_p}{h_p} = \frac{0{,}7}{\sqrt{2}} \cdot \frac{84{,}5}{50} \cdot \frac{90-50}{50} = 0{,}66$ {BIGGER REDUCTION}

$P_{Rd,r} = P_{Rd} \cdot k_t = 64{,}88 \cdot 0{,}66 = 43{,}21 \text{ kN}$

$N_f = \frac{F_{cf,a}}{P_{Rd,r}} = \frac{784{,}9}{43{,}21} = 18{,}16 \longrightarrow 20$ head studs in each half of the beam.

(10 studs each row and half of the beam).

But it is known that it is possible to set 12 instead of 10, so they will be put a total of 24 studs each half.

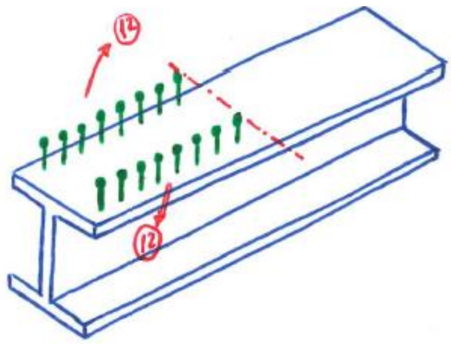


Fig. 5.10. Position of the two rows of studs on the beam.

The second option is based on changing the calculation assumption as if we had a total shear connection, and change it by a partial shear connection.

It is known that with this option not all the concrete will be working, but that is not essential, as long as the verifications that are explained below are fulfilled.

***2º OPTION:* PARTIAL SHEAR CONNECTION**

$\boxed{\eta = \frac{n}{n_f} \geq 0{,}4}$ THIS CONDITION MUST BE FULFILLED TO BE A COMPOSITE STRUCTURE. IT MEANS THAT MORE THAN THE 40% OF CONCRETE IS WORKING.

$\rightarrow n_{min} = 0{,}4 \cdot n_f = 0{,}4 \cdot 13 = 5{,}2 \longrightarrow 6$ studs in each half at least.

FORCE THAT HAS TO BE TRANSFERED:

$$F_c = \frac{M_{sd} - M_{apl,Rd}}{M_{pl,Rd} - M_{apl,Rd}} \cdot F_{cf,pl}$$

• $M_{sd} = 131{,}6$ kN • $M_{apl,Rd} = 67$ kN • $M_{pl,Rd} = 146{,}7$ kN • $F_{cf,pl} = 784{,}9$ kN

$$\underline{F_c} = \frac{(131{,}6 - 67) \cdot 784{,}9}{146{,}7 - 67} = \underline{636{,}2 \text{ kN}}$$

$M_{apl,Rd}$ = {THE FORCE THAT THE IPE PROFILE CAN RESIST BY ITSELF}.

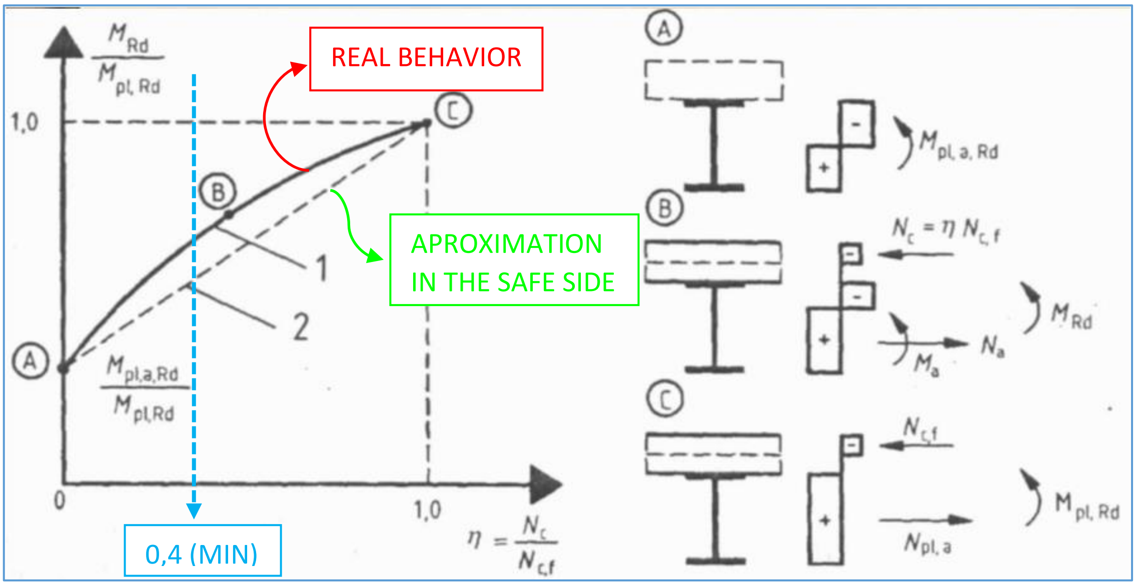


Fig. 5.11. Real behaviour compared to the approximation in a partial shear connection.

$$N_{f,reduc.} = \frac{F_{cf}}{P_{rd,r}} = \frac{636{,}2}{61{,}37} = 10{,}39 \rightarrow 11 \text{ studs in each half.}$$

- MORE THAN 6 WHICH MEANS IT IS A COMPOSITE STRUCTURE. ✓
- THERE IS SPACE FOR 12 SO THAT IS NUMBER OF STUDS WE WILL PUT TO BE IN THE SAFE SIDE.

<u>12 studs in each half.</u>

SLS

$$E_{cm} = 30\ GPa \longrightarrow E'_c = \frac{E_{cm}}{2} = \frac{30}{2} = 15\ GPa$$

$$n = \frac{E_a}{E'_c} = \frac{210}{15} = 14$$

$$h_a = t_c + h_{TR} + \frac{h_{IPE}}{2} = 50 + 50 + \frac{220}{2} = 210\ mm$$

$$A_a = 3340\ mm^2$$

$$t_c = 50\ mm$$

$$b_{eff} = 1500\ mm$$

$$\underline{\underline{x}} = \frac{A_a \cdot h_a + \frac{1}{n} \cdot t_c \cdot b_{eff} \cdot \frac{t_c}{2}}{A_a + \frac{1}{n} \cdot t_c \cdot b_{eff}} = \frac{3340 \cdot 210 + \frac{1}{14} \cdot 50 \cdot 1500 \cdot \frac{50}{2}}{3340 + \frac{1}{14} \cdot 50 \cdot 1500} = \underline{\underline{96\ mm}}$$

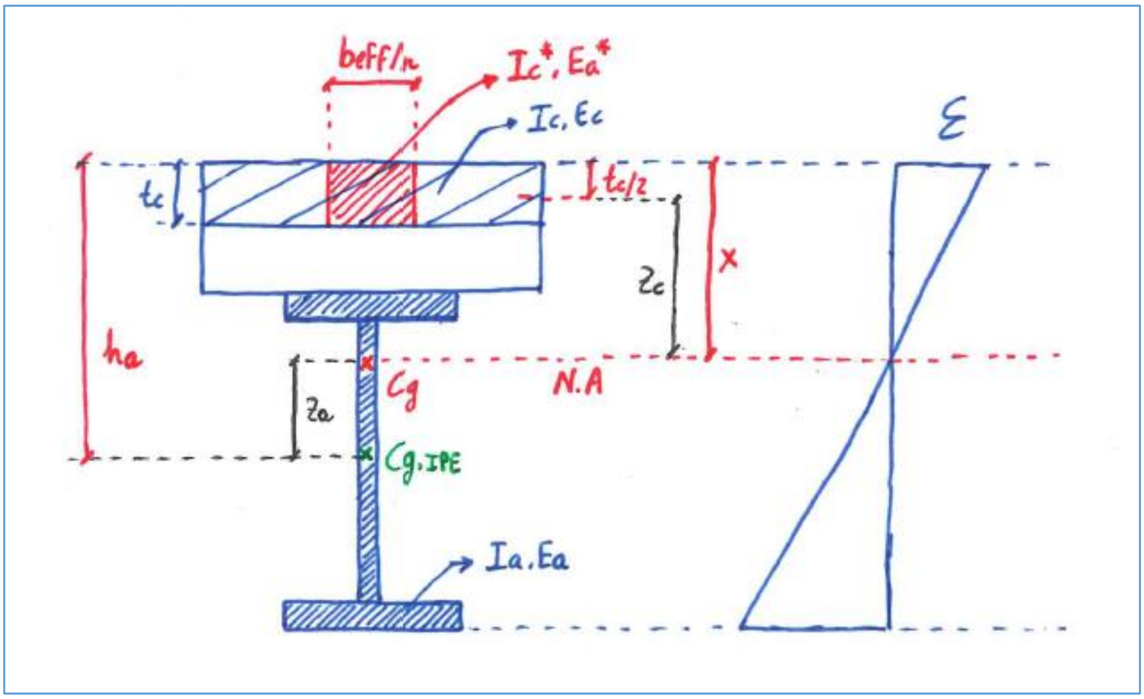


Fig. 5.12 Dimensions explained necessary for the calculation of the S.L.S.

$$I_i = I_a + A_a \cdot z_a^2 + \frac{1}{n}\left(\frac{1}{12} \cdot b_{eff} \cdot t_c^3 + t_c \cdot b_{eff} \cdot z_c^2\right) = 27{,}72 \cdot 10^6 + 3340 \cdot 114^2 + \frac{1}{14} \cdot \left(\frac{1}{12} \cdot 1500 \cdot 50^3 + 50 \cdot 1500 \cdot 71^2\right)$$

$$z_a = h_a - x = 210 - 96 = 114\ mm$$

$$z_c = x - \frac{t_c}{2} = 96 - \frac{50}{2} = 71\ mm$$

$$\underline{I_i = 99{,}25 \cdot 10^6\ mm^4}$$

TO CALCULATE THE CHARACTERISTIC LOAD THEY WILL BE CONSIDERED THE VARIABLE LOADS AND THE CEILING & FLOOR LAYERS LOADS.

$$q_k = (7{,}5 + 3) + (5{,}3)\ kN/m = \underline{15{,}8\ kN/m}$$

$$\delta = \frac{5 \cdot q_k \cdot L^4}{384 \cdot E \cdot I_i} = \frac{5 \cdot 15{,}8\ N/mm \cdot (6000\ mm)^4}{384 \cdot 210 \cdot 10^3\ N/mm^2 \cdot 99{,}25 \cdot 10^6\ mm^4} = 12{,}8\ mm \leq \frac{L}{300} = 20\ mm\ ✓$$

BUT IT IS NECCESARY TO ADD THE PARTIAL CONNECTION CONSIDERATION.

$$\delta_a = \delta \cdot \frac{I_i}{I_a} = 12{,}8 \cdot \frac{74{,}05}{27{,}72} = 34{,}19\ mm$$

$$\delta_{TOTAL} = \delta \cdot \left[1 + 0{,}3 \cdot \left(1 - \frac{n}{n_f}\right) \cdot \left(\frac{\delta_a}{\delta} - 1\right)\right] = 12{,}8 \cdot \left[1 + 0{,}3 \cdot \left(1 - \frac{12}{13}\right) \cdot \left(\frac{34{,}19}{12{,}8} - 1\right)\right] = 13{,}29\ mm < 20\ mm\ ✓$$

NOW WE PROVE IF CEILING IS NECCESARY OR NOT.

$\delta_{during\ fabrication} + \delta_{normal\ use} > 20\ mm \Rightarrow$ CEILING NECCESARY.

$\rightarrow$ $14\ mm + 13{,}29\ mm = 27{,}29\ mm > 20\ mm \longrightarrow$ CEILING ✓

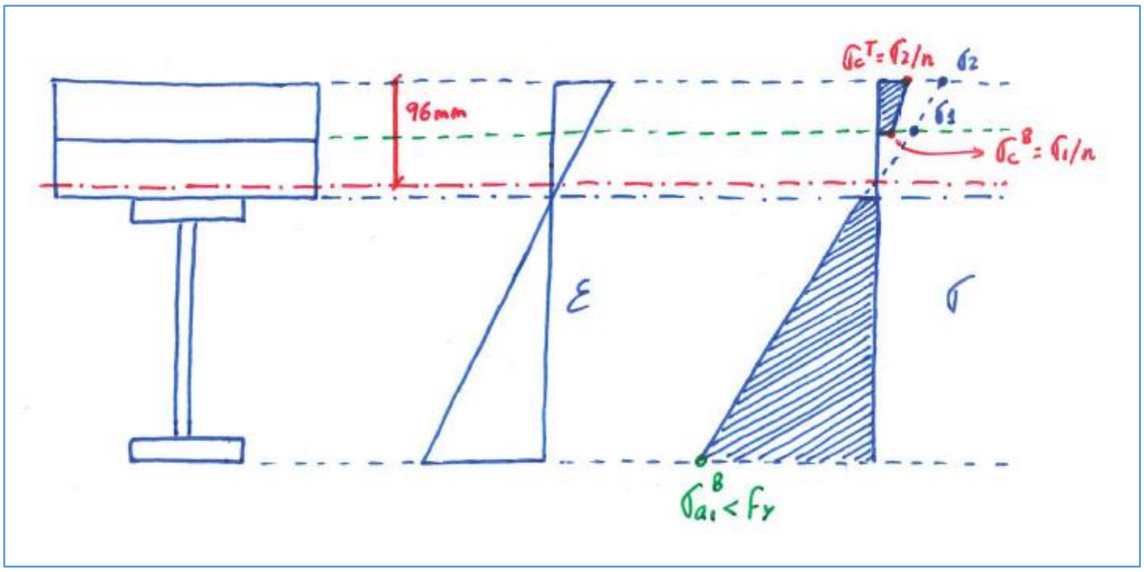
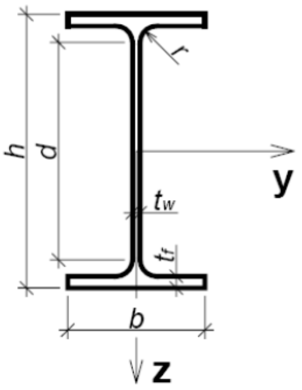


Fig. 5.13. Disposition of deformation and stresses in the composite beam.

5.2.3. PROFILE SELECTED



	G	h	b	t_w	t_f	r	d	A	$A_{v.z}$	I_y	W_y	$W_{pl.y}$	i_y
	kg/m	mm	mm	mm	mm	mm	mm	mm^2	mm^2	10^4 mm^4	10^3 mm^3	10^3 mm^3	mm
IPE 80	6, 0	80	46	3, 8	5, 2	5	59, 6	764	358	80, 14	20, 03	23, 22	32, 4
IPE 100	8, 1	100	55	4, 1	5, 7	7	74, 6	1032	509	171, 0	34, 20	39, 41	40, 7
IPE 120	10, 4	120	64	4, 4	6, 3	7	93, 4	1321	631	317, 8	52, 96	60, 73	49, 0
IPE 140	12, 9	140	73	4, 7	6, 9	7	112, 2	1643	764	541, 2	77, 32	88, 34	57, 4
IPE 160	15, 8	160	82	5, 0	7, 4	9	127, 2	2009	966	869, 3	108, 7	123, 9	65, 8
IPE 180	18, 8	180	91	5, 3	8, 0	9	146, 0	2395	1125	1317	146, 3	166, 4	74, 2
IPE 200	22, 4	200	100	5, 6	8, 5	12	159, 0	2848	1400	1943	194, 3	220, 6	82, 6
IPE 220	26, 2	220	110	5, 9	9, 2	12	177, 6	3337	1588	2772	252, 0	285, 4	91, 1
IPE 240	30, 7	240	120	6, 2	9, 8	15	190, 4	3912	1914	3892	324, 3	366, 6	99, 7
IPE 270	36, 1	270	135	6, 6	10, 2	15	219, 6	4595	2214	5790	428, 9	484, 0	112

I_z	W_z	$W_{pl.z}$	i_z	I_t	I_w	bending				compression				
10^4 mm^4	10^3 mm^3	10^3 mm^3	mm	10^4 mm^4	10^6 mm^6	S235	S275	S355	S460	S235	S275	S355	S460	
8, 489	3, 691	5, 818	10, 5	0, 6977	118, 0	1	1	1	1	1	1	1	1	IPE 80
15, 92	5, 789	9, 146	12, 4	1, 202	351, 4	1	1	1	1	1	1	1	1	IPE 100
27, 67	8, 646	13, 58	14, 5	1, 735	889, 6	1	1	1	1	1	1	1	1	IPE 120
44, 92	12, 31	19, 25	16, 5	2, 447	1981	1	1	1	1	1	1	1	2	IPE 140
68, 31	16, 66	26, 10	18, 4	3, 604	3959	1	1	1	1	1	1	1	2	IPE 160
100, 9	22, 16	34, 60	20, 5	4, 790	7431	1	1	1	1	1	1	2	3	IPE 180
142, 4	28, 47	44, 61	22, 4	6, 980	12990	1	1	1	1	1	1	2	3	IPE 200
204, 9	37, 25	58, 11	24, 8	9, 066	22670	1	1	1	1	1	1	2	4	IPE 220
283, 6	47, 27	73, 92	26, 9	12, 88	37390	1	1	1	1	1	2	2	4	IPE 240
419, 9	62, 20	96, 95	30, 2	15, 94	70580	1	1	1	1	2	2	3	4	IPE 270

5.3. MAIN BEAMS

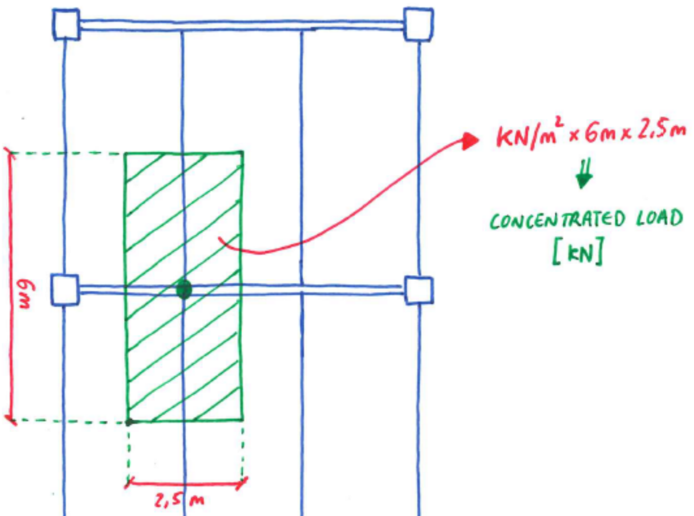
5.3.1. NORMAL USE

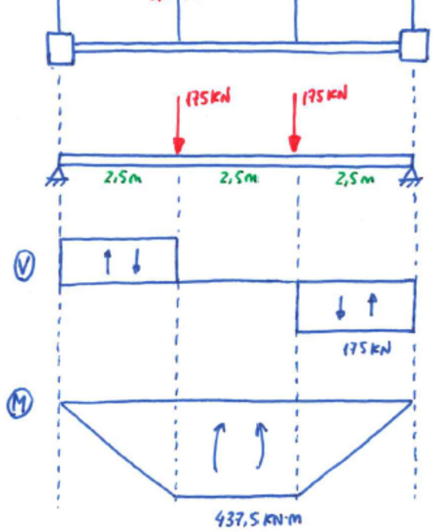
- SUMMARY OF LOADS

• VARIABLES
- • OVERLOADS OF USE : $3\,kN/m^2 \cdot (6m \cdot 2{,}5m) = 45\,kN$
- • MOVABLE PARTITIONS : $1{,}2\,kN/m^2\,(6m \cdot 2{,}5m) = 18\,kN$

• PERMANENTS
- • IPE-220 : $0{,}262\,kN/m \cdot (6m) = 1{,}58\,kN$
- • TR 50/250-1,13mm : $11{,}38\,kg/m^2 \cdot (6m \cdot 2{,}5m) = 1{,}71\,kN$
- • CONCRETE SLAB : $(56+16{,}4)\cdot 10^{-3}m \cdot (2500\,kg/m^3)\cdot(6m \cdot 2{,}5m) = 24{,}9\,kN$
- • CEILING & FLOOR LAYERS : $-\,2300\,kg/m^3 \cdot 80\,mm \cdot (2{,}5 \cdot 6m^2) = 27{,}6\,kN$
  $-\,1400\,kg/m^3 \cdot 20\,mm \cdot (2{,}5 \cdot 6m^2) = 4{,}2\,kN$

$$Q_{TOT} = 1{,}35 \cdot (45+18)\,kN + 1{,}5 \cdot (1{,}58 + 1{,}71 + 24{,}9 + 27{,}6 + 4{,}2)\,kN = 175\,kN$$





THE DISTRIBUITED LOAD OF THE PROFILE OF THE MAIN BEAM IS NEGLECTIVE COMPARED WITH THE CONCENTRATED LOADS. THEREFORE IT IS NOT CONSIDERED IN THE DIAGRAMS.

$V_{sd} = 175\,kN$

$M_{sd} = 437{,}5\,kN \cdot m$

ULS

a) SHEAR

AN IPE-300 IT IS GOING TO BE SUPPOSE IN THE MAIN BEAM.

$$A_v = 1{,}04 \cdot h \cdot t_w = 1{,}04 \cdot 300\,mm \cdot 7{,}1\,mm = 2215{,}2\,mm^2$$

$$V_a = A_v \cdot \frac{f_y}{\gamma_a \cdot \sqrt{3}} = 2215{,}2\,mm^2 \cdot \frac{235\,N/mm^2}{1{,}0 \cdot \sqrt{3}} = 300{,}5\,kN > 2 \cdot V_{sd} = 350\,kN \quad \otimes$$

AN IPE-400 IT IS GOING TO BE SUPPOSE.

$$A_v = 1{,}04 \cdot h \cdot t_w = 1{,}04 \cdot 400\,mm \cdot 8{,}6\,mm = 3577{,}6\,mm^2$$

$$V_a = A_v \cdot \frac{f_y}{\gamma_a \cdot \sqrt{3}} = 3577{,}6\,mm^2 \cdot \frac{235\,N/mm^2}{1{,}0 \cdot \sqrt{3}} = 485{,}4\,kN > 2 \cdot V_{sd} = 350\,kN \quad \checkmark$$

b) BENDING

$$b_{eff} = \frac{2 \cdot L}{8} = \frac{2 \cdot 7500}{8} = 1875\,mm$$

ASSUMPTION: N.A IN CONCRETE SLAB.

$$X = \frac{\frac{A_a \cdot f_y}{\gamma_a}}{\frac{b_{eff} \cdot 0{,}85 \cdot f_{ck}}{\gamma_c}} = \frac{\frac{8450 \cdot 235}{1{,}0}}{\frac{1875 \cdot 0{,}85 \cdot 20}{1{,}5}} = 93{,}45\,mm \leq 50\,mm \quad \otimes$$

N.A IN THE IPE PROFILE:

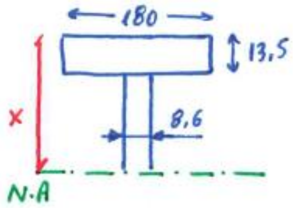
$$N_c = b_{eff} \cdot t_c \cdot 0{,}85 \cdot \frac{f_{ck}}{\gamma_c} = 1875 \cdot 50 \cdot 0{,}85 \cdot \frac{20}{1{,}5} = 1062{,}5\,kN$$

$$N_a = A_a \cdot \frac{f_y}{\gamma_{M0}} = 8450 \cdot \frac{235}{1{,}0} = 1984{,}8\,kN$$

$$N_c + 2 \cdot N_{a1} = N_a \Rightarrow N_{a1} = \frac{N_a - N_c}{2} = \frac{1984{,}8 - 1062{,}5}{2} = 461{,}15\,kN$$

$$N_{a1} = A_{a1} \cdot \frac{f_y}{\gamma_{M0}} \Rightarrow A_{a1} = \frac{N_{a1} \cdot \gamma_{M0}}{f_y} = \frac{461{,}15 \cdot 1{,}0}{235} = 1962{,}4\,mm^2$$

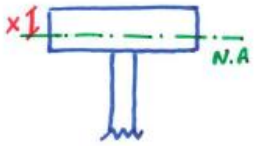
ASSUMPTION: N.A IS IN THE WEB.



$$A_{a1} = 1962{,}4\,mm^2 = (180 \times 13{,}5) + (8.6 \cdot (x - 13{,}5))$$

$$x = -40.87 \otimes$$

N.A IN THE FLANGE



$$A_{a1} = 1962.4\,mm^2 = (180 \cdot x) \Rightarrow \underline{x = 10{,}9\,mm}$$

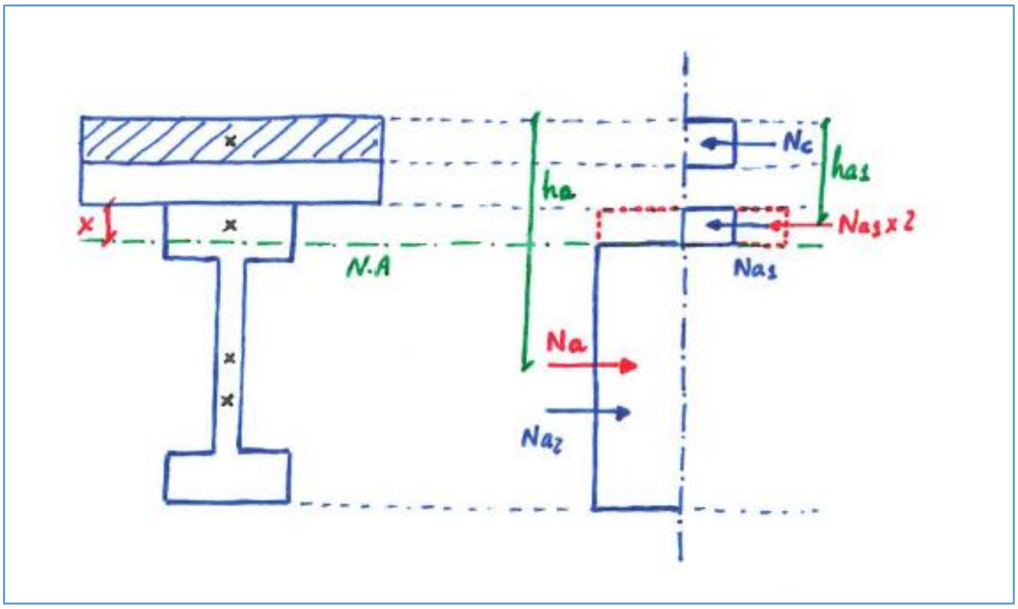


Fig. 5.14. Disposition of the forces when the neutral axis is in the IPE profile.

$$h_{a1} = (h_c + h_{TR}) + \frac{x}{2} = 100 + \frac{10{,}9}{2} = 105{,}45\,mm$$

$$h_a = (h_c + h_{TR}) + \frac{h_{IPE}}{2} = 100 + \frac{400}{2} = 300\,mm$$

$$M_{pl,Rd} = N_a \cdot \left(h_a - \frac{t_c}{2}\right) - 2 \cdot N_{a1} \cdot \left(h_{a1} - \frac{t_c}{2}\right) = 1984{,}8 \cdot 10^3 \cdot \left(300 - \frac{50}{2}\right) - 2 \cdot 461{,}15 \cdot 10^3 \left(105{,}4 - \frac{50}{2}\right)$$

$$M_{pl,Rd} = 471{,}6\,kN > M_{sd} = 437{,}5\,kN \quad \varnothing$$

c) SHEAR CONNECTION

THE SAME STUDS WITH THE SAME GEOMETRY AND QUALITY OF THE STEEL ARE GOING TO BE USED. THEREFORE, SOME PARAMETERS WILL NOT CHANGE FROM THE CALCULATION IT WAS DONE IN THE CROSS BEAMS.

$h_{sc} = 90\,mm$; $P_{Rk} = 81,1\,kN$; $P_{Rd} = 64,88\,kN$

REDUCTION DUE TO SHEETING WAVES:

$$k_l = 0,6 \cdot \frac{b_0}{h_p} \cdot \frac{h_{sc} - h_p}{h_p} = 0,6 \cdot \frac{84,5}{50} \cdot \frac{90-50}{50} = 0,81 \longrightarrow \underline{P_{Rd,r}} = k_l \cdot P_{Rd} = \underline{52,63\,kN}$$

$$F_{cf} = \begin{cases} F_{cf,a} = \dfrac{A_a \cdot f_y}{\gamma_a} = \dfrac{8450\,mm^2 \cdot 235\,N/mm^2}{1,0} = 1984,8\,kN \\ \underline{F_{cf,c}} = \dfrac{A_c \cdot 0.85 \cdot f_{ck}}{\gamma_c} + \dfrac{A_s \cdot f_{sk}}{\gamma_s} = \dfrac{50 \cdot 1500 \cdot 0,85 \cdot 20}{1,5} + \dfrac{1656,7 \cdot 410}{1,0} = 850 + 679,2 = \underline{1530\,kN} \end{cases}$$

THE SECOND PART OF THE ECUATION REFERS TO THE CONCRETE REINFORCEMENT, WHICH PARAMETERS ARE GOING TO BE SUPPOSED AS: Ø15mm ; Separation: 200 mm ; $f_{sk} = 410\,N/mm^2$

$$\underline{A_s} = \frac{\pi \cdot d^2}{4} \cdot \frac{b_{eff}}{separat.} = \frac{\pi \cdot 15^2}{4} \cdot \frac{1875}{200} = \underline{1656,7\,mm^2}$$

NOTE: WE CAN ASSUME AS TRUE THE SUPPOSITION OF THE CONCRETE REINFORCEMENT BECAUSE IT IS 80% OF THE VALUE OF THE CONCRETE SLAB.

$$N_f = \frac{F_{cf,c}}{P_{Rd,r}} = \frac{1530}{52.63} = 29,07 \longrightarrow$$ 30 studs in each half of the beam.

$$\frac{L/2}{N_f} = \frac{7500/2}{30} = 125\,mm > 5 \cdot d = 5 \cdot 19 = 95\,mm$$ ✓ IT IS POSSIBLE TO DO A TOTAL CONNECTION.

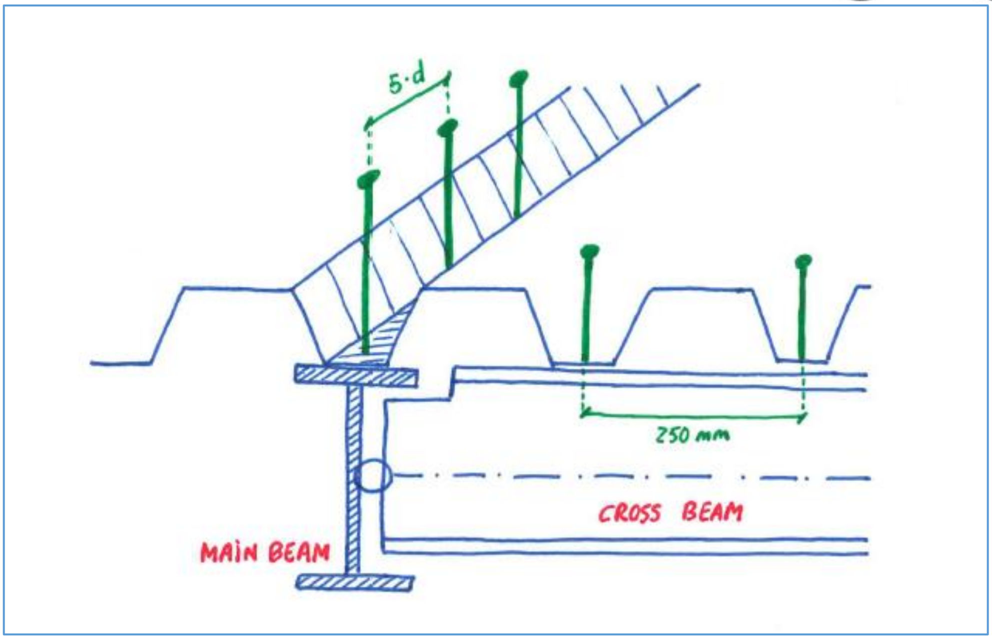


*Fig. 5.15. This draw is useful to see the disposition of the shear studs in the Cross Beam and in Main Beam. It can be appreciated also why, while in the cross beams the separation between the studs had to be 250mm or some multiple number like 500, 750..., and in the main beam that is not necessary because the whole beam matches with the wave of the trapezoidal sheeting. That is the reason why the only condition that it must comply is that the separation is bigger than 5*d.*

STRESS

$$M_k = \frac{1}{3} \cdot F \cdot L = \frac{1}{3} \cdot 123 \cdot 7{,}5 = 307{,}5 \text{ kN}\cdot\text{m}$$

$$h = h_{IPE} + h_c + h_{TR} = 400 + 50 + 50 = 500\text{mm}$$

$$\sigma_{k,a} = \frac{M_k \cdot (h - z_a)}{I_i} = \frac{307{,}5 \cdot 10^6 \cdot (500 - 121{,}6)}{515{,}16 \cdot 10^6} = 225{,}8 \text{ MPa} \leq f_y = 235 \text{ MPa} \checkmark$$

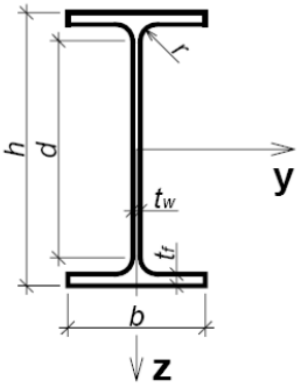
$$\sigma_{k,c} = \frac{1}{n} \cdot \frac{M_k \cdot z_a}{I_i} = \frac{1}{14} \cdot \frac{307{,}5 \cdot 10^6 \cdot 121{,}6}{515{,}16 \cdot 10^6} = 5{,}2 \text{ MPa} \leq f_c = \frac{0{,}85 \cdot f_{ck}}{\gamma_c} = \frac{0{,}85 \cdot 20}{1{,}5} = 11{,}3 \text{ MPa} \checkmark$$

DEFORMATION

$$\delta = \frac{1}{E_a \cdot I_i}(0{,}0355 \cdot F \cdot L^3) = \frac{1}{210 \cdot 10^3 \cdot 515{,}16 \cdot 10^6}(0{,}0355 \cdot 123 \cdot 10^3 \cdot 7500^3) = 17\text{mm}$$

$$\delta = 17\text{mm} \leq \frac{L}{250} = \frac{7500}{250} = 30\text{mm} \checkmark$$

5.3.2. PROFILE SELECTED



	G	h	b	t_w	t_f	r	d	A	$A_{v,z}$	I_y	W_y	$W_{pl,y}$	i_y
	kg/m	mm	mm	mm	mm	mm	mm	mm^2	mm^2	10^4 mm^4	10^3 mm^3	10^3 mm^3	mm
IPE 180	18, 8	180	91	5, 3	8, 0	9	146, 0	2395	1125	1317	146, 3	166, 4	74, 2
IPE 200	22, 4	200	100	5, 6	8, 5	12	159, 0	2848	1400	1943	194, 3	220, 6	82, 6
IPE 220	26, 2	220	110	5, 9	9, 2	12	177, 6	3337	1588	2772	252, 0	285, 4	91, 1
IPE 240	30, 7	240	120	6, 2	9, 8	15	190, 4	3912	1914	3892	324, 3	366, 6	99, 7
IPE 270	36, 1	270	135	6, 6	10, 2	15	219, 6	4595	2214	5790	428, 9	484, 0	112
IPE 300	42, 2	300	150	7, 1	10, 7	15	248, 6	5381	2568	8356	557, 1	628, 4	125
IPE 330	49, 1	330	160	7, 5	11, 5	18	271, 0	6261	3081	11770	713, 1	804, 3	137
IPE 360	57, 1	360	170	8, 0	12, 7	18	298, 6	7273	3514	16270	903, 6	1019	150
IPE 400	66, 3	400	180	8, 6	13, 5	21	331, 0	8446	4269	23130	1156	1307	165
IPE 450	77, 6	450	190	9, 4	14, 6	21	378, 8	9882	5085	33740	1500	1702	185

I_z	W_z	$W_{pl,z}$	i_z	I_t	I_w	bending				compression				
10^4 mm^4	10^3 mm^3	10^3 mm^3	mm	10^4 mm^4	10^6 mm^6	S235	S275	S355	S460	S235	S275	S355	S460	
100, 9	22, 16	34, 60	20, 5	4, 790	7431	1	1	1	1	1	1	2	3	IPE 180
142, 4	28, 47	44, 61	22, 4	6, 980	12990	1	1	1	1	1	1	2	3	IPE 200
204, 9	37, 25	58, 11	24, 8	9, 066	22670	1	1	1	1	1	1	2	4	IPE 220
283, 6	47, 27	73, 92	26, 9	12, 88	37390	1	1	1	1	1	2	2	4	IPE 240
419, 9	62, 20	96, 95	30, 2	15, 94	70580	1	1	1	1	2	2	3	4	IPE 270
603, 8	80, 50	125, 2	33, 5	20, 12	125900	1	1	1	1	2	2	4	4	IPE 300
788, 1	98, 52	153, 7	35, 5	28, 15	199100	1	1	1	1	2	3	4	4	IPE 330
1043	122, 8	191, 1	37, 9	37, 32	313600	1	1	1	1	2	3	4	4	IPE 360
1318	146, 4	229, 0	39, 5	51, 08	490000	1	1	1	1	3	3	4	4	IPE 400
1676	176, 4	276, 4	41, 2	66, 87	791000	1	1	1	1	3	4	4	4	IPE 450

5.4. COLUMNS

As it is shown in the drawing which is below, it is going to be realised the dimensioning of one of the columns which are inside the structure of the building. This means that it is loaded symmetrically by two cross beams and two main beams.

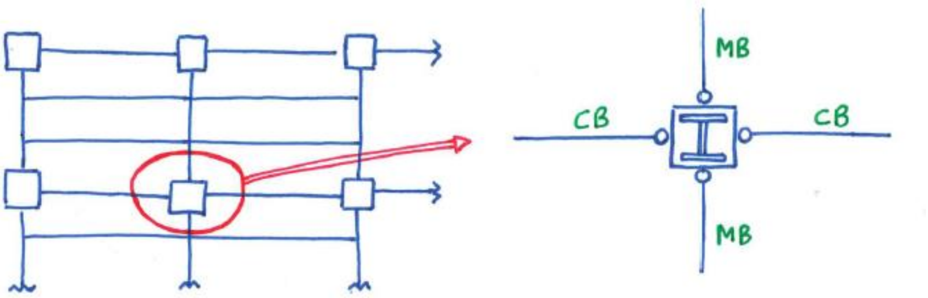


Fig. 5.16. Detail of the loads that the columns carry with.

The main effect to be considered in the dimensioning of a column is the buckling is the buckling length of the column. In our case, this distance is 5 meters, as shown below, since in each plant, the columns are supported by supports.

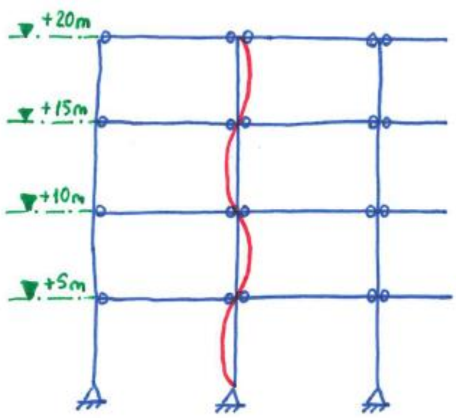


Fig. 5.17. Deformation and bucking lengths in the calculation of the column profile.

There is a coefficient that is responsible for reducing the effect of loading on the column of the first floor. This coefficient depends on the number of floors, being in this case 4.

$$\alpha_n = \frac{2+(n-2)\cdot\Psi_0}{n} = \frac{2+(4-2)\cdot 0{,}7}{4} = 0{,}85$$

Now the total compressive load acting on the column can already be calculated.

$$N_{sd}(1\,Floor) = 2\cdot V_{sd}(CB) + 2\cdot V_{sd}(MB) = 2\cdot 87{,}75\,kN + 2\cdot 175\,kN = 525{,}5\,kN.$$

$$N_{sd} = n\cdot\alpha_n\cdot N_{sd}(1\,Floor) = 4\cdot 0{,}85\cdot 525{,}5\,kN = 1786{,}7\,kN$$

The critical length of buckling must now be calculated. It depends on the β coefficient, and this in turn depends on the way in which said column is supported.

In this case, it is a bi-supported column, therefore β is equal to 1 and the critical length coincides with the distance between supports.

$L_{cr} = \beta \cdot L$

a) $\beta = 1{,}0$

$L_{cr} = L = 5000mm$

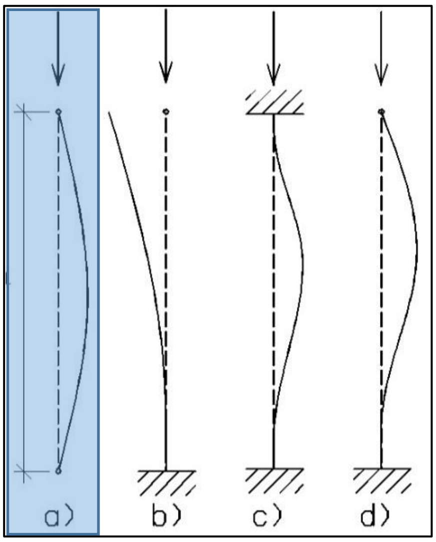


Fig. 5.18. Support situations.

AN HEB-260 IS ASSUMED.

• CLASSIFICATION OF THE SECTION: {WEB & FLANGE WORKING IN COMPRESSION}

$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{235}} = 1{,}0$

WEB: $\frac{d}{t_w} = \frac{177}{10} = 17{,}7 < 33 \cdot \varepsilon = 33$ ✓

FLANGE: $\frac{c}{t_f} = \frac{b/2}{t_f} = \frac{260/2}{17{,}5} = 7{,}43 < 10 \cdot \varepsilon = 10$ ✓

CLASS 1 $\Rightarrow$ $\beta_A = 1{,}0$

$\lambda_y = \frac{l_{cr,y}}{i_y} = \frac{5000\,mm}{112{,}2\,mm} = 44{,}56$

$\lambda_z = \frac{l_{cr,z}}{i_z} = \frac{5000\,mm}{65{,}8\,mm} = 75{,}98$ (GREATER SLENDERNESS) → CHECK THIS PLANE ONLY.

$\lambda_1 = 93{,}9 \cdot \varepsilon = 93{,}9$

$\bar{\lambda} = \frac{\lambda_z}{\lambda_1} = \frac{75{,}98}{93{,}9} = 0{,}809$

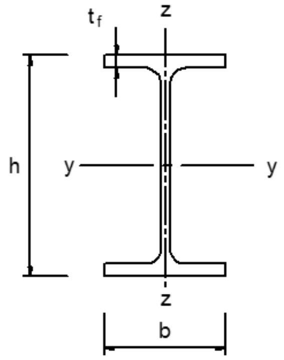
cross-section		limits		buckling about axis	buckling curve	
					S 235 S 275 S 355 S 420	S 460
rolled sections		$h/b > 1{,}2$	$t_f \leq 40$ mm	y – y z – z	a b	a_0 a_0
			40 mm < $t_f \leq 100$	y – y z – z	b c	a a
		$h/b \leq 1{,}2$	$t_f \leq 100$ mm	y – y z – z	b c	a a
			$t_f > 100$ mm	y – y z – z	d d	c c

Table 5.4. Bending curve corresponding to the HEB 260 profile.

1ºOPTION: *With ecuations.*

buckling curve	a_0	a	b	c	d
the imperfection factor α	0,13	0,21	0,34	0,49	0,76

Table 5.5. Coefficient α corresponding to curve C.

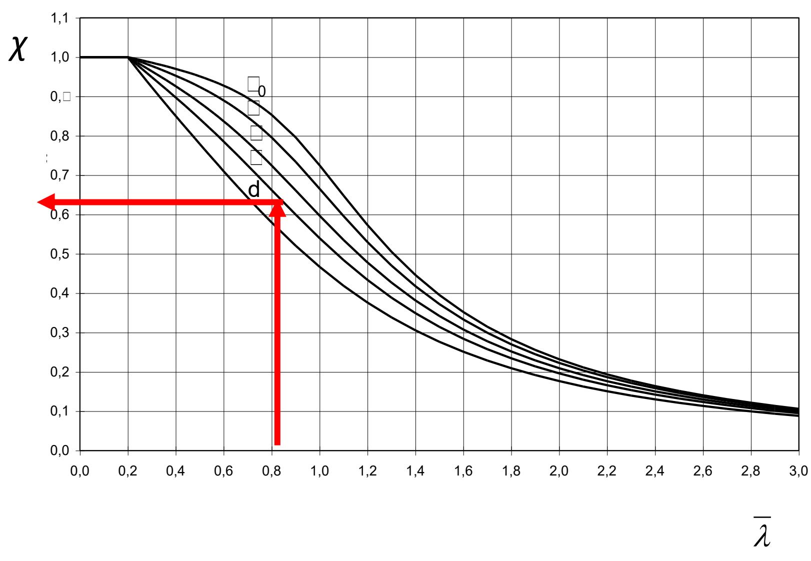
$$\phi = \frac{1}{2}\left[1+\alpha\left(\bar{\lambda}-0{,}2\right)+\bar{\lambda}^2\right]$$

$$\phi = \frac{1}{2}\cdot\left[1+0{,}49\cdot(0{,}809-0{,}2)+0{,}809^2\right]=0{,}976$$

$$\chi = \frac{1}{\phi+\sqrt{\phi^2-\bar{\lambda}^2}} \quad but \quad \chi \leq 1{,}0$$

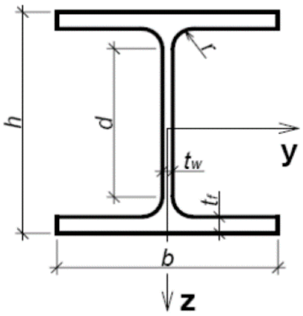
$$\chi = \frac{1}{0{,}976+\sqrt{0{,}976^2-0{,}809^2}} = 0{,}656$$

2ºOPTION: *With a graph.*



$$N_{b,Rd} = \beta_A \cdot \chi \cdot \frac{A \cdot f_y}{\gamma_{M1}} = 1{,}0 \cdot 0{,}656 \cdot \frac{11850 \cdot 235}{1{,}0} = 1827{,}3\ kN > N_{sd} = 1786{,}7\ kN$$

5.4.1. PROFILE SELECTED [*]



	G	h	b	t_w	t_f	r	d	A	$A_{v,z}$	I_y	W_y	$W_{pl,y}$	i_y
	kg/m	mm	mm	mm	mm	mm	mm	mm^2	mm^2	10^4 mm^4	10^3 mm^3	10^3 mm^3	mm
HE 100 B	20, 4	100	100	6, 0	10, 0	12	56	2604	904	449, 5	89, 91	104, 2	41, 6
HE 120 B	26, 7	120	120	6, 5	11, 0	12	74	3401	1096	864, 4	144, 1	165, 2	50, 4
HE 140 B	33, 7	140	140	7, 0	12, 0	12	92	4296	1308	1509	215, 6	245, 4	59, 3
HE 160 B	42, 6	160	160	8, 0	13, 0	15	104	5425	1759	2492	311, 5	354, 0	67, 8
HE 180 B	51, 2	180	180	8, 5	14, 0	15	122	6525	2024	3831	425, 7	481, 4	76, 6
HE 200 B	61, 3	200	200	9, 0	15, 0	18	134	7808	2483	5696	569, 6	642, 5	85, 4
HE 220 B	71, 5	220	220	9, 5	16, 0	18	152	9104	2792	8091	735, 5	827, 0	94, 3
HE 240 B	83, 2	240	240	10, 0	17, 0	21	164	10600	3323	11260	938, 3	1053	103
HE 260 B	93, 0	260	260	10, 0	17, 5	24	177	11840	3759	14920	1148	1283	112
HE 280 B	103, 1	280	280	10, 5	18, 0	24	196	13140	4109	19270	1376	1534	121

I_z	W_z	$W_{pl,z}$	i_z	I_t	I_w	bending				compression				
10^4 mm^4	10^3 mm^3	10^3 mm^3	mm	10^4 mm^4	10^6 mm^6	S235	S275	S355	S460	S235	S275	S355	S460	
167, 3	33, 45	51, 42	25, 3	9, 248	3375	1	1	1	1	1	1	1	1	HE 100 B
317, 5	52, 92	80, 97	30, 6	13, 84	9410	1	1	1	1	1	1	1	1	HE 120 B
549, 7	78, 52	119, 8	35, 8	20, 06	22480	1	1	1	1	1	1	1	1	HE 140 B
889, 2	111, 2	170, 0	40, 5	31, 24	47940	1	1	1	1	1	1	1	1	HE 160 B
1363	151, 4	231, 0	45, 7	42, 16	93750	1	1	1	1	1	1	1	1	HE 180 B
2003	200, 3	305, 8	50, 7	59, 28	171100	1	1	1	1	1	1	1	1	HE 200 B
2843	258, 5	393, 9	55, 9	76, 57	295400	1	1	1	1	1	1	1	1	HE 220 B
3923	326, 9	498, 4	60, 8	102, 7	486900	1	1	1	1	1	1	1	1	HE 240 B
5135	395, 0	602, 2	65, 8	123, 8	753700	1	1	1	2	1	1	1	2	HE 260 B
6595	471, 0	717, 6	70, 9	143, 7	1130000	1	1	1	2	1	1	1	2	HE 280 B

5.5. CONNECTION CROSS BEAM & MAIN BEAM

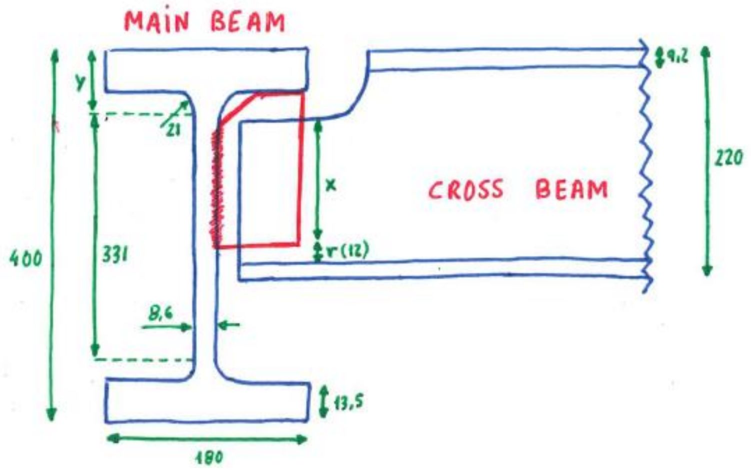


Fig. 5.19. Main beam and cross beam connection.

$$400 = 331 + 2 \cdot Y \longrightarrow \underline{Y} = \frac{400-331}{2} = \underline{34{,}5\,mm}$$

$$\underline{x} = 220 - Y - 9{,}2 - 12 = 220 - 34{,}5 - 9{,}2 - 12 = \underline{164{,}3\,mm}$$ (SPACE FOR THE BOLTS).

BOLTS: $\boxed{M18\ (\varnothing 19)\ ;\ QUALITY\ 8.8}$

Nominal bolt diameter *d* (mm)	12	16	18	20	24	27	30
Tensile stress area of the bolt A_s (mm^2)	84,3	157	192	245	353	459	561

Table 5.6. Tensile stress area associated with each bolt diameter.

BOLT GRADE	f_{yb} (MPa)	f_{ub} (MPa)	
	Yield strength of the bolt	Ultimate tensile strength of the bolt	
4.6	240	400	
4.8	320	400	
5.6	300	500	**NORMAL BOLTS**
5.8	400	500	
6.8	640	600	
8.8	640	800	**HIGH-STRENGTH BOLTS**
10.9	900	1000	

Table 5.7. Yield strength and ultimate tensile strength of the bolt for each grade of bolt.

$1{,}2 \cdot d_0 < e_1 < \min(12.t,\ 150\ mm)$,

$1{,}2 \cdot d_0 < e_2 < \min(12.t,\ 150\ mm)$,

$2{,}2 \cdot d_0 < p_1 < \min(14.t,\ 200\ mm)$,

$3{,}0 \cdot d_0 < p_1 < \min(14.t,\ 200\ mm)$,

where d_0 is the diameter of the hole in the bolt connection,
t is the minimal thickness of the connected plates.

bolts	recommended bolt distances (mm)		
	p_1, p_2	e_1, e_2	e_{min}
M12	40	30	25
M16	55	40	30
M20	70	50	40
M24	80	60	50
M26	90	70	55
M27	100	75	60
M30	120	90	70

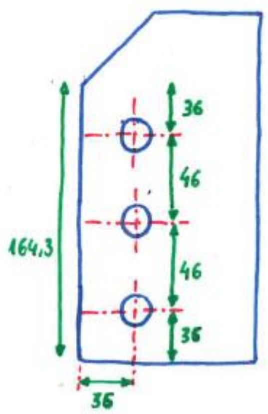
Table 5.8. Recommended bolt distances and intervals where these distances must be in.

$e_1/e_2: \ 1{,}2 \cdot d_0 = 1{,}2 \cdot 19 = 22{,}8$ // $12 \cdot t = 12 \cdot 5{,}9 = 70{,}8$

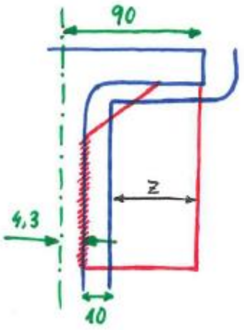
$p_1: \ 2{,}2 \cdot d_0 = 2{,}2 \cdot 19 = 41{,}8$ // $14 \cdot t = 14 \cdot 5{,}9 = 82{,}6$

$e_1, e_2: \ [\underset{e_{min}}{30\text{-}40}\ ;\ 70{,}8]$ // $p_1: \ [40\text{-}50\ ;\ 82{,}6]$

$\Longrightarrow \ \underline{e_1 = e_2 = 36mm}$ // $\underline{p_1 = 46mm} \ \Longleftarrow$



$2 \cdot e_1 + 2 \cdot p_1 = 2 \cdot 36 + 2 \cdot 46 = 164 \leq 164{,}3$ ✓



$90 = 4{,}3 + 10 + z$; $z = 75{,}7\,mm$

CROSS BEAMS $\longrightarrow$ $\{ V_{Ed} = 87{,}75\ kN \}$

$e = e_2 + 10mm = 36 + 10 = 46\ mm$

$F_h = V_{Ed} \cdot \frac{e}{r} = 87{,}75 \cdot \frac{46}{(2 \cdot 46)} = 43{,}9\ kN$

$F_v = \frac{V_{Ed}}{n} = \frac{87{,}75}{3} = 29{,}25\ kN$

$F_{v,Ed} = \sqrt{F_v^2 + F_h^2} = \sqrt{43{,}9^2 + 29{,}25^2} = 52{,}75\ kN$

SHEAR RESISTANCE:

$$F_{v,Rd} = \frac{\alpha_v \cdot f_{ub} \cdot A}{\gamma_{M2}}$$

- bolt grades 4.6, 5.6, 8.8: $\alpha_v = 0{,}6$
- bold grades 4.8, 5.8, 6.8 a 10.9: $\alpha_v = 0{,}5$

BOLT GRADE: 8.8 $\longrightarrow$ $\alpha_v = 0{,}6$; $f_{ub} = 800\ MPa$ // $\gamma_{M2} = 1{,}25$

M18 $\longrightarrow$ $A = A_s = 192\ mm^2$

$F_{v,Rd} = \frac{0{,}6 \cdot 800 \cdot 192}{1{,}25} = 73{,}7\ kN > F_{v,Ed} = 52{,}75\ kN$ ✓

BEARING RESISTANCE:

$$F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{M2}}$$

where α_b is the smallest of the values (in the direction of load transfer):

$\frac{f_{ub}}{f_u}$; 1,0; $\frac{e_1}{3d_0}$ (for end bolds); $\frac{p_1}{3d_0} - \frac{1}{4}$ (for inner bolds),

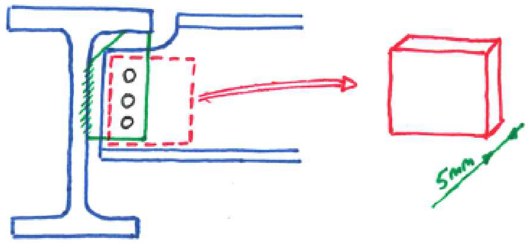
$k_1 = 2{,}5$; $f_u = 360$ MPa ; $t = t_w(\text{IPE-220}) = 5{,}9$ mm ; $d = 18$ mm

αb:

$\frac{f_{ub}}{f_u} = \frac{800}{360} = 2{,}2$; $1{,}0$; $\frac{e_1}{3 \cdot d_0} = \frac{36}{3 \cdot 19} = 0{,}63$; $\frac{p_1}{3 \cdot d_0} - 0{,}25 = \frac{46}{3 \cdot 19} - 0{,}25 = \underline{\underline{0{,}55}}$

$F_{b,Rd} = \frac{2{,}5 \cdot 0{,}55 \cdot 360 \cdot 18 \cdot 5{,}9}{1{,}25} = 42{,}05$ kN $>$ $F_{v,Ed} = 52{,}75$ kN ✗

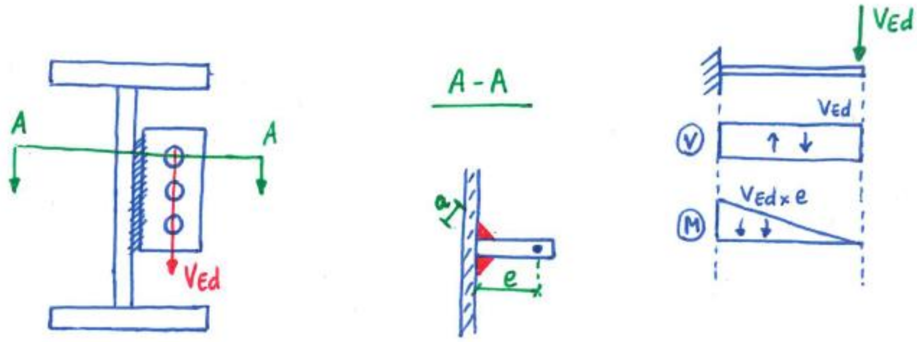
SOLUTION: Weld a sheet of 5mm thick in the web of the cross beam to achieve better transmission of stress.



$t = 5{,}9 + 5 = 10{,}9$ mm

$F_{b,Rd} = \frac{2{,}5 \cdot 0{,}55 \cdot 360 \cdot 18 \cdot 10{,}9}{1{,}25} = 77{,}7$ kN $>$ $F_{v,Ed} = 52{,}75$ kN ✓

WELD CONNECTION RESISTANCE:



PURE SHEAR STRESS: τ_{II}

$$\tau_{II} = \frac{V_{Ed}}{2 \cdot a_w \cdot l_w} \left\{ a_w = 5mm \; ; \; l_w = 164mm \right\} = \frac{87{,}75 \cdot 10^3 N}{2 \cdot 5mm \cdot 164mm} = 53{,}5 MPa$$

BENDING STRESSES: $\tau_{\perp}, \sigma_{\perp}$

$$\left. \begin{array}{l} \left. \begin{array}{l} \sigma_w = \sqrt{\sigma_{\perp}^2 + \tau_{\perp}^2} \\ \sigma_{\perp} = \tau_{\perp} \end{array} \right\} \sigma_{\perp} = \tau_{\perp} = \frac{\sigma_w}{\sqrt{2}} \\ \sigma_w = \frac{M_{Ed}}{W_w} = \frac{M_{Ed}}{\left(\frac{1}{6} \cdot a_w \cdot l_w^2\right) \cdot 2} \end{array} \right\} \tau_{\perp} = \sigma_{\perp} = \frac{V_{Ed} \cdot e}{\frac{2}{6} \cdot a \cdot l_w^2 \cdot \sqrt{2}} = \frac{87{,}75 \cdot 10^3 \cdot 45}{\frac{\sqrt{2}}{3} \cdot 5 \cdot 164^2} = 62{,}3 MPa$$

- $\sqrt{\sigma_{\perp}^2 + 3 \cdot \left(\tau_{\perp}^2 + \tau_{II}^2\right)} = \sqrt{62{,}3^2 + 3 \cdot \left(62{,}3^2 + 53{,}5^2\right)} = 155{,}26 MPa$

  $$155{,}26 MPa < \frac{f_u}{\beta_w \cdot \gamma_{M2}} = \frac{360}{0{,}8 \cdot 1{,}25} = 360 MPa \checkmark$$

- $\sigma_{\perp} = 62{,}3 MPa < \frac{f_u}{\gamma_{M2}} = \frac{360}{1{,}25} = 288 MPa \checkmark$

5.6. CONNECTION COLUMN & FOUNDATION

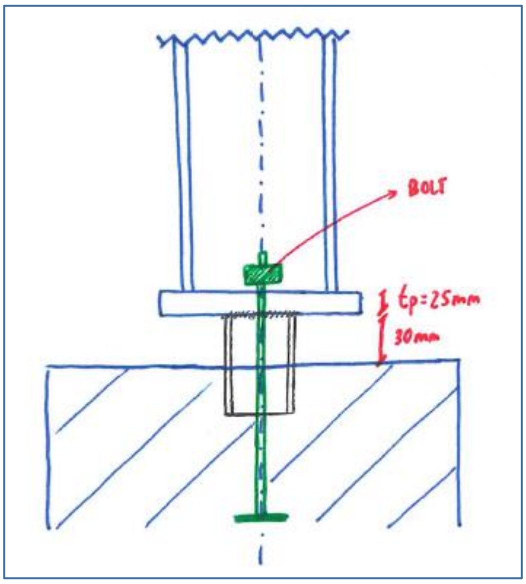


Fig. 5.20. Column and foundation connection.

HINGED FEET

The hinged feet are predominantly stressed by the centrifugally acting N_{sd} pressure force.

It is assumed that the pressure force transferred to the base is evenly distributed over the effective surface of the A_{eff} base plate. The stress under the foot should not be greater than the design strength of the f_{jd} concrete in the joint.

$$f_{jd} = \beta_j \cdot k_j \cdot f_{cd}$$

f_{cd}: Design strength of concrete in compression.

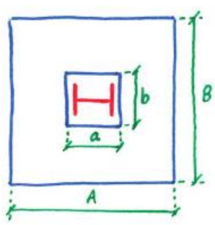
β_j: The influence factor of the casting can be taken as $\beta = 2/3$ if the mortar strength f_{md} ≥ 0,2·f_{cd}

k_j: The concentration factor k_j ≥ 1,0 - expresses the influence of higher load bearing capacity in concentric pressure.

Countable foot dimensions:

$$a_1 = \min(A_{feet}; 5 \cdot a; a + h_{feet}; 5 \cdot b)$$

$$b_1 = \min(B_{feet}; 5 \cdot a; b + h_{feet}; 5 \cdot b)$$



$k_j = \sqrt{\frac{a_1 \cdot b_1}{a \cdot b}}$ Assuming $\left\{ \begin{array}{l} a_1 = 5 \cdot a \\ b_1 = 5 \cdot b \end{array} \right\}$ and designing $a = b \Rightarrow k_j = \sqrt{\frac{5a \cdot 5a}{a \cdot a}} = \sqrt{25} = 5$

The reinforced foot effectively reduces the thickness of the base plate and hence the weight of the entire foot, but it is more labour-intensive and therefore has a more rigid foot (the foot is formed by a foot sheet and transverse or longitudinal reinforcements). The system of reinforcement must be as simple as possible.

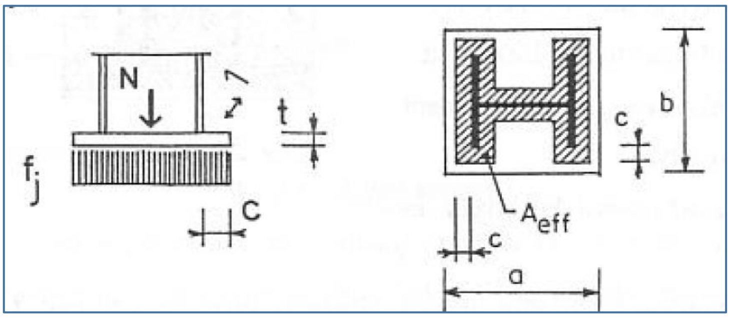
Designing should also be considered for reinforcement and welding.

For technological reasons, the thickness of the sheet metal is limited to 60 mm. This connection is designed with a thickness of: $t_p = 25mm$.

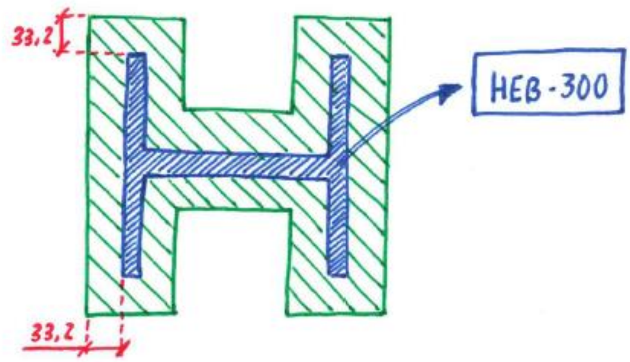
$\beta_j = 2/3$ & $f_{cd} = \frac{20}{1,5} N/mm^2 = 13,3 MPa$

$f_{jd} = 2/3 \cdot 5 \cdot 13,3 = 44,4 MPa$

To calculate the effective area, it is necessary to know first the value of c, dimension that is represented in the following images:



$c = t_p \cdot \sqrt{\frac{f_y}{3 \cdot f_{jd} \cdot \gamma_{M0}}}$ $c = 25mm \cdot \sqrt{\frac{235 N/mm^2}{3 \cdot 44,4 N/mm^2 \cdot 1,0}} = 33,20 mm$



$A_{EFF} = 63400 mm^2$

The step sheet design process is usually iterative:

• The floor plan is selected a & b (approximately $a = b = N_{sd}/f_{cd}$).

WE RELY ON THE PREVIOUS APROXIMATION AND DECIDE THE VALUES OF a AND b.

$$a^2 = \frac{N_{sd}}{F_{cd}} \; ; \; a = \sqrt{\frac{N_{sd}}{F_{cd}}} = \sqrt{\frac{2673 \cdot 10^3}{20/1,5}} = 447,75\,mm \Rightarrow a = b = 450\,mm$$

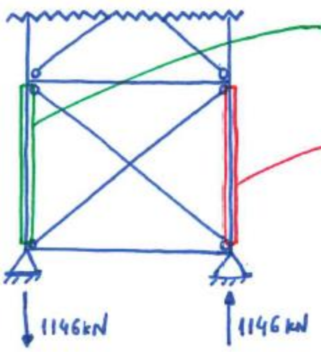
• The design strength of the concrete is determined under the foot (f_{jd}).

• The thickness of the foil plate t_p is chosen, hence the effective length of the bracket c.

• Assess the effective area of the foot ($A_{eff} \geq N_{sd}/f_{jd}$).

• Designed floor plans are corrected and the procedure repeated.

BOLT



-1146 kN + MIN. COMPRESSION FORCE AT THE COLUMN
-1146 kN + 340 kN = -806 kN

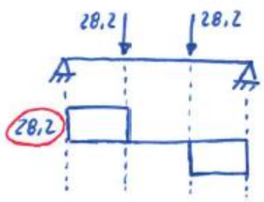
1146 kN + MAX. COMPRESSION FORCE AT THE COLUMN
1146 kN + 1786 kN = 2932 kN

MINIMUN COMPRESSION FORCE:

CROSS BEAM
- IPE 220 : 0,262 kN/m
- TR 50-250 : 0,285 kN/m
- CONCRETE SLAB : 4,15 kN/m

x 3m x 2 beams = 28,2 kN (1 floor).

MAIN BEAM:



28,2 kN x 2 beams = 56,4 kN (1 floor).

N_{min} = (28.2 kN + 56,4 kN) x 4 floors = 340 kN

Tension at the ground: - 806 kN.

Shear at the ground: 405 + 291 + 311 = 1007 kN

Screws with anchor head:

Thread	M36x3	M42x3	M48x3	M56x4	M64x4	M72x4	M80x4	M90x4	M100x4
A_s [mm^2]	865	1206	1604	2144	2851	3658	4566	5842	7276

Table 5.9. Tensile stress area for each group of bolts.

$$F_{t,Rd} = \frac{0{,}8 \cdot A_s \cdot f_y}{\gamma_{M0}}$$

3 M36 BOLTS ARE ASSUMED.

- Supposition: QUALITY 5.6.

$$F_{t,Rd} = \frac{0.8 \cdot 865 \cdot 300}{1{,}0} = 207{,}6\ kN > N_{t,Sd,Max} = \frac{806}{3} = 268{,}6\ kN$$ ✗

- Supposition: QUALITY 6.8.

$$F_{t,Rd} = \frac{0.8 \cdot 865 \cdot 480}{1{,}0} = 332{,}16\ kN > 268{,}6\ kN$$ ✓

5.7. STIFFENING SYSTEM

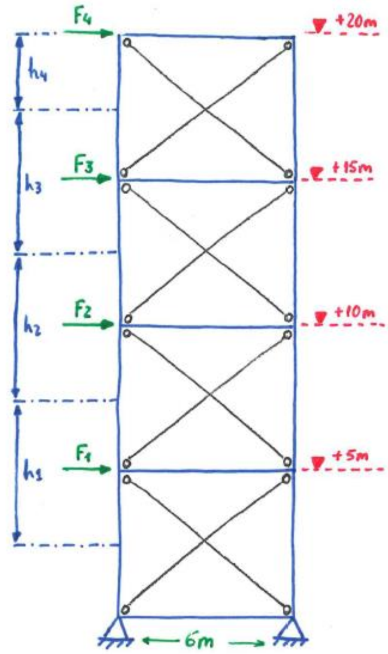


Fig. 5.21. Representation of the stiffening system.

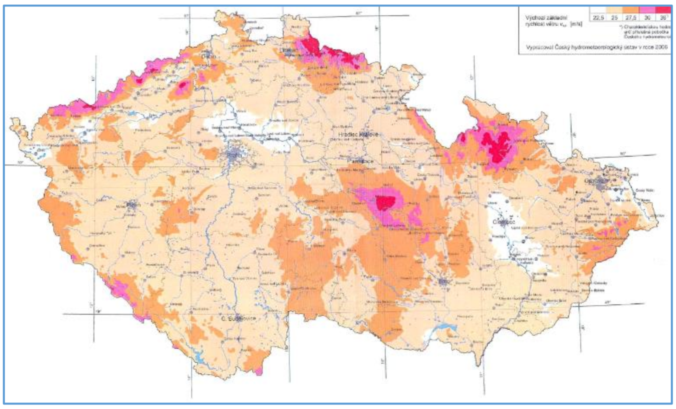
WIND ACTION:

BASIC SPEED OF THE WIND: $\underline{v_b = c_{dir} \cdot c_{season} \cdot v_{b,0}}$

For common situations: $c_{dir} = 1,0$; $c_{season} = 1,0$

$v_{b,0} = 25\,m/s$ (MAP)

$\underline{\underline{v_b}} = 1,0 \cdot 1,0 \cdot 25 = \underline{\underline{25\,m/s}}$



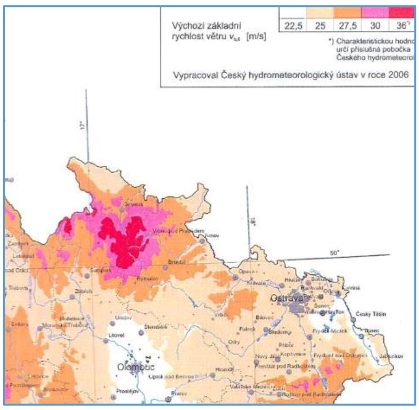


Fig. 5.22. & Fig. 5.23. Basic speed of the wind map in Czech Republic. Ostrava.

$$h = 20m \begin{cases} z_e = h = 20m \\ z_i = h = 20m \end{cases} \quad z = z_e = z_i = h = 20m \geq z_{min} = 5m$$

ROUGHNESS FACTOR : $\underline{C_r(z) = k_r \cdot \ln \frac{z}{z_0}}$

Satellite area → GROUND CATEGORY III → $z_0 = 0,3$

Coefficient of the ground category → $k_r = 0,19 \cdot \left(\frac{z_0}{z_{0,II}}\right)^{0,07} = 0,19 \cdot \left(\frac{0,3}{0,05}\right)^{0,07} = 0,215$

$$C_r(z) = 0,215 \cdot \ln \frac{20}{0,3} = 0,903$$

Coefficient of the athography → $C_o(z) = 1,0$

CHARACTERISTIC MEAN SPEED OF THE WIND :

$$v_m(z) = C_r(z) \cdot C_o(z) \cdot v_b = 0,903 \cdot 1,0 \cdot 25 = 22,57\,m/s$$

MAXIMAL CHARACTERISTIC PRESSURE : $\underline{q_p(z) = [1 + 7 \cdot I_v(z)] \cdot \frac{1}{2} \cdot \rho \cdot v_m^2}$

Intensity of the turbulence → $I_v(z) = \frac{k_l}{C_o(z) \cdot \ln \frac{z}{z_0}} = \frac{1,0}{1,0 \cdot \ln \frac{20}{0,3}} = 0,238$

$$q_p(z) = [1 + 7 \cdot 0,238] \cdot \frac{1}{2} \cdot 1,25 \cdot 22,57^2 = 0,849\,kN/m^2$$

$$\boxed{W_k = q_p(z) \cdot (C_{pe} - C_{pi})}$$

Zone	A		B		C		D		E	
h/d	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,7	
1	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,5	
≤ 0,25	-1,2	-1,4	-0,8	-1,1	-0,5		+0,7	+1,0	-0,3	

Table 5.10. Coefficients of external pressure for each part on the structure.

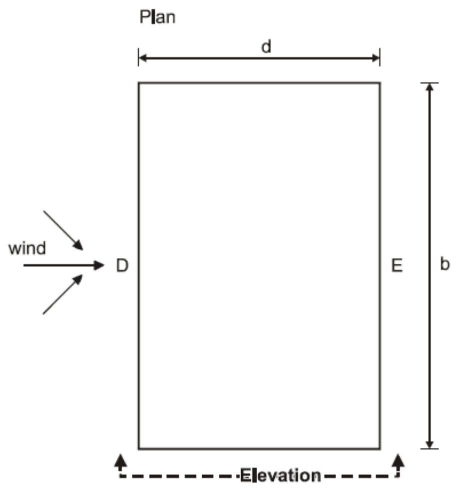
COEFFICIENTS OF WIND PRESSURE:

$$\underline{h/d} = \frac{20}{18} = 1,\hat{1} \simeq \underline{1} \longrightarrow \underline{C_{pe,10}(D) = +0,8} \;//\; \underline{C_{pe,10}(E) = -0,5}$$

$$C_{pi} \begin{cases} +0,2 \text{ (PRESSURE)} \\ -0,3 \text{ (SUCTION).} \end{cases}$$

$$\underline{W_K^D} = 0,849 \cdot (0,8 - (-0,3)) = \underline{0,934 \text{ kN/m}^2}$$

$$\underline{W_K^E} = 0,849 \cdot (-0,5 - (0,2)) = \underline{0,594 \text{ kN/m}^2}$$



$$\boxed{F_i = \frac{(\overrightarrow{W^D} + \overrightarrow{W^E}) \cdot b \cdot h_i}{n}}$$

$$\left.\begin{array}{l} \bullet\ \underline{F_4} = \dfrac{(0,934 + 0,594) \cdot 60 \cdot 2,5}{4} = \underline{57,3 \text{ kN}} \\[2ex] \bullet\ \underline{F_3 = F_2 = F_1} = \dfrac{(0,934 + 0,594) \cdot 60 \cdot 5}{4} = \underline{114,6 \text{ kN}} \end{array}\right\} F_{ki}$$

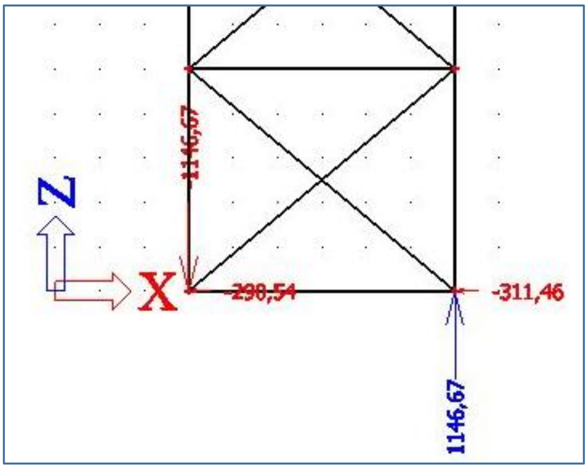
- $\underline{F_4} = 1,5 \cdot 57,3 \text{ kN} = \underline{86 \text{ kN}}$
- $\underline{F_3} = \underline{F_2} = \underline{F_1} = 1,5 \cdot 114,6 \text{ kN} = \underline{172 \text{ kN}}$

The two supports of the stiffening system are two fixed supports.

Each of them will cause 2 reactions; Since the final sum of reactions is 4, it is a hyper-static system.

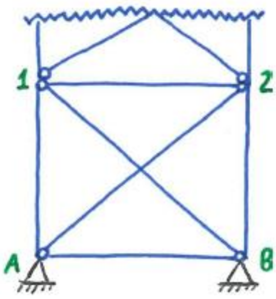
As the resolution at hand is very laborious, I will use a software that helps to calculate the reactions and some other parameter. It will also be useful to check that the calculations made by hand are consistent.

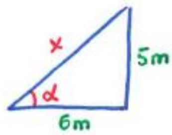
The software used is *SCIA Engineering.*



Once the reactions in the supports are calculated, it is necessary to know which is the load acting in the most loaded columns and diagonal, which are the ones of the first floor.

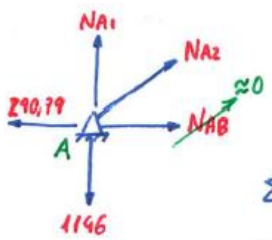
$$\left\{ R_A = -1146\ kN \ ;\ H_A = -290,79\ kN \ ;\ R_B = +1146\ kN \ ;\ H_B = 310,86\ kN \right\}$$





$x = \sqrt{6^2+5^2} = 7,81\ m$

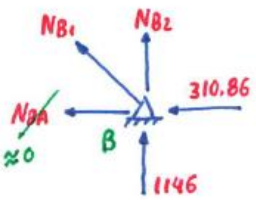
$\alpha = tg^{-1}\left(\frac{5}{6}\right) = 39,80^\circ$



AS THE LOAD TO BE RESISTED BY THE BEAM AB IS ASSUMED TO BE VERY SMALL, AN APPROXIMATION IS MADE SUPPOSING THAT $\underline{N_{AB} = 0}$.

$\Sigma F_H = 0: \ N_{A2} \cdot \cos 39,8^\circ = 290,79 \ ; \ \underline{N_{A2} = 378,49\ kN}$

$\Sigma F_V = 0: \ N_{A1} + N_{A2} \cdot \sin 39,8^\circ = 1146 \ ; \ \underline{N_{A1} = 903,72\ kN}$



$\Sigma F_H = 0: \ 310,86 + N_{B1} \cdot \cos 39,8^\circ = 0 ; \ \underline{N_{B1} = -404,62\ kN}$

$\Sigma F_V = 0: \ 1146 + N_{B2} + N_{B1} \cdot \sin 39,8^\circ = 0 ; \ \underline{N_{B2} = -887,0\ kN}$

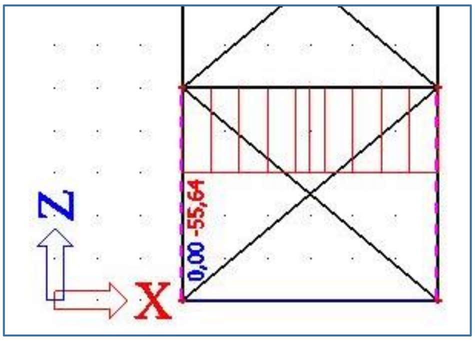


Fig. 5.24. Internal force in horizontal beam.

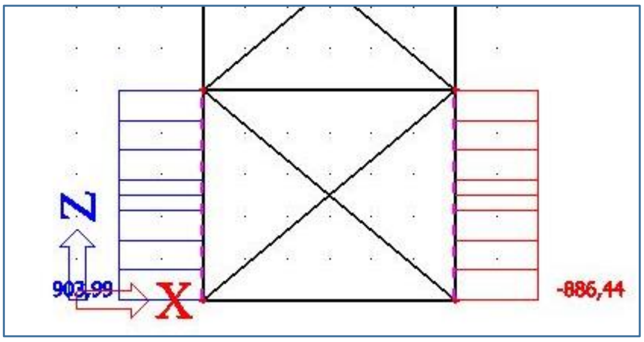


Fig. 5.25. Internal force in column.

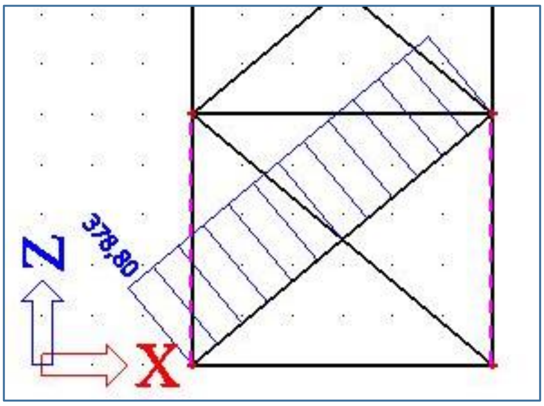


Fig. 5.26. Internal force in tensioned diagonal.

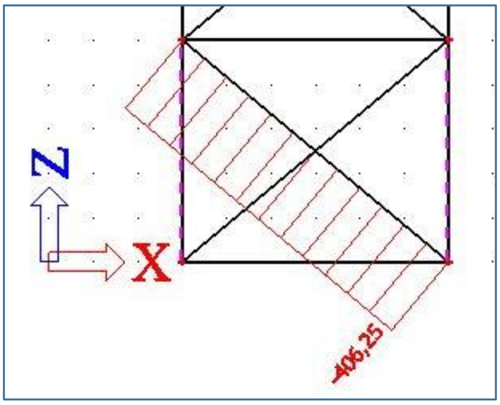


Fig. 5.27. Internal force in compressed diagonal.

If we compare the efforts in the bars of the calculation by hand with the efforts obtained with the software they are practically the same.

Also, thanks to the software, it can be said that the assumption that the first horizontal beam does not work is true.

As it can be seen in the results, the load value of the steel bars that work in tension is very similar to the value of the steel bars that work in compression.

But, it is known that the compressed elements are more restrictive and dangerous because of the buckling effect, so they will be the ones that will give us the dimensions of the necessary profiles.

COLUMS: [*]

Previously, the columns had already been dimensioned with a HEB-260 profile.

However, when considering this new load, it must be checked if that profile will remain valid or we must choose a greater profile.

- BEFORE : $N_{sd} = 1786{,}7\,kN < N_{b,Rd} = 1827{,}3\,kN$ ✓

- NOW : $N_{sd} = 1786{,}7\,kN + 887\,kN = 2673{,}7\,kN > N_{b,Rd} = 1827{,}3\,kN$ ✗

AN HEB-300 IS ASSUMED.

- CLASS OF THE SECTION :

$$\left.\begin{array}{l} \text{WEB}: d/t_w = 208/11 = 18{,}91 < 33\cdot1{,}0 \checkmark \\ \text{FLANGE}: c/t_f = 150/19 = 7{,}89 < 10\cdot1{,}0 \checkmark \end{array}\right\} \text{CLASS 1} \Rightarrow \beta_A = 1{,}0$$

$$\lambda_y = \frac{l_{cr,y}}{i_y} = \frac{5000\,mm}{129{,}9\,mm} = 38{,}49 \qquad \lambda_z = \frac{l_{cr,z}}{i_z} = \frac{5000\,mm}{75{,}8\,mm} = 65{,}96$$

$$\bar{\lambda} = \lambda_z/\lambda_1 = 65{,}96/93{,}9 = 0{,}702 \qquad \text{Curve } C \rightarrow \alpha = 0{,}49$$

$$\phi = 0{,}5\cdot\left[1 + 0{,}49\cdot(0{,}702-0{,}2) + 0{,}702^2\right] = 0{,}869$$

$$\chi = \frac{1}{0{,}869^2 + \sqrt{0{,}869^2 - 0{,}702^2}} = 0{,}787$$

$$N_{b,Rd} = \beta_A\cdot\chi\cdot\frac{A\cdot f_y}{\gamma_{M1}} = 1{,}0\cdot0{,}787\cdot\frac{14910\cdot235}{1{,}0} = 2758{,}5\,kN > N_{sd} = 2673{,}7\,kN$$ ✓

DIAGONALS:

As these elements are just particular from the stiffening system, they have not been dimensioned and calculated yet.

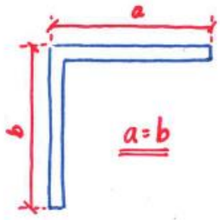
For diagonals, the type of angular profiles is commonly used, since the characteristics they present make them suitable for supporting the stresses to which they are subjected, without being too heavy.

The dimensioning of the angular profile is laborious because this section is not doubly symmetrical.

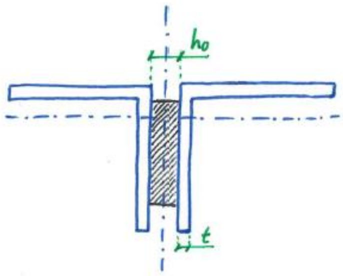
This fact causes that the section can buckle in many ways and slenderness results at many levels: $\lambda_y, \lambda_z, \lambda_w, \lambda_{zw}$.

There is an approximation that facilitates the calculations, but can only be used if the following conditions are met:

• Equal leg angles.

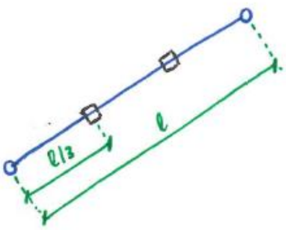


• Thickness of the join plate greater than the thickness of the profile. $(h_0 > t)$



• Critical length of buckling of equal length in both planes. $(l_{CRy} \cong l_{CRz})$

• Connecting members are used with a maximum separation of one third of the length critic.



Once all those conditions are ensured, the slenderness is reduced to one level: λ_y, which is the critical value.

In our structure, because all the conditions are fulfilled, that is the only plane that is going to be checked.

AN L 150x14 IS ASSUMED. $\{2 \cdot L\ 150 \times 14\}$

• CLASS OF THE SECTION:

$$\frac{h}{t} = \frac{150}{14} = 10{,}71 < 11{,}5 \cdot \varepsilon \checkmark \quad \text{AT LEAST CLASS 3} \Rightarrow \beta_A = 1{,}0$$

$$2 \cdot L\,150 \times 14 \left\{ \begin{array}{l} A = 2 \cdot A\,(L\text{-}150 \times 14) \\ I_y = 2 \cdot I_y\,(L\text{-}150 \times 14) \end{array} \right\} \quad i_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{2 \cdot I_y}{2 \cdot A}} = i_y\,(L\text{-}150 \times 14).$$

$$\lambda_y = \frac{l_{cr,y}}{i_y} = \frac{7810}{45{,}8} = 170{,}5 \qquad \bar{\lambda} = \frac{\lambda_y}{\lambda_1} = \frac{170{,}5}{93{,}9} = 1{,}816$$

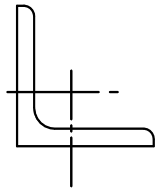
cross-section		limits	buckling about axis	buckling curve	
				S 235 S 275 S 355 S 420	S 460
L-sections			any	**b**	**b**

Table 5.11. Bending curve corresponding to an angular (L) profile.

buckling curve	a_0	a	b	c	d
the imperfection factor α	0,13	0,21	0,34	0,49	0,76

Table 5.12. Coefficient α corresponding to curve B.

Curve B $\rightarrow \alpha = 0{,}34 \qquad \phi = 0{,}5 \cdot \left[1 + 0{,}34 \cdot (1{,}816 - 0{,}2) + 1{,}816^2\right] = 2{,}42$

$$\chi = \frac{1}{2{,}42 + \sqrt{2{,}42^2 - 1{,}816^2}} = 0{,}25$$

$$N_{b,Rd} = \beta_A \cdot \chi \cdot \frac{2 \cdot A \cdot f_y}{\gamma_{M1}} = 1{,}0 \cdot 0{,}25 \cdot \frac{2 \cdot 4030 \cdot 235}{1{,}0} = 470{,}1\,kN > N_{sd} = 404{,}6\,kN \checkmark$$

Once, we know the dimension of each part of the stiffening system, we can calculate again with the software, how much the reactions will change when the self-weight is considered.

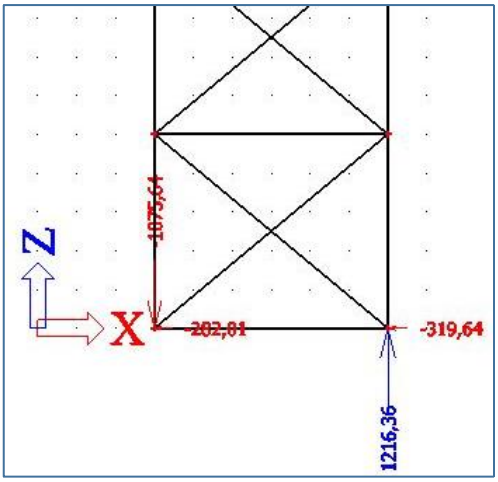


Fig. 5.28. Reactions in the stiffening system considering the self-weight of the structure.

There is no noticeable change in the reaction, so there is no need to recalculate any section of the system.

Using the help of SCIA Engineering, the deformation of the structure in the Service Limit State is calculated.

The result is a maximum deformation of 1.6 mm.

$\delta_{max} = 1{,}6mm < L/250 = 20000mm/250 = 80mm$

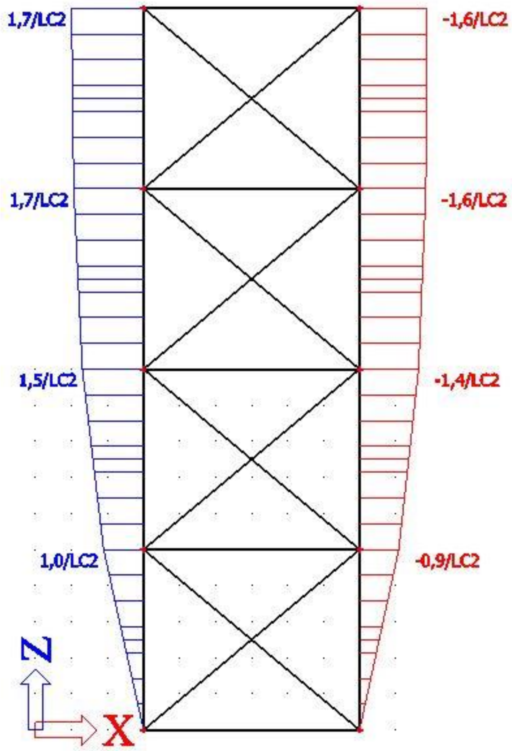
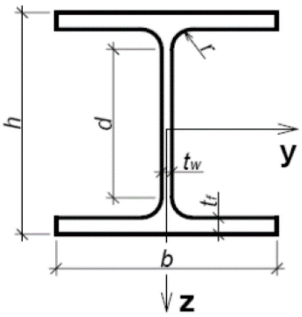


Fig. 5.29. Representation of the total deflection of the structure because of the wind action.

5.7.1. PROFILES SELECTED

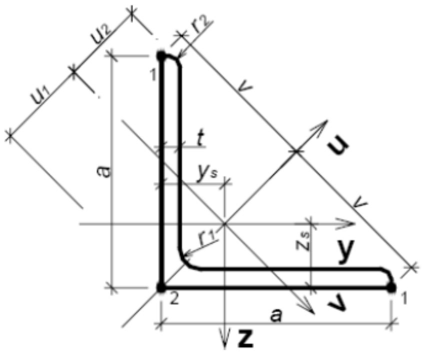
COLUMS:



	G	h	b	t_w	t_f	r	d	A	$A_{v.z}$	I_y	W_y	$W_{pl.y}$	i_y
										10^4	10^3	10^3	
	kg/m	mm	mm	mm	mm	mm	mm	mm^2	mm^2	mm^4	mm^3	mm^3	mm
HE 200 B	61, 3	200	200	9, 0	15, 0	18	134	7808	2483	5696	569, 6	642, 5	85, 4
HE 220 B	71, 5	220	220	9, 5	16, 0	18	152	9104	2792	8091	735, 5	827, 0	94, 3
HE 240 B	83, 2	240	240	10, 0	17, 0	21	164	10600	3323	11260	938, 3	1053	103
HE 260 B	93, 0	260	260	10, 0	17, 5	24	177	11840	3759	14920	1148	1283	112
HE 280 B	103, 1	280	280	10, 5	18, 0	24	196	13140	4109	19270	1376	1534	121
HE 300 B	117, 0	300	300	11, 0	19, 0	27	208	14910	4743	25170	1678	1869	130
HE 320 B	126, 7	320	300	11, 5	20, 5	27	225	16130	5177	30820	1926	2149	138
HE 340 B	134, 2	340	300	12, 0	21, 5	27	243	17090	5609	36660	2156	2408	146
HE 360 B	141, 8	360	300	12, 5	22, 5	27	261	18060	6060	43190	2400	2683	155
HE 400 B	155, 3	400	300	13, 5	24, 0	27	298	19780	6998	57680	2884	3232	171

I_z	W_z	$W_{pl.z}$	i_z	I_t	I_w	bending				compression				
10^4	10^3	10^3		10^4	10^6	S235	S275	S355	S460	S235	S275	S355	S460	
mm^4	mm^3	mm^3	mm	mm^4	mm^6									
2003	200, 3	305, 8	50, 7	59, 28	171100	1	1	1	1	1	1	1	1	HE 200 B
2843	258, 5	393, 9	55, 9	76, 57	295400	1	1	1	1	1	1	1	1	HE 220 B
3923	326, 9	498, 4	60, 8	102, 7	486900	1	1	1	1	1	1	1	1	HE 240 B
5135	395, 0	602, 2	65, 8	123, 8	753700	1	1	1	2	1	1	1	2	HE 260 B
6595	471, 0	717, 6	70, 9	143, 7	1130000	1	1	1	2	1	1	1	2	HE 280 B
8563	570, 9	870, 1	75, 8	185, 0	1688000	1	1	1	3	1	1	1	3	HE 300 B
9239	615, 9	939, 1	75, 7	225, 1	2069000	1	1	1	2	1	1	1	2	HE 320 B
9690	646, 0	985, 7	75, 3	257, 2	2454000	1	1	1	1	1	1	1	1	HE 340 B
10140	676, 1	1032	74, 9	292, 5	2883000	1	1	1	1	1	1	1	1	HE 360 B
10820	721, 3	1104	74, 0	355, 7	3817000	1	1	1	1	1	1	1	1	HE 400 B

DIAGONALS:



	G	a	t	r_1	r_2	y_s	u_1	u_2	v	A	I_y	W_y	i_y
	kg/m	mm	mm	mm	mm	mm	mm	mm	mm	mm^2	10^4 mm^4	10^3 mm^3	mm
L 130 × 12	23, 6	130	12	14, 0	7, 0	36, 4	51, 5	46, 0	91, 9	3000	472	50, 4	39, 7
× 14	27, 2	130	14	14, 0	7, 0	37, 2	52, 6	46, 3	91, 9	3470	540	58, 2	39, 5
L 140 × 10	21, 4	140	10	15, 0	7, 5	37, 9	53, 7	49, 3	99, 0	2720	504	49, 4	43, 0
× 12	25, 4	140	12	15, 0	7, 5	38, 8	54, 9	49, 5	99, 0	3240	595	58, 8	42, 8
× 13	27, 4	140	13	15, 0	7, 5	39, 2	55, 5	49, 6	99, 0	3500	639	63, 4	42, 7
× 14	29, 4	140	14	15, 0	7, 5	39, 6	56, 1	49, 7	99, 0	3750	681	67, 9	42, 6
L 150 × 10	23, 0	150	10	16, 0	8, 0	40, 3	57, 1	52, 8	106, 1	2930	624	56, 9	46, 2
× 12	27, 3	150	12	16, 0	8, 0	41, 2	58, 3	52, 9	106, 1	3480	737	67, 8	46, 0
× 14	31, 6	150	14	16, 0	8, 0	42, 1	59, 5	53, 2	106, 1	4030	845	78, 3	45, 8
× 15	33, 8	150	15	16, 0	8, 0	42, 5	60, 1	53, 3	106, 1	4300	898	83, 5	45, 7
× 18	40, 1	150	18	16, 0	8, 0	43, 7	61, 7	53, 7	106, 1	5100	1050	98, 7	45, 4

I_u	W_u	i_u	I_v	W_{v1}	W_{v2}	i_v	I_{yz}	classification				
10^4 mm^4	10^3 mm^3	mm	10^4 mm^4	10^3 mm^3	10^3 mm^3	mm	10^4 mm^4	S235	S275	S355	S460	
751	81, 7	50, 0	194	37, 6	42, 1	25, 4	−279	3	3	3	4	L 130 × 12
857	93, 2	49, 7	225	42, 8	48, 6	25, 5	−316	3	3	3	3	× 14
802	81, 0	54, 3	207	38, 5	41, 9	27, 6	−298	3	4	4	4	L 140 × 10
945	95, 5	54, 0	247	45, 0	49, 9	27, 6	−349	3	3	3	4	× 12
1020	103	53, 9	262	47, 2	52, 8	27, 4	−377	3	3	3	4	× 13
1080	109	53, 7	283	50, 5	56, 9	27, 5	−399	3	3	3	3	× 14
992	93, 5	58, 2	256	44, 8	48, 5	29, 6	−368	3	4	4	4	L 150 × 10
1170	110	58, 0	302	51, 8	57, 1	29, 4	−435	3	3	4	4	× 12
1340	127	57, 7	347	58, 3	65, 2	29, 3	−499	3	3	3	3	× 14
1430	134	57, 6	369	61, 4	69, 2	29, 3	−529	3	3	3	3	× 15
1670	157	57, 1	434	70, 3	80, 8	29, 2	−616	3	3	3	3	× 18

6. ANNEX

6.1. LIST OF PICTURES

6.2. LIST OF TABLES

6.3. LIST OF USED SOURCES

6.3.1. STANDARTS

• EN 1990: Basic of structural design.

• EN 1991: Actions on structures.

• EN 1993-1-1: Design of steel structures – Part 1-1: General rules and rules for buildings.

• EN 1993-1-8: Design of steel structures – Part 1-8: Design of connections.

• EN 1994-1-1: Composite steel and concrete structure.

6.3.2. LITERATURE AND PROFESSIONAL MAGAZINES

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• Wald, F. et all: Structural steel design according to Eurocodes, Prague 2012, ISBN: 978-80-01-05046-0.

• da Silva, L.S. et all: Design of steel structures, ECCS Eurocode Design Manuals, 2010, ISBN: 978-92-9147-098-3.

• M.G. Lay: Structural Steel Fundamentals, Australia 1982. ISBN: 0-86910-077-7.

6.3.3. INTERNET SOURCES

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[2] BAUTECHNIKGESCHICHTE. *On the evolution of steel-concrete composite construction*. [online]. Available from: goo.gl/Vn7A0N

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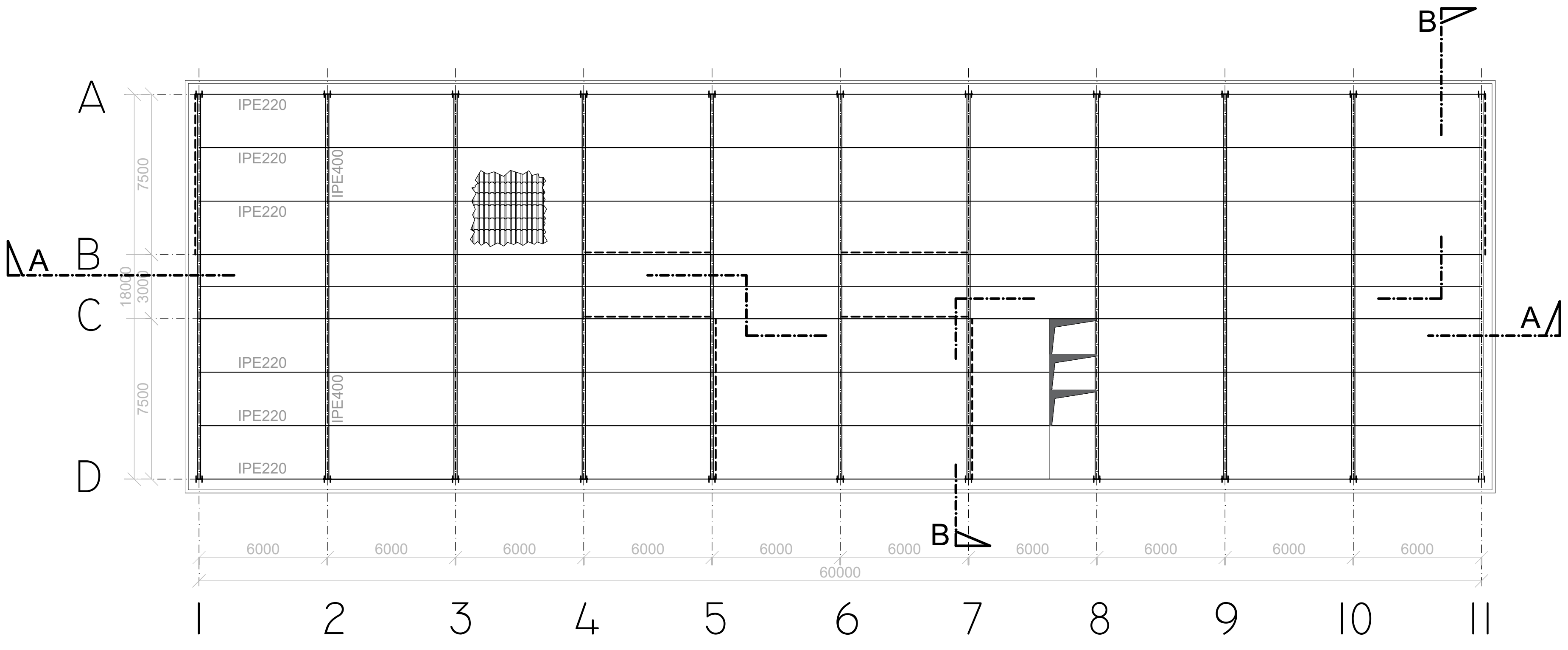
7. BUILDING AND DETAILS PLAN

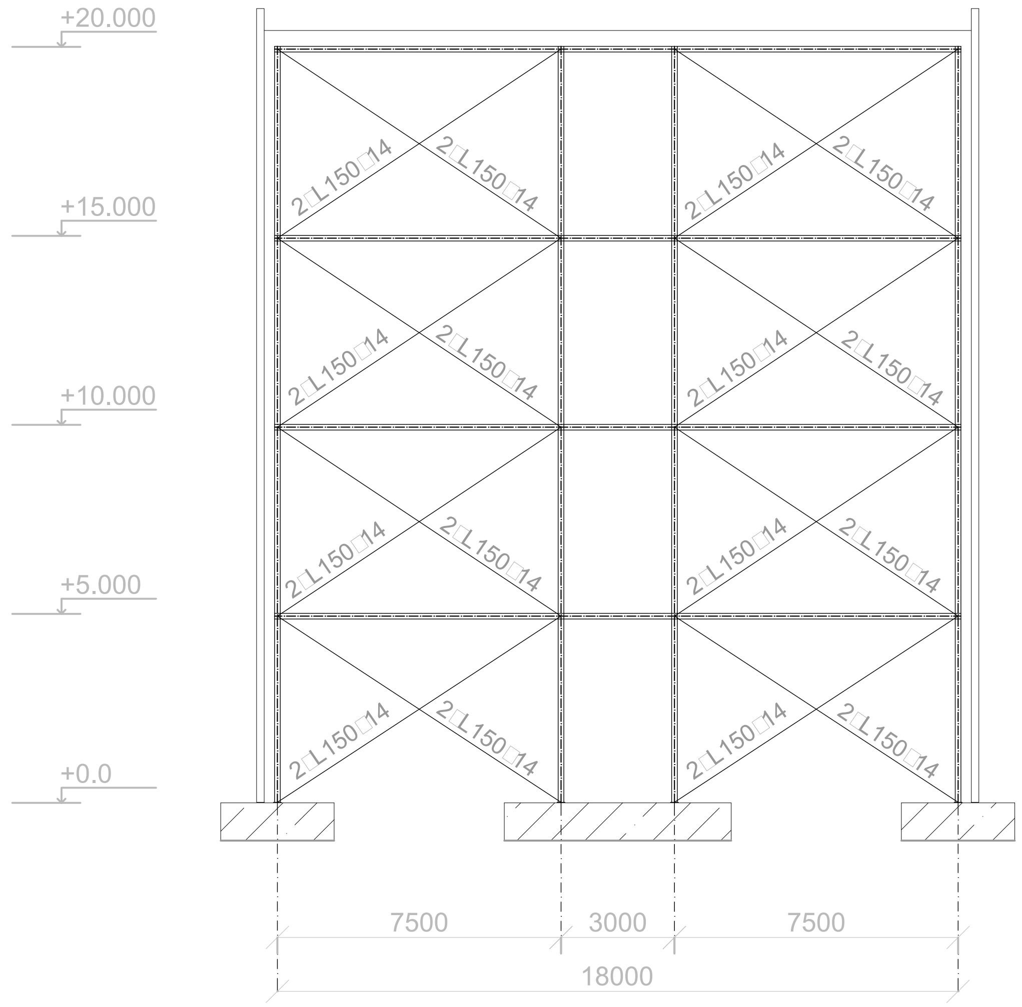
IN APRECIATION FOR THE TECHNICAL UNIVERSITY OF OSTRAVA (VSB), SPECIALLY THE FACULTY OF CIVIL ENGINEERING FOR PROVIDING THE MEDIA AND FACILITATING THE POSSIBILITY TO WRITE THE THESIS IN ITS UNIVERSITY.

IN ADDITION TO ING. ROSMANIT MIROSLAV, PH.D. FOR DEDICATING HIS TIME AND TO SHARE WITH ME MANY OF HIS KNOWLEDGE.

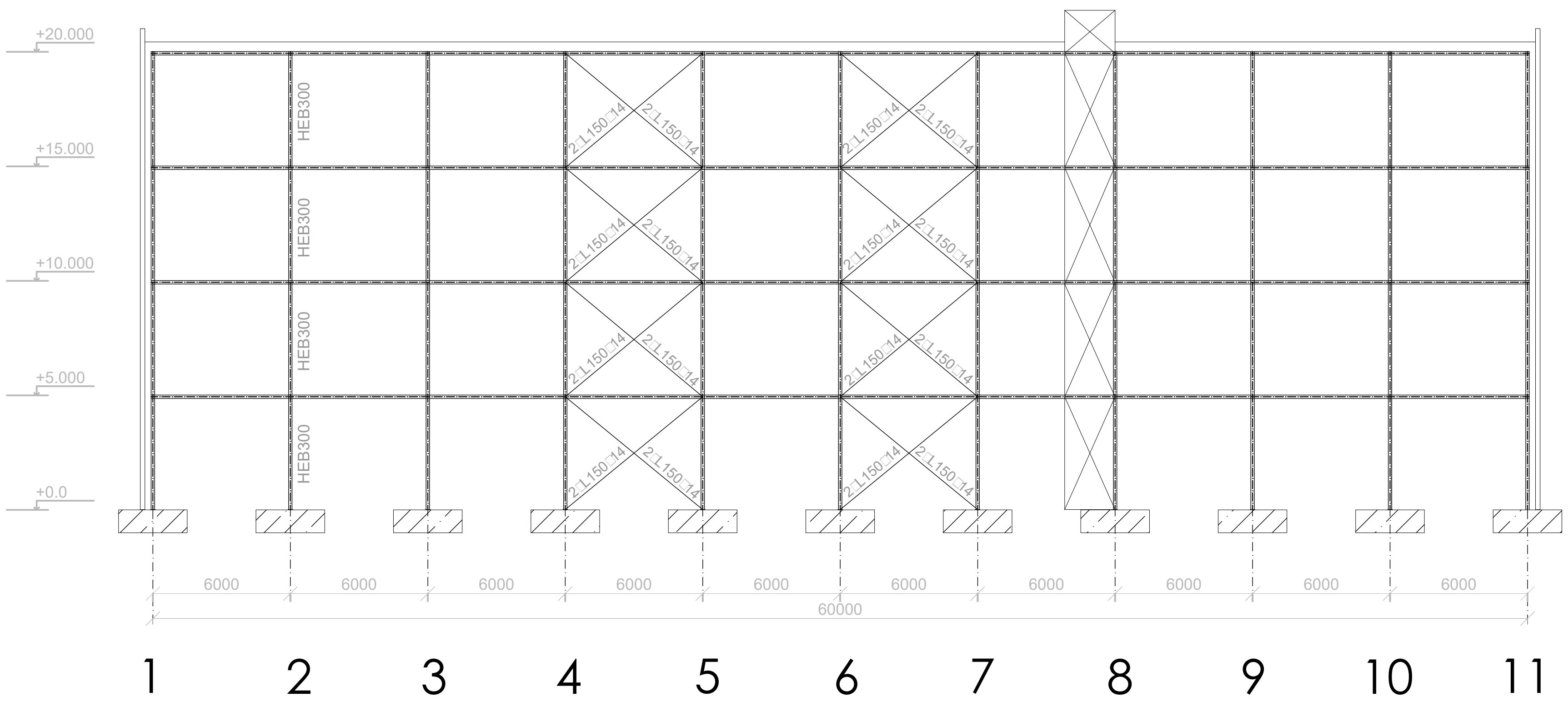
AND LAST TO THE ESCUELA POLITÉCNICA DE GIJÓN, IN A BIG WAY TO MARIÁN GARCÍA PRIETO, FOR THE FACILITIES OFFERED TO MAKE THIS THESIS POSSIBLE.



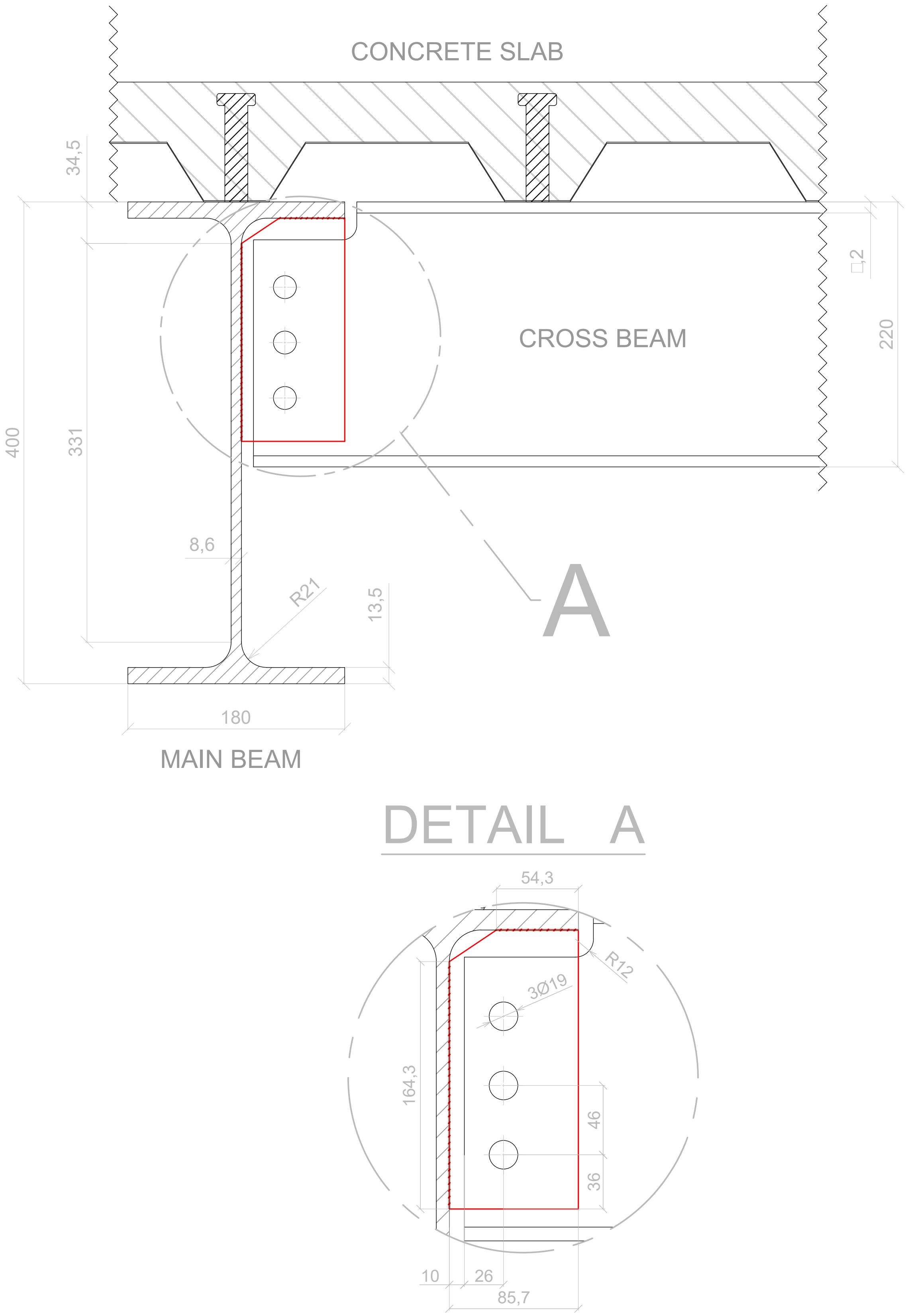
CUT B-B



CUT A-A



SUPERVISOR	STUDENT	FACULTY OF CIVIL ENGINEERING VSB - TECHNICAL UNIVERSITY OF OSTRAVA DEPARTMENT OF STRUCTURES	
ING. MIROSLAV ROMANIT, Ph.D.	ALEJANDRO VALLE		
NAME OF THE THESIS		FORMAT	2□A4
STATIC DESIGN OF COMPOSITE STEEL AND CONCRETE BUILDING STRUCTURE		DATE	05/05/2017
NAME OF THE PLANE		STUDY PROGRAM	ERASMUS +
SIDE & FRONT & GROUND PLAN		SCALE:	1:300 & 1:200



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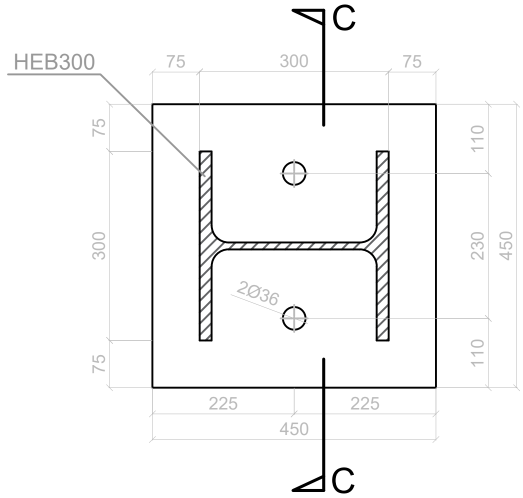
ING. MIROSLAV ROMANIT, Ph.D.	ALEJANDRO VALLE		
NAME OF THE THESIS		FORMAT	A4
STATIC DESIGN OF COMPOSITE STEEL AND CONCRETE BUILDING STRUCTURE		DATE	05/05/2017
NAME OF THE PLANE		STUDY PROGRAM	ERASMUS +
BOLT CONNECTION		SCALE:	1:5 & 1:4



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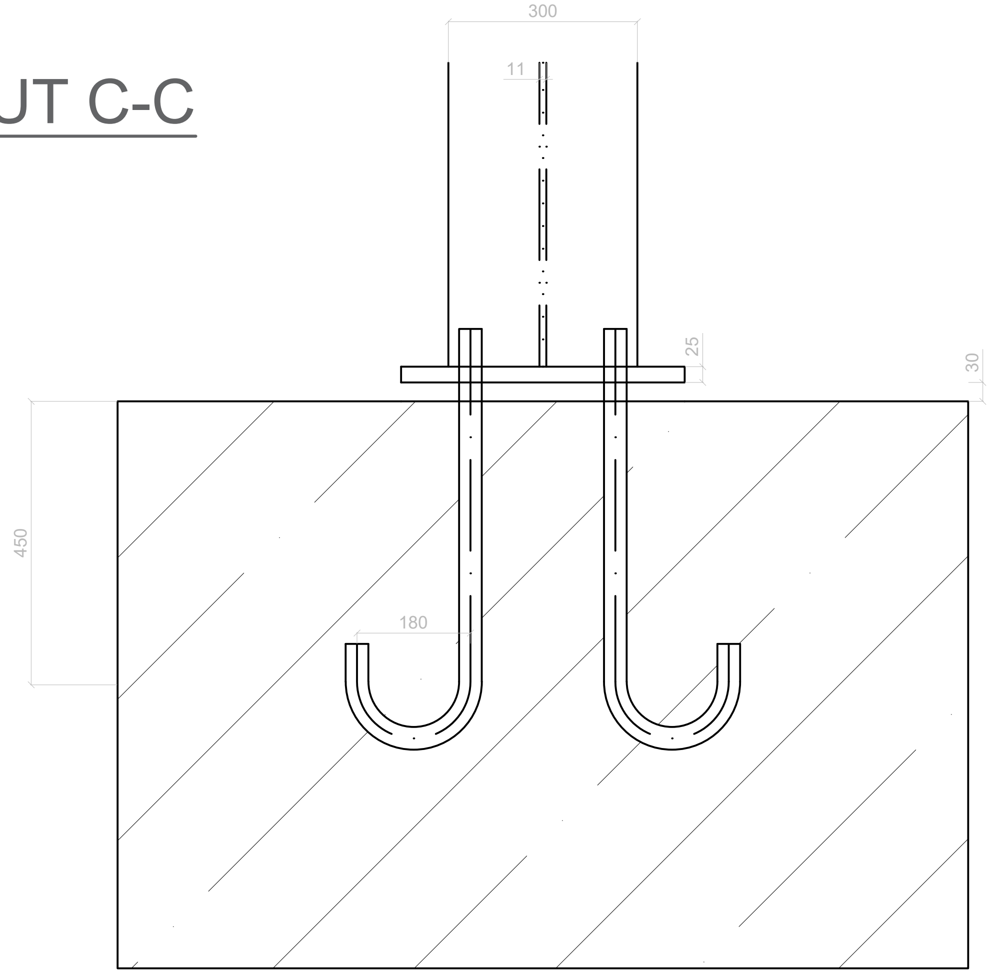

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ING. MIROSLAV ROMANIT, Ph.D.	ALEJANDRO VALLE		
NAME OF THE THESIS		FORMAT	A4
STATIC DESIGN OF COMPOSITE STEEL AND CONCRETE BUILDING STRUCTURE		DATE	05/05/2017
NAME OF THE PLANE		STUDY PROGRAM	ERASMUS +
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