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Faculty of Architecture, Urban Planning, Civil Engineering and Hydraulics

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Course handout

STEEL WORK

Courses and corrected exercises



***Intended for students in Master 1 Materials in Civil Engineering,
Structure, VOA and Master 2 Mechanical Construction***

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Foreword

This handout, entitled "Steel Structures: Courses and Exercises," is intended for Master 1 students in Materials in Civil Engineering. It also represents a very useful reference for those with steel structures in their curriculum, such as students in Structure, VOA, and Mechanical Engineering with a mechanical construction option. It is written in a simplified manner, and examples are introduced after providing basic concepts so that students can assimilate the course content and have a clear understanding of its application in real life. Exercises are accompanied by their solutions at the end for students to practice. The methods given in this handout are based on the Algerian rules for the design of steel structures (CCM97) and Eurocode 3. This document is divided into four chapters:

Chapter I: Design of Assemblies

Chapter II: Design of Column Bases

Chapter III: Design of Composite Floors with Composite Slabs

Chapter IV: Design of Hall-Type Industrial Buildings

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CHAPTER I: CALCULATION OF ASSEMBLIES

I.1 Introduction

Joints in steel structures are generally made by welding and/or bolting. The essential characteristic of steel structures consisting of beams and columns is that they are composed of rolled or welded sections produced in a factory and then assembled on the construction site. Joints therefore have a dual role: to enable the construction of the steel structure capable of supporting the applied loads and overloads and, above all, to contribute to its durability and overall stability. Steel structures are a set of bar elements (columns and beams) that are assembled together to form a frame. Therefore, joints play a very important role in this type of construction.

I.2 Welding

Welding involves melting steel locally with or without filler metal (always steel) to restore the material's continuity as perfectly as possible. The most common process in steel construction is arc welding, which uses the heat produced by an electric arc to bring the steel to its melting point. Welding is a very efficient process, but it may require subsequent inspection of the assembled parts (visual inspection, X-rays, etc.). Some welding is most often performed in the workshop, sometimes on automated benches (e.g., for PRS). Most steel construction companies are now well-equipped with welding benches. Welding positions can be performed for parts:

- flat end-to-end;
- flat overlapped;
- flat corner.

There are two types of welded joints:

- Those for which the joints are designed to ensure material continuity, which requires appropriate machining before welding, as shown in Figure I.1.

In these cases, verification is not necessary since it is considered that there is continuity of the material.

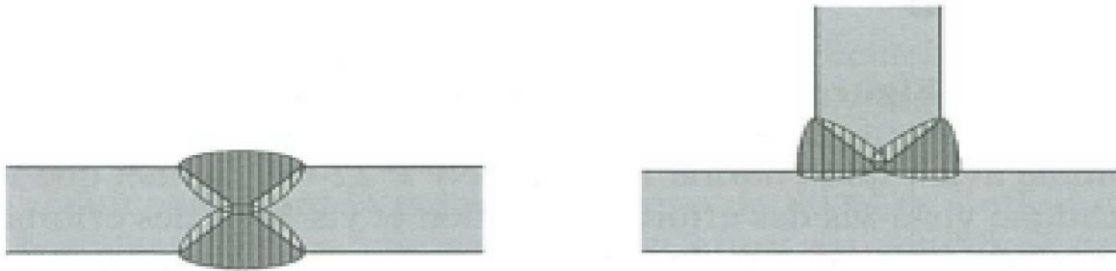


Figure I.1: Double penetration weld joint [7]

Those for which the transmission of forces is carried out through angle beads consisting of metal deposits forming dihedrals between the faces of the parts to be joined, Figure I.2. In this case, a check of the dimensioning of the weld bead is necessary.

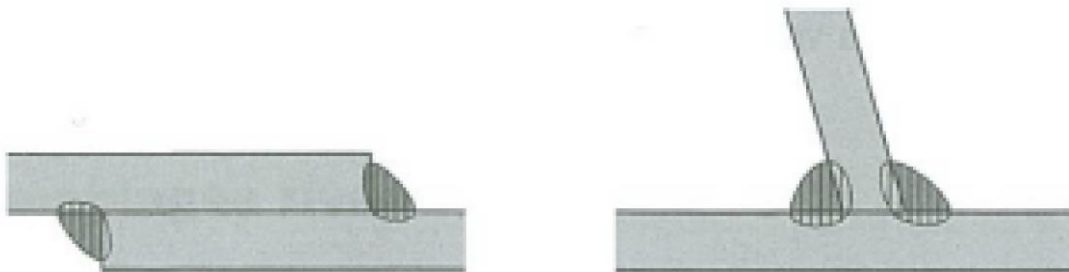


Figure I.2: Fillet weld joint [7]

Welding is a process that allows parts to be joined by an intimate bond between the material, obtained by fusion or plasticization. Welding has several advantages over bolting:

- It ensures material continuity, and therefore guarantees good stress transmission;
- It eliminates the need for secondary parts (gussets, fasteners, etc.);
- It is smaller and more aesthetically pleasing than bolting. However, it has several disadvantages:

- The base metal must be weldable;
- Weld inspection is necessary and expensive;
- Welders' inspection is unpredictable;
- Welding requires skilled labor and specific equipment.

I. 3 Welding Processes

These include:

- pressure process;
- electrical resistance process;
- friction process;
- chemical process using an oxyacetylene torch;
- laser process;
- electron bombardment process;
- plasma arc process;
- electric arc process.

I. 3.1 Chemical Process Using an Oxyacetylene Torch

This process uses the combustion of oxygen and acetylene at a temperature of approximately 3000°C. The filler metal is supplied by fusible steel rods. This process is rarely used in steel construction because it is more expensive than arc processes for thick steel sections.

I.3.2 Electric arc processes

These are the most commonly used processes in metal construction. An electric arc is created between a consumable electrode (cathode) and the parts to be welded (anode) using a low-voltage, high-amperage alternating or direct current generator. The arc (temperature at the core of the arc $\cong 5500^{\circ}\text{C}$, 2700°C at the cathode, and 3500°C at the anode) melts the metal of the electrode and part of the parts to be joined, as well as the electrode coating. The molten metal fills the voids between the parts and, after cooling, forms the assembly.

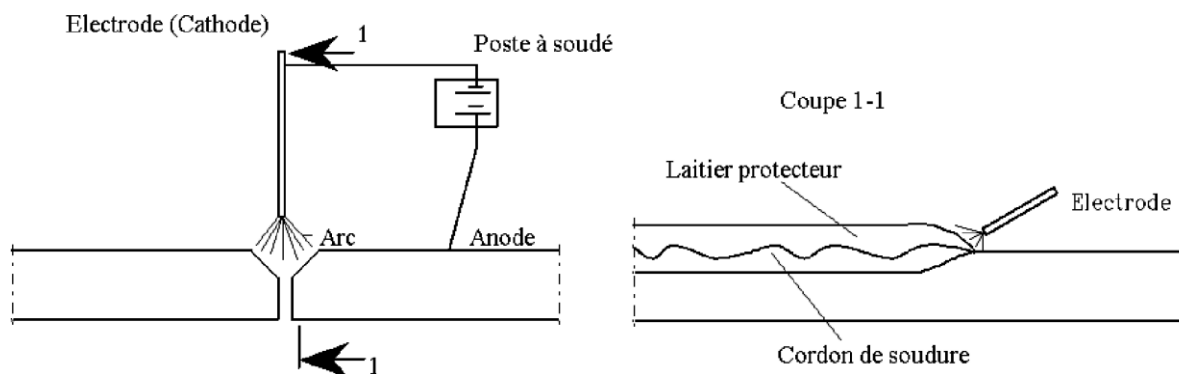


Figure I.3: Arc welding principle [9]

I.3.3 Non-consumable electrode (TIG) processes

The arc is produced between a non-consumable tungsten electrode and the work pieces, under an ARGON jet, which is an inert gas. The filler metal is obtained by melting a separate rod. In the workshop, this process is semi-automatic or automatic.

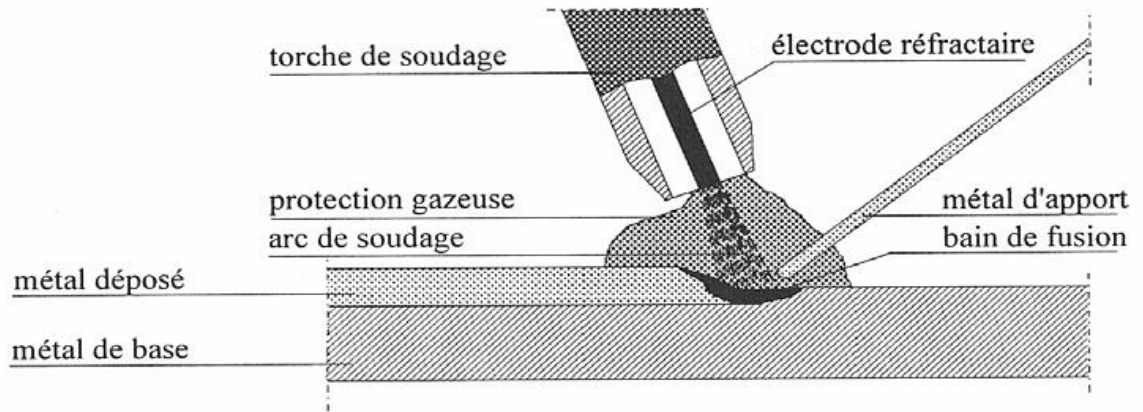


Figure I.4: Gas-filled arc welding – TIG process [10]

I.3.4 Consumable electrode processes

An electric arc is created between a consumable electrode (cathode) and the work pieces to be welded (anode), using a low-voltage, high-amperage (50 to 600 amperes) alternating or direct current generator. The molten particles from the cathode are projected onto the anode through the arc, where they are deposited. The cathode then simply needs to be moved along the assembly joint to create a continuous weld bead.

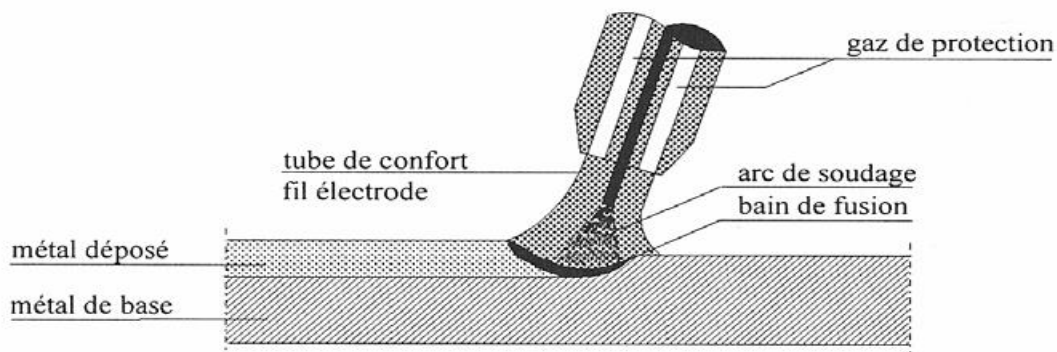


Figure I.5: Gas-assisted welding [10]

Protection from the ambient atmosphere is achieved by a gas blown into a nozzle concentric with the electrode. If the gas is inert, this is the MIG process. For an active gas, this is the MAG process.

I.3.5 Other Types of Welded Joints

1.3.5.1 Notch Weld

Notch welds, consisting of fillet welds made in circular or elongated holes



Figure I.6: Notch weld [9]

1.3.5.2 Plug welds

Plug welds are made in circular or oblong holes.



Figure I.7: Plug Weld [9]

I.4 Construction Precautions

Welding steel requires high temperatures, which will cause local expansion of the parts.

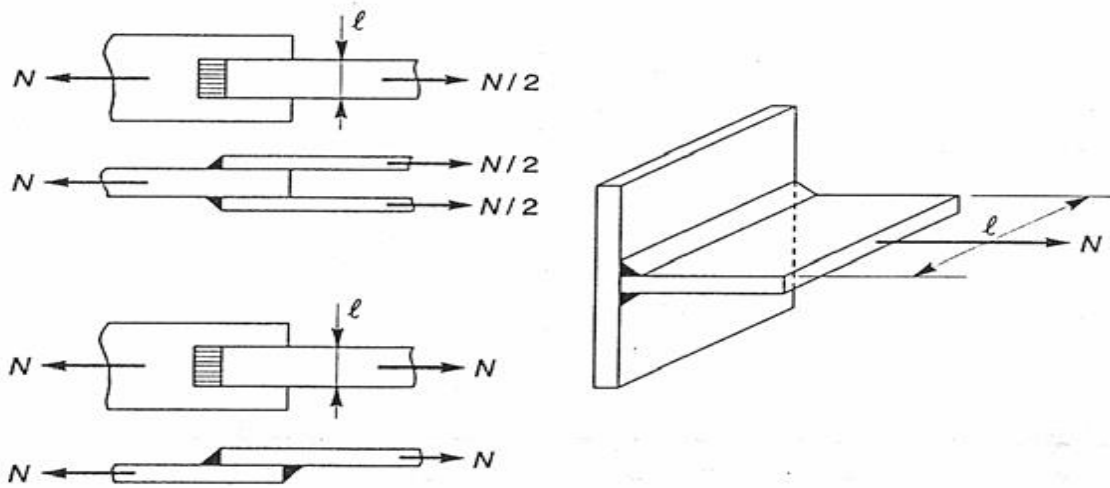
As the weld bead area cools, shrinkage will:

1. either cause deformations in the parts, if they are freely expandable.
2. or generate internal stresses in the parts, if they are clamped. In the first case, to address the deformations, it is possible to:
 - either give the parts initial inverse deformations, which will compensate for the shrinkage deformations;
 - or straighten the parts cold, using a press;
 - either create the beads in discontinuous sections spaced apart over time; or preheat the parts to avoid sudden cooling. In the second case, to limit internal stresses, it is possible to:
 - either relax the clamping, which will allow for acceptable, low deformations;

- or post heat the parts. A few other basic precautions must be taken:
- Avoid joining parts with excessively different thicknesses, as there is a risk of deformation of the thinner part and the risk of cracking of the weld bead upon cooling.
- Avoid joining parts with a thickness greater than 30 mm.
- Use beads with a diameter greater than 4 mm and a length greater than 50 mm.
- Ensure a good correlation between the bead thickness and the thickness of the thinnest part to be joined.

I.5 Calculating Weld Beads Butt welds are not calculated.

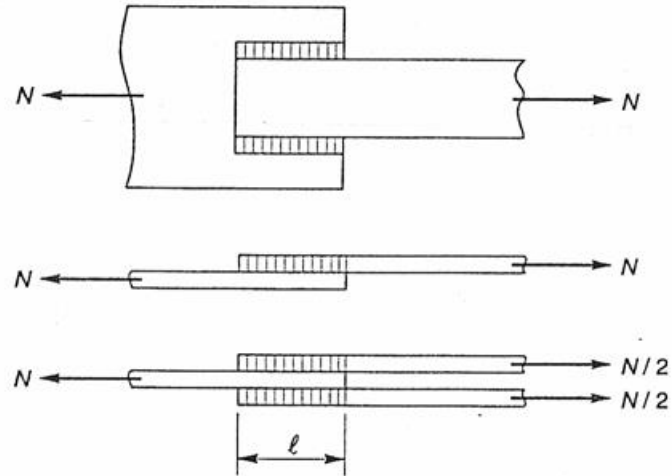
I. 5.1 Face Beads



Assembly by front cords

$$a \cdot \sum l \geq \beta_w \cdot \gamma_{Mw} \frac{N \sqrt{2}}{f_u}$$

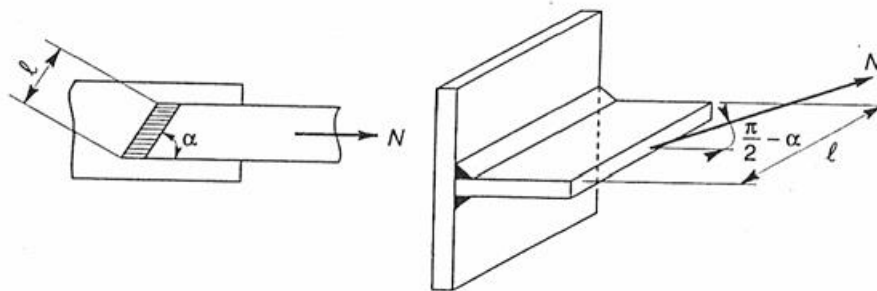
I.5.2 Lateral cords



Assembly by side cords

$$a \cdot \sum l \geq \beta_w \cdot \gamma_{Mw} \frac{N\sqrt{3}}{f_u}$$

I.5.3 Oblique cords



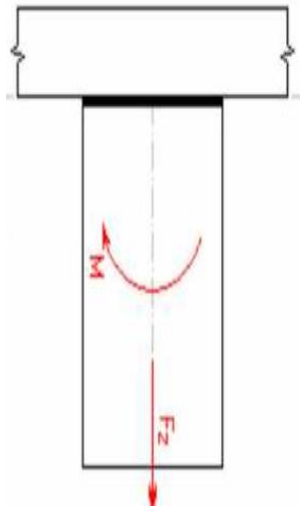
Assembly by oblique cords

$$a \cdot \sum l \geq \beta_w \cdot \gamma_{Mw} \frac{N\sqrt{3 - \sin^2 \alpha}}{f_u}$$

NOTATIONS: an effective thickness or groove, minimum distance from the root to the surface of the bead; l: effective length of the bead; N: factored force applied to each bead, assumed to be centered in the middle of the bead length; The coefficients β_w and γ_{Mw} vary depending on the steel grade:

Nuances d'acier		γ_{Mw}	β_w	$\beta_w \cdot \gamma_{Mw}$
f_y	f_u			
235 MPa	360 MPa	1.25	0.80	1.00
275 MPa	430 MPa	1.30	0.85	1.10
355 MPa	510 MPa	1.35	0.90	1.20

Exercise 1: Consider an 85x85x5 tube, welded to a plate with a perimeter bead of thickness $a=5$ mm. 1/ What weighted axial force N can it withstand? S235 steel



This is a front cord, you need to check that

$$: a \sum L \geq Bw \times \gamma_{Mw} \cdot \frac{N\sqrt{2}}{f_u}$$

$$N \leq \frac{a \cdot \sum L \times f_u}{\gamma_{Mw} \times \beta_w \sqrt{2}}$$

With : $a = 5$ mm

$$\sum L = 4 \times 85 = 340 \text{ mm}$$

$$\gamma_{Mw} \cdot \beta_w = 1$$

$$F_u = 360 \text{ MPa}$$

From which we derive $N \leq 400 \text{ KN}$

However, it is advisable to check the tensile stress in the tube.

$$\sigma = \frac{N}{A} = \frac{400}{14,7 \times 12^2} = 275,86 \text{ MPa} > f_y = 235$$

$$(C.N.V), \sigma = \frac{N}{A} \leq [\sigma] \rightarrow N \leq A \cdot [\sigma] = 345,45 \text{ KN}$$

Weld Length Verification The weld bead length must be:

$L_j \leq \min(6 \times a ; 40 \text{ mm})$ if (c.v), so the bead is short and is not taken into account for force transmission.

$$L_j = 80 \text{ mm} \leq \min(6 \times 5 ; 40 \text{ mm}) = \min(30 ; 40 \text{ mm})$$

$$L_j = 80 \text{ mm} \leq 30 \text{ mm} \text{ (C.V)}$$

So the beads are not short

$$L_j > 150 \times a \text{ (long weld bead)}$$

$$80 < 150 \times 5 = 750 \text{ mm}$$

So the weld beads are not considered long

Exercise 2:

Consider a welded assembly of two UAP85 angles attached to an intermediate gusset plate. Each weld bead has a groove $a = 8 \text{ mm}$ and a length $L_j = 70 \text{ mm}$. The total force applied to the two angles is $N_{sd} = 480 \text{ kN}$. The strength of the steel used is S235 ($f_u = 360 \text{ MPa}$). Check the resistance of the weld beads to this force.

Solution

- Verifying the weld length: The weld bead length must be:

$$L_j \leq \min(6a ; 40 \text{ mm}) \rightarrow L_j = 70 \text{ mm} \quad \min(6 \times 8 ; 40 \text{ mm})$$

$$L_j = 70 \text{ mm} > 40 \text{ mm}$$

$$L_j > 150 \times a = 1200 \text{ mm} \text{ (C.V.N)}$$

Weld Bead Strength Verification The stress applied to these weld beads is lateral

$$\rightarrow \sigma_{\perp} = 0 \text{ and } \tau_{\perp} = 0 \text{ And } a \times \sum L \geq \beta_w \times \gamma_{Mw} \times \frac{N \times \sqrt{3}}{f_u} \rightarrow N \leq \frac{f_u \cdot a \times \sum L}{\beta_w \times \gamma_{Mw} \times \sqrt{3}}$$

$$F_u = 360 \text{ MPa} \quad ; \beta_w \times \gamma_{Mw} = 1$$

$a = 8 \text{ mm}$ (the weld bead throat).

$$(360 \times 8 \times (70 + 70)) / (1 \times \sqrt{3}) \text{ therefore } N \leq 249.42 \text{ KN}$$

- Force absorbed by each angle:

$$N_{i.sd} = 480 / 2 = 240 \text{ KN.}$$

Force absorbed by each weld bead:

$$F_{i.sd} = 240 / 2 = 120 \text{ KN}$$

The stress applied to each bead per unit length is:

$$F_{w.sd} = (F_{l.sd})/L_j \rightarrow F_{w.sd} = (120 \times 10^3)/70 = 1600$$

$$\sigma = N/A_{uAP85} \leq f_y$$

$$\sigma = \frac{N}{10,7 \times 10^2 \times 2} = \frac{249,42 \times 10^3}{2140} = 116,55 \text{ MPa}$$

so we have 116.55 MPa < 235 MPa (CV)

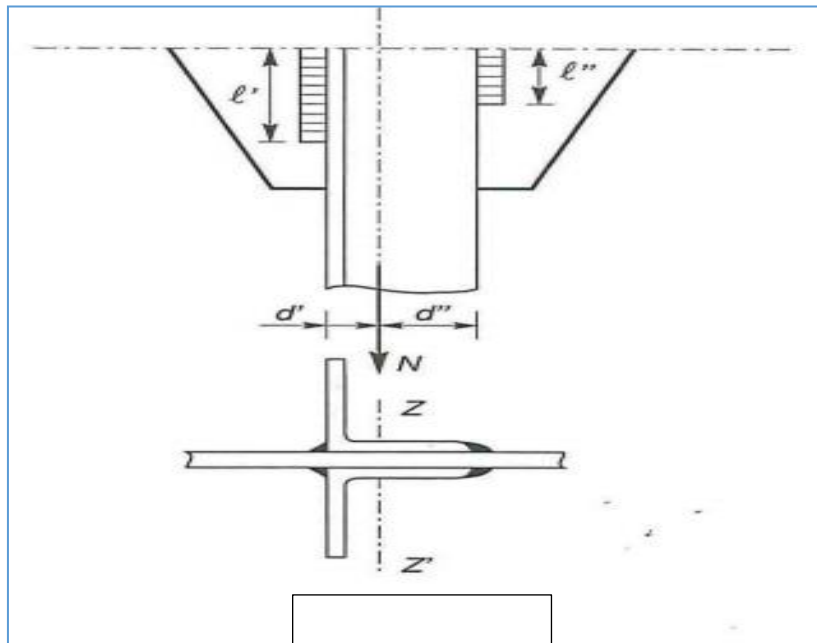
Exercise 3

Consider two 90×90×9 angle bars, welded to a gusset plate with beads of thickness a=4mm.

- The factored tensile force N applied to the neutral axis is N=40000 daN.

- d'= 23mm and d'' = 57mm. - S235 steel.

1/ Calculate the length of the weld beads.



Solution

$$\text{Side cords: } a \sum L \geq \beta w \times \gamma M_w \frac{N \cdot \sqrt{3}}{f_u}$$

The center of gravity of the welding beads must be located on the neutral axis of the ZZ' angles; in this case, their static moments are equal:

$$L' \cdot d' = L'' \cdot d''$$

$$\sum L = 2(L' + L'')$$

$$\sum L \geq \beta w \times \gamma M \cdot W \frac{N \times \sqrt{3}}{f_u \times a}$$

$$L' \cdot d' = L'' \cdot d'' \rightarrow L'' = L' \times \frac{d'}{d''} \text{ et } \sum L = 2(L' + L' \cdot \frac{d'}{d''}) \rightarrow \sum L = 2L'(1 + \frac{d'}{d''})$$

$$2L'(1 + \frac{d'}{d''}) \geq \frac{\beta w \times \gamma M w \times N \times \sqrt{3}}{f_u \times a}$$

$$L' \geq \frac{\beta w \times \gamma M w \times N \times \sqrt{3}}{2 \times f_u \times a \times (1 + \frac{d'}{d''})}$$

$$L' \geq \frac{1 \times 400000 \times \sqrt{3}}{2 \times 360 \times (1 + \frac{23}{57})}$$

$$L' \geq 171,4 \text{ mm} \rightarrow L'' = 69,16 \text{ mm}$$

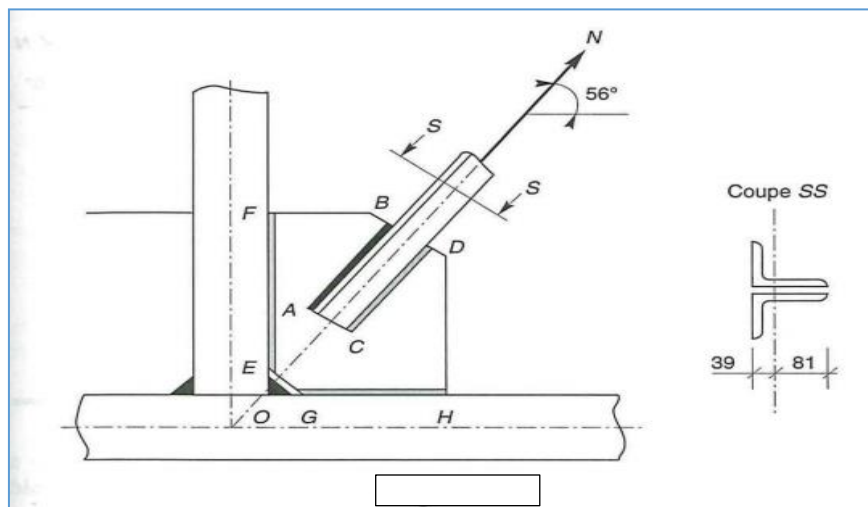
So we adopt: $L' = 17,14 \text{ cm}$

$$L'' = 6,916 \text{ cm}$$

Exercise 4

Given a truss diagonal, consisting of a double angle iron L120×80×10, $N=800\text{KN}$ (400KN for each of the angles)

1/ Calculate the weld seams.



Solution:

$$\text{Oblique cords: } a \cdot \sum L \geq \beta w \times \gamma M w \frac{N \sqrt{3 - \sin^2 \alpha}}{f_u}$$

So you have to check:

$$\sum L \geq \beta W \times \gamma M w \frac{N \sqrt{\sin^2 \alpha}}{f_u \times a}$$

Effort in the cord

$$AB : A L120 \times 80 \times 10 = 19,1 \times 10^2 \text{mm}^2$$

$$\sigma = \frac{N}{A} \leq [\sigma]$$

$$N \leq A \cdot \sigma$$

$$N_{AB} \leq 19,1 \times 10^2 \times 235 = 448,85 \text{KN}$$

$$N_{AB} \leq 19,1 \times 10^2 \times 235 = 448,85 \text{KN}$$

$$L_{AB} \geq \frac{1 \times 448,85 \times \sqrt{3 - \sin^2 34}}{360 \times a}$$

Let's adopt: $a = 5 \text{mm}$

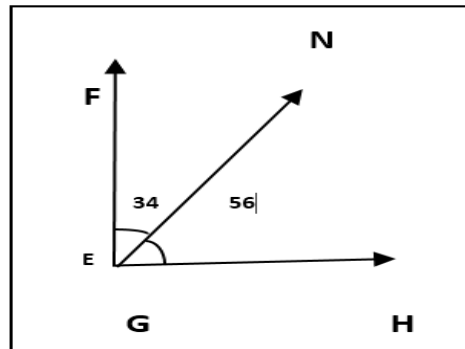
$$\sin 34 = 0,56$$

$$L_{AB} = 408,7 \text{mm}$$

$$\alpha = 56$$

$$L_{cd} \geq 380,43 \text{mm}$$

$$\sin \alpha = 0,82$$



EF cords:

$$\alpha 1 = 34$$

$$\sin \alpha = 0,56$$

$$\sum L = 2 \times EF = 800$$

$$N_{EF} = 663,23 \text{KN}$$

$$\alpha 2 = 56$$

$$\sin \alpha = 0,82$$

$$\sum L = 2 \times GH = 500$$

Cords must be checked

$$a \geq \frac{\beta w \times \gamma MW \cdot N \sqrt{3 - \sin^2 \alpha}}{f \times \Sigma L}$$

$$a_1 \geq \frac{1 \times 663,23 \times 10 \sqrt{3 - \sin^2 \alpha}}{360 \times 800}$$

$$a_1 \geq 3,77 \text{ donc on adopte } a_1 = 4 \text{ mm}$$

$$a \geq \frac{1 \times 447,35 \times 10^3 \sqrt{3 - \sin^2 56}}{360 \times 500}$$

$$a \geq 3,79 \text{ mm}$$

so we adopt $a_2 = 4 \text{ mm}$

Exercise 05

What is the maximum axial force that the S235 steel U-shaped profile connection with the plate can withstand, using a 6 mm thick bead?

Solution

These are front beads that must satisfy the following condition:

$$a \Sigma l \leq \beta w \times \gamma MW \times \frac{\sqrt{2} \times N}{f_u}$$

The maximum axial force is:

$$N \leq \frac{a \times \Sigma L \times f_u}{\beta w \times \gamma MW \times \sqrt{2}} = \frac{6 \times 150 \times 360}{0,8 \times 1,25 \times \sqrt{2}}$$

$$N \leq 229,13 \times 10^3 \text{ N}$$

The tensile stress in the U-shaped profile:

$$\frac{N}{A_b} = \frac{229,13 \times 10^3}{8,62 \times 10^2} = 265,81 \text{ MPa} > f_y = 235 \text{ MPa} \quad (C.N.V)$$

$$\text{So } A_{\min} = 9,75 \times 10^2 \text{ mm}^2$$

$$N \leq A \times f_y = 8,62 \times 10^2 \times 235 = 202570 \text{ N}$$

$$N = 202570 \text{ KN}$$

Exercise 06

Consider the connection of a U-shaped profile made of S235 steel to a 7 mm thick gusset plate, using weld beads of thickness $a = 6 \text{ mm}$. The connection is subjected to a force $N_{sd} = 398 \text{ kN}$ applied along the axis of the profile.

- 1) Calculate the length of the weld beads.

Solution

These are lateral beads; the following condition must be met:

$$a \sum L \geq \beta w. \gamma MW. \frac{N \cdot \sqrt{3}}{f_u} \rightarrow$$

$$\sum L = \frac{0,8 \times 1,25 \times 398 \times 10^3 \times \sqrt{3}}{6 \times 360}$$

$$\sum L \geq 319,146 \text{ mm} \rightarrow \sum L = 320 \text{ mm}$$

The length of the welding bead:

$$L = \frac{320}{2} = 160 \text{ mm}$$

So : $L = 160 \text{ mm}$

I.6 Definition and Role of an Assembly

The essential characteristic of metal structures is that they are composed of a set of bar elements (columns and beams) made of rolled or welded profiles, often in the shape of an I or H, that must be assembled together to form the frame. The connections between these different elements represent what are commonly called assemblies. These are components specific to metal construction; they play a very important role. They can be defined as connecting members that allow several elements to be joined and secured together, ensuring the transmission and distribution of various stresses between the assembled elements without generating unwanted forces. A poorly designed, poorly calculated, or poorly executed assembly can lead to the collapse of the structure. Therefore, the design and calculation of assemblies is of paramount importance.

I.7 Different Forms of Assembly Encounters in C.M.

In the frames of metal buildings, structural elements are connected by assemblies. Depending on the nature of the assembled elements, we distinguish:

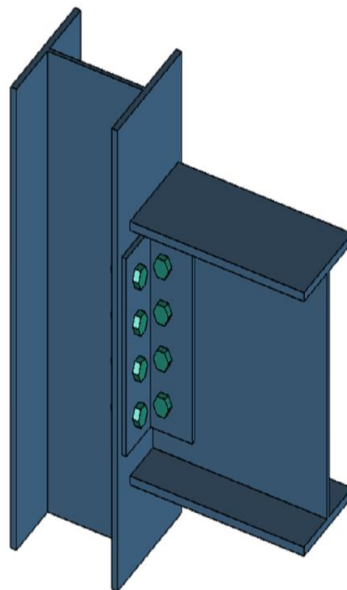


Figure I.8: Articulated beam-column assembly [6]

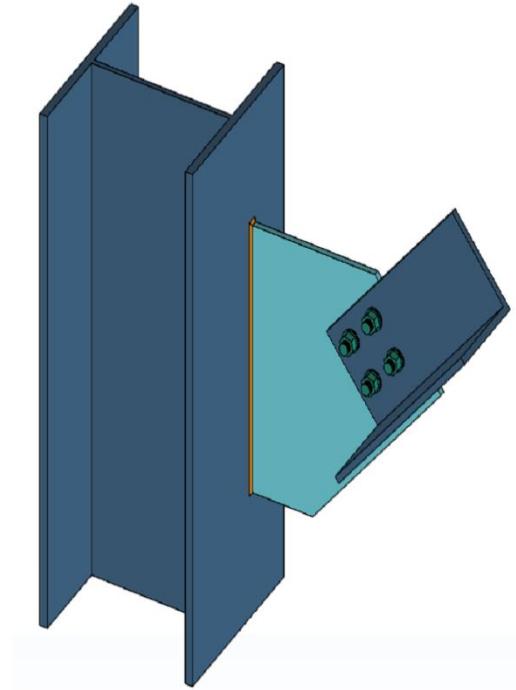


Figure I.9: Assembly of a bracing angle on a post [6]

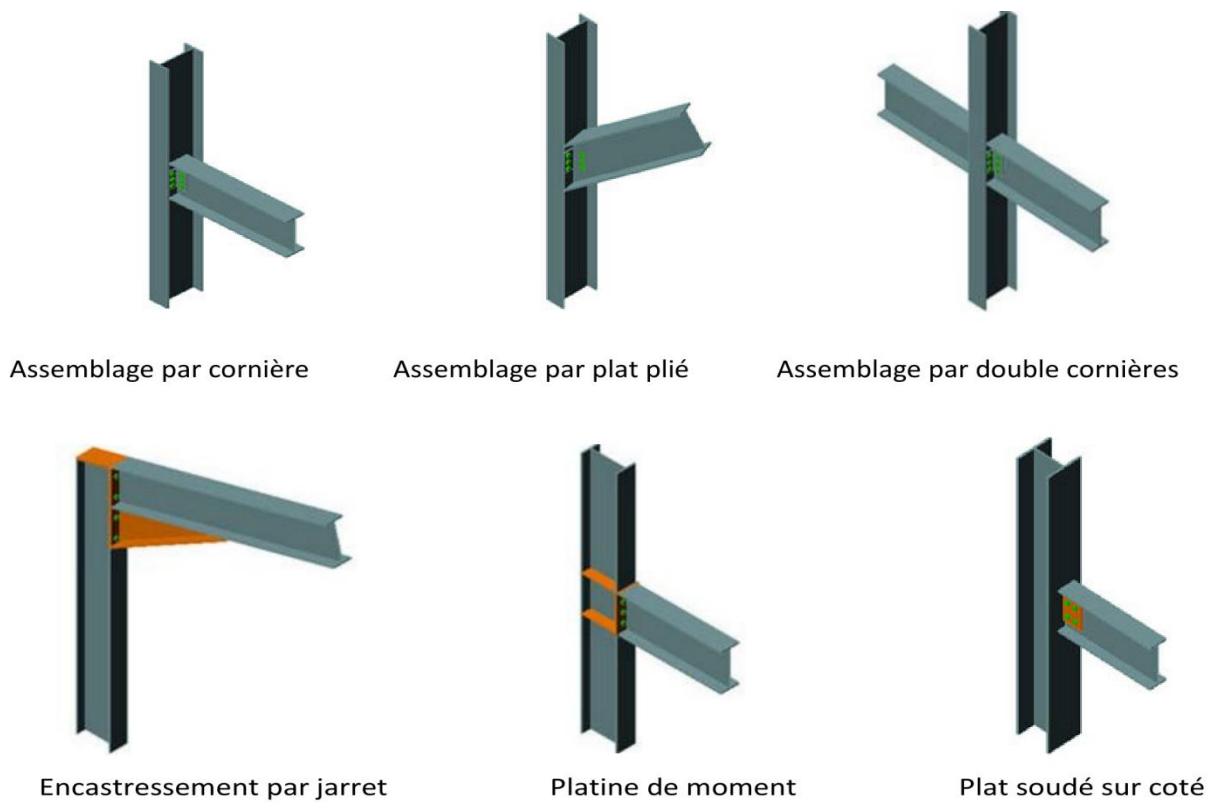


Figure I.10: Beam-column assembly [9]

I.8 Joining Methods

The various types of joining mentioned above are generally achieved using the following main joining methods:

A) - Riveting Rivets were the first joining method used in metal construction. Currently, their use is limited, and in most industrialized countries, bolts and welding are preferred. They are therefore mainly found in older structures dating from the beginning of this century. Their diameter generally ranges from 10 to 28 mm. (Figure I.11)

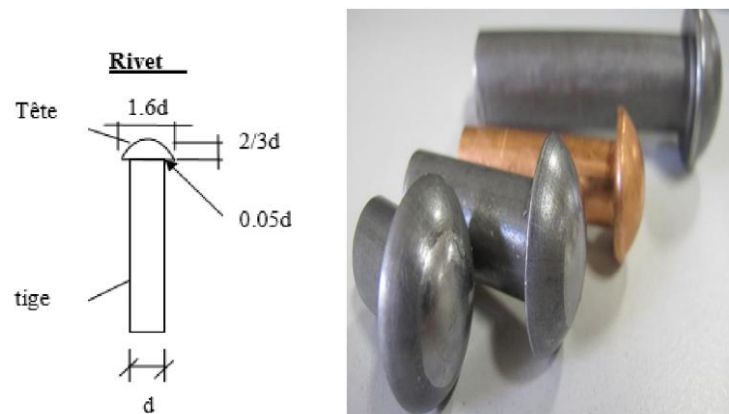


Figure I.11: Rivets [9]

B) Bolting

The characteristics of the different types of steel used for bolts are shown in Table I.1, which shows the yield strength (f_{yb}) and tensile strength (f_{ub}) values for the four quality classes of steel used for bolts.

Table I.1: Mechanical properties of steels for bolts

Boulons	Classe de qualité	f_{yb} (N/mm ²)	f_{ub} (N/mm ²)
De charpente	4.6	240	400
	5.6	300	500
A haute résistance	8.8	640	800
	10.9	900	1000

This table also shows that there are two types of bolts, which differ in their mechanical properties:

Structural steel bolts (steels 4.6 and 5.6).

High-strength bolts (steels 8.8 and 10.9).

Structural steel bolts are commonly used for lightly loaded connections in halls and buildings. High-strength bolts are generally used for bridge connections, as well as for connections subject to high stress or dynamic effects. Only high-strength bolts can be prestressed. Eurocode 3 adds quality classes 4.8, 5.8, and 6.8 to those given in the table and uses the concept of ordinary bolt instead of structural bolt.

Note:

Regardless of the bolt type, the normal clearance between the bolt shank and the hole in the assembled parts is set at 2 mm for bolt diameters less than or equal to 24 mm, and at 3 mm for bolt diameters equal to or greater than 27 mm. The hole diameter d_0 is therefore:

$$d_0 = d + 2 \text{ mm for } d \leq 24 \text{ mm.}$$

$$d_0 = d + 3 \text{ mm for } d \leq 27 \text{ mm.}$$

Eurocode 3 provides the same hole diameters, except for bolt diameters $d \leq 14$ mm, for which

$$d_0 = d + 1 \text{ mm.}$$

In some cases, a smaller clearance may be required; these are called close-fit bolts when the clearance is only 0.3 mm. The use of close-fit bolts offers the advantage of creating assemblies with very little possible relative movement, thus creating structures with very low deformation. However, the holmaking must be precise, which significantly increases the cost of this type of assembly. Close-fit bolts should therefore only be used when absolutely necessary. The following figure shows the different components of a structural bolt and a high-strength bolt. These are distinguished from structural steel bolts by the steel grade marking on their heads and washers. A washer must always be included under the part that will be rotated when installing the bolt (usually the nut, sometimes the head, often both). To install bolts in the flanges of profiles,

I.9 Joint Operation

a)- Obstacle operation: This is the case for ordinary, non-prestressed bolts whose shanks absorb the forces and operate in shear.

b)- Adhesive operation: In this case, the forces are transmitted by adhesion of the surfaces of the parts in contact. This applies to welding and bolting with high-pressure bolts.

c)- Mixed operation: This is the case for riveting (and, in extreme cases, high-pressure bolting), where the rivets transmit forces by adhesion of the parts up to a certain limit, which, when exceeded, triggers the rivets to act as an obstacle, in shear.

I.10 Bolted Connections

1.10.1 Connection with Ordinary Bolts

The load transmission mode from one part to another is through the bolt shank, which acts as an obstacle between the joined parts, hence its shear stress. The connection allows for relative movement of the parts (to come into contact with the bolt shank) upon initial loading and then upon each force reversal.

1.10.1.1 Bolt Shear Strength by Shear Plane

The strength depends, among other things, on the position of the shear plane. If the shear plane passes through the threaded portion of the bolt:

For bolt classes 4.6, 5.6, 6.6, and 8.8, the bolt resistance is:

$$\mathbf{F_v.R_d = 0.6 m f_{ub} A_s / \gamma_{Mb} \text{ with } \gamma_{Mb} = 1.25}$$

For bolt classes 4.8, 5.8, 6.8, and 10.9, the resistance is:

$$\mathbf{F_v.R_d = 0.5 m f_{ub} A_s / \gamma_{Mb} \text{ with } \gamma_{Mb} = 1.25}$$

If the shear plane passes through the unthreaded portion of the bolt

$$\mathbf{F_v.R_d = 0.6 f_{ub} A / \gamma_{Mb} \text{ with } \gamma_{Mb} = 1.25}$$

1.10.1.2 Resistance to diametrical pressure of assembled parts:

When the bolt comes into contact with the part, it exerts pressure on the latter (action and reaction principle) for which the part must have sufficient resistance, in order to avoid ovalization of the holes and tearing of the clamps. The resistance is:

$$\mathbf{F_b, R_d = 2.5 \cdot \alpha \cdot f_u \cdot d \cdot t / \gamma_{Mb}}$$

With minimum α of: $\{e/3d_0; p/3d_0 - 1/4; f_{ub}/f_u \text{ or } 1\}$

With: d bolt diameter d_0 hole diameter t part thickness e longitudinal clamp p bolt center distance f_{ub} tensile strength of bolts f_u tensile strength of the assembled part

I.10.1.3 Tensile strength of bolts

The tensile strength of a bolt is equal to:

$$F_t.Rd = 0.9 A_s f_{ub} / \gamma_{Mb} \quad \text{with } \gamma_{Mb} = 1.5$$

I.10.1.4 Strength of bolts subjected to simultaneous shear and tension

Bolts subjected to combined shear and tension forces must meet the following strength criterion:

$$F_v.Sd / F_v.Rd + F_t.Sd / 1.4 F_t.Rd \leq 1$$

I.10.2 Connections using controlled tightening bolts

prestressed bolts A high-strength (HR) bolt is made of high-strength steel and has a washer attached to the head. During assembly or bolting, strong tightening is required, which causes a prestressing force, which is oriented parallel to the bolt axis, therefore perpendicular to the contact planes of the parts. This is why HR bolts are also called prestressed bolts.

I.10.2.1 Operating Principle of an H.R.

Bolt Assembly Mutual friction between the parts is developed by prestressing, which provides high resistance to relative sliding. H.R. bolts do not operate in shear, but transmit forces by friction, unlike the operating conditions of ordinary bolts. Parts in frictional contact, whose coefficient μ , therefore plays an important role. If F is the shear force transmitted by the assembly and acting on the bolt, and F_p is the axial prestressing force in a bolt, it will be necessary to verify that the interface of the contacting parts can transmit the tangential force without slippage. Since H.R. bolts are not designed to operate as obstacles (in shear), their shanks are theoretically not in contact with the cross-sections of the bores of the assembled parts. However, in some cases, the rods may come into contact with the parts, either due to poor assembly or accidentally due to slipping of the parts (insufficient coefficient of friction or excessive shear force). In these cases, HR bolts will operate in shear.

I.10.2.2 Slip-resistant connections

The slip resistance F_s of a prestressed HR bolt is:

$$F_s.Rd = k_s m \mu F_p C_d / \gamma_{Ms} \quad \text{With}$$

m : number of adhesion surfaces

μ : Friction coefficient

$F_p.C_d$: Prestressing force with:

$$F_p.C_d = 0.7 f_{ub} A_s$$

k_s : hole shape coefficient $k_s = 1.0$ for holes with normal tolerances, namely:

- 1 mm for bolts ϕ 12 and ϕ 14
- 2 mm for bolts ϕ 16 to ϕ 24
- 3 mm for bolts ϕ 27 and larger $k_s = 0.85$ for oversized circular holes and short oblong holes.

$k_s = 0.7$ for long oblong holes.

γ_{MS} : is the partial safety factor which is equal to:

- at ULS: $\gamma_{MS} = 1.25$ for holes with normal tolerances, as well as for slotted holes whose major axis is perpendicular to the stress axis.

$\gamma_{MS} = 1.40$ for oversized holes, as well as for slotted holes whose major axis is parallel to the stress axis.

- at SLS: $\gamma_{MS} = 1.10$ for holes with normal tolerances, as well as for slotted holes whose major axis is perpendicular to the stress axis.

I.10.2.3 Tensile Strength of a Prestressed Bolt

To prevent deboning, the tensile force applied per bolt must be less than the prestressing force, i.e.: $F_{t,Sd} \leq F_p.C_d$

I.10.2.4 Joints Loaded Simultaneously in Shear and Tension

If a slip-resistant joint is subjected to a tensile force F_t concomitant with a shear force F_v , which tends to cause slippage, the slip resistance per bolt must be calculated using the following formula:

$$F_v \leq F_s = k_s \cdot m \cdot \mu \cdot (F_p - 0.8 \cdot F_T) / \gamma_{MS}$$

I.10.2.5 Plate connections subjected to bending moment and shear force.

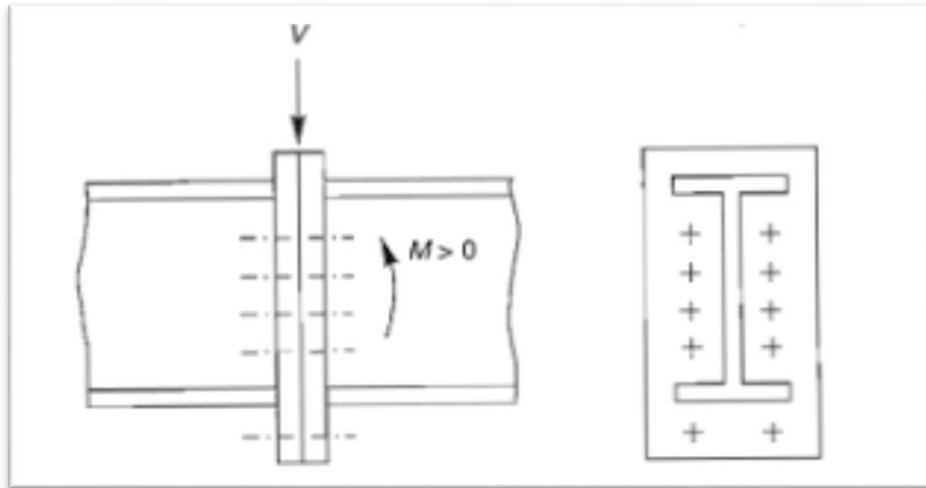


Figure I.12: Plate connections [2]

Connection resistance to shear force V : It must be verified that the shear force V per bolt is such that:

$$V_1 = \frac{V}{n} \leq f_s r_d = K_s \cdot m \cdot u \cdot f_{p c d} / \gamma_{M S}$$

N : Total number of bolts. Connection resistance to bending moment: The connection moment resistance M_R is obtained by summing the products of the tensile forces in the bolt rows located in the tension zone and their respective distances from the center of resistance of the compression zone (i.e., the neutral axis of the compression flange).

$$M_R = N_1 \cdot d_1 + N_2 \cdot d_2 + \dots = \sum N_i \cdot d_i$$

Force N_i in the bolts:

$$\frac{N_1}{d_1} = \frac{N_2}{d_2} = \frac{N_3}{d_3} = \dots$$

and

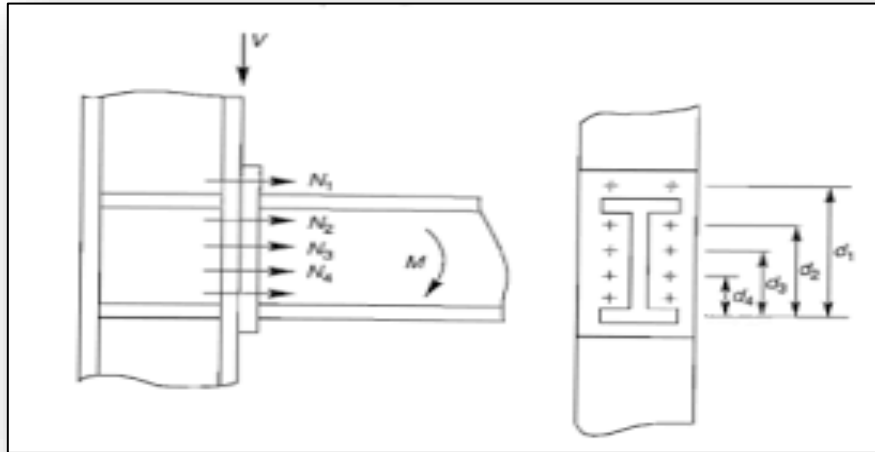
$$M_R = N_1 \cdot d_1 + N_2 \cdot d_2 + N_3 \cdot d_3 + \dots$$

$$= \frac{N_1}{d_1} (d_1^2 + N_2 \cdot d_2 \cdot \frac{d_1}{N_1} + N_3 \cdot d_3 + \frac{N_1}{d_1})$$

$$\text{Hence } N_1 = \frac{M_R \cdot d_1}{\sum d_i^2} \text{ or } N_i = \frac{M_R \cdot d_1}{\sum d_i^2} \leq n \cdot F_p$$

With n : number of bolts per row.

The tension zone is determined by: $X = t_f \sqrt{\frac{b}{t_w}}$



At the same time, the column web strength should be checked: in the tension zone in the compression zone in the shear zone. Column web strength in the tension zone

$$F_t = f_y \cdot t_{wc} \cdot b_{eff} / \gamma M_0$$

With t_{wc} : column web thickness

$b_{eff} = p$ = bolt row center distance. Column web resistance in the compression zone

$$\text{Unstiffened web: } F_c = f_y \cdot t_{wc} (1.25 - 0.5 \times \gamma M_0 \times \sigma_n / f_y) b_{eff} / \gamma M_0$$

Where σ_n : normal compressive stress in the column web due to the compressive force and bending moment

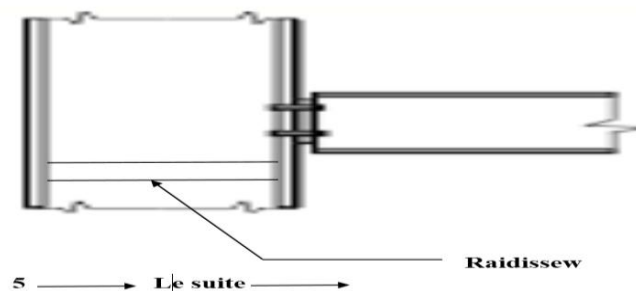
$$b_{eff} = t_{fb} + 2 t_p + 5(t_{fc} + r_c)$$

t_{fb} : beam flange thickness

t_{fc} : column flange thickness

t_p : end plate thickness

r_c : column web/flange connection radius Stiffened web: no verification is necessary provided that the stiffeners have a thickness equal to that of the beam flanges.



I.11 Construction Precautions

Proper assembly using HR bolts requires basic precautions, including:

- ✓ The bolt head must not puncture the assembled parts (hence the use of a washer).
- ✓ The preload force must be applied at its design value (hence the importance of tightening torque and the need for torque or pneumatic wrenches).
- ✓ The coefficient of friction μ must correspond to its design value. This requires surface preparation, by brushing or shot blasting, to remove all traces of rust, scale, grease, etc.
- ✓ $\mu = 0.50$ for Class A surfaces
- ✓ $\mu = 0.40$ for Class B surfaces
- ✓ $\mu = 0.30$ for Class C
- ✓ surfaces $\mu = 0.20$ for Class D surfaces

Class A:

- Surfaces blasted or sandblasted, with removal of all loose rust and without corrosion pits
- Surfaces blasted or sandblasted and metallized by spraying aluminum. Surfaces blasted or sandblasted and metallized by spraying a zinc-based coating, guaranteed to ensure a slip coefficient of at least 0.5

Class B:

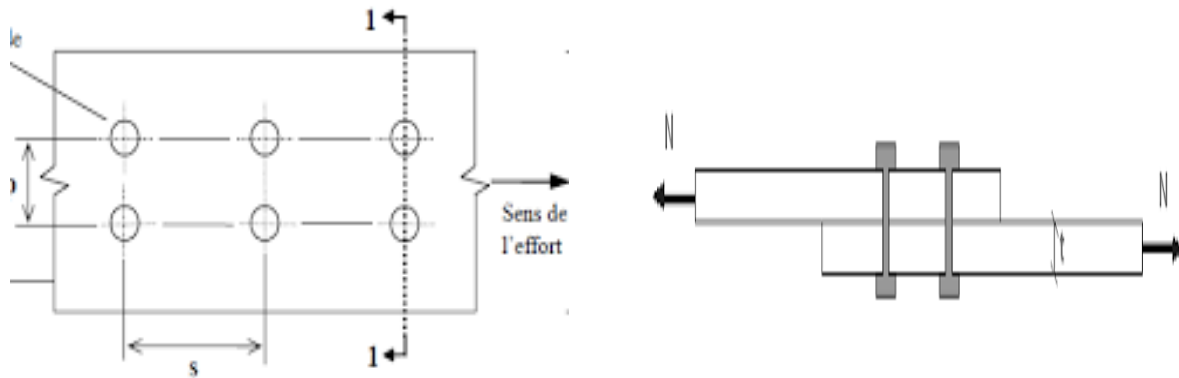
- Surfaces blasted or sandblasted and coated with a layer of alkaline zinc silicate paint 50 to 80 μm thick

Class C:

- Surfaces cleaned by wire brushing or flame cleaning with removal of all loose rust.

Class D: Untreated surfaces.

Exercise 1



Solution

This is a prestressed bolting system acting in shear.

The shear force absorbed by each bolt is:

$$F_{v,sd} = N_{sd}/(n) = 230/5 = 46 \text{ kN.}$$

Calculating the bolt diameter For a shear plane, we have

$$F_{v,sd} \leq F_{s,Rd} = (K_s \cdot m \cdot \mu \cdot F_{pcd}) / (\gamma_{Ms}) ; \mu = 0.5; K_s = 1; \gamma_{Ms} = 1.25; m = 1$$

$$F_{v,sd} \leq (K_s \cdot m \cdot \mu \cdot 0.7 \cdot f_{ub} \cdot A_s) / \gamma_{Ms}$$

$$A_s \geq (1.25 \times 46 \times 10^3) / (0.7 \times 1 \times 1 \times 0.5 \times 800) \rightarrow A_s \geq 205.36 \text{ mm}^2 \text{ We use } \varnothing 20 \text{ bolts}$$

($A_s = 245 \text{ mm}^2$). Diametral pressure verification $F_{v,sd} \leq F_{b,Rd}$

$$F_{b,Rd} = (2.5 \times \alpha \times f_u \times d \times t) / \gamma_{Mb} ; \text{ With } d_o = d + 2 = 22 \text{ mm.}$$

Assembled parts resisting diametral pressure $C = 551.936 \text{ N.m}$

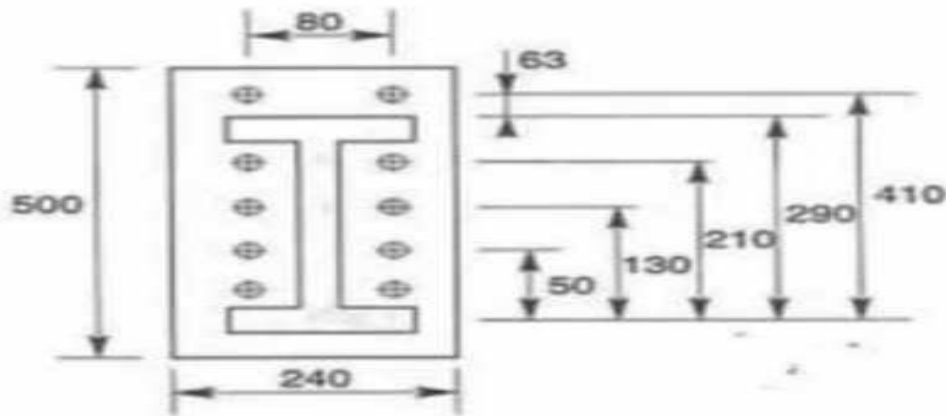
Exercise 2:

Verification of a Column-Beam Connection Given a connection loaded by:

_ A bending moment $M = 320 \text{ kN.m}$

_ A shear force $V = 80 \text{ kN}$ and consisting of 10 HR10.9 bolts. The plate has a thickness of 28 mm and a coefficient of friction $\mu = 0.30$, S235; $r_c = 21 \text{ mm}$.

1) Determine the bolt diameters, given that the beam is an IPE360 and the column is an IPE400.



Solution

1) Determination of forces in the bolts: We only consider the bolts in tension, i.e. the three upper rows of bolts, namely:

$$N_i = \frac{M \cdot d_i}{\sum d_i^2}$$

$$d_1 = 410 \text{ mm}$$

$$d_2 = 290 \text{ mm} \quad \sum d_i^2 = 296300 \text{ mm}^2$$

$$d_3 = 210 \text{ mm}$$

$$N_1 = \frac{M \cdot d^1}{\sum d_i^2} = \frac{320 \times 10^3 \times 410}{296300} = 442,79 \text{ KN.}$$

$$N_2 = \frac{M \cdot d^2}{\sum d_i^2} = \frac{320 \times 10^3 \times 290}{296300} = 313,19 \text{ KN.}$$

$$N_3 = \frac{M \cdot d^3}{\sum d_i^2} = \frac{320 \times 10^3 \times 210}{296300} = 226,79 \text{ KN.}$$

So it must be that: $N_1 \leq n \times F_p$

So: $N_1 \leq n \times 0.7 \times f_{ub} \times A_s$

That is: $A_s \geq (N_1) / (n \times 0.7 \times f_{ub}) \quad A_s \geq 442.79 / (2 \times 0.7 \times 1000 \times 10^{-3})$

$\rightarrow A_s \geq 316.28 \text{ mm}^2. \quad A_s \geq 316.28 \text{ mm}^2.$

According to the table, we have: A bolt with a diameter of $d = 24 \text{ mm}$ ($A_s = 353 \text{ mm}^2$)

Effective moment resistance of the connection

$$MR = \frac{N_1 \cdot \sum d_i^2}{d_1} = \frac{442,79 \times 296300}{410} = 320 \text{ KN. m}$$

$N_1 = 442.79$ (for 2 bolts) for one bolt $N = 221.395$ KN

Resistance of the connection under shear force:

Per bolt $V_1 = v/n \leq F_{srd} = K_s.m.\mu \times F_{pcd}/\gamma_{MS}$

$V = v/n = 80/10 = 8$ KN.

It must be verified that: $V_1 \leq 0.3 \times 1 \left((0.7 \times 1000 \times 353) \right) / 1.20$

$V_1 \leq (0.3 \times 1 \times 247.1) / 1.20$ 8KN = $V_1 < 61.77$ KN (C.V)

Resistance of the column web in tension:

$F_v \leq F_t$ $F_t = f_y.t_{wc} \times b_{eff} / \gamma_{M0}$ $F_t = 235 \times 8.6 \times 80 / 1.1 = 146.98$ KN.

$F_v = M / (h - t_f) = (320 \times [10]^3) / (400 - (13.5)) = 827.94$ KN

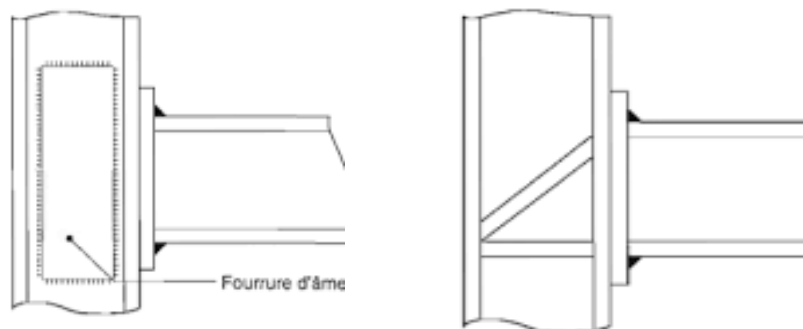
$F_v < F_t$ 827.94KN > 146.98 KN (CNV) Column web compression strength: Unstiffened web ($F_c \leq f_y$) $b_{eff} = 12.7 + (2 \times 28) + 5(13.5 + 21)$ $b_{eff} = 241.2$ mm.

$\sigma_n = V/A + (M.Y)/I \rightarrow \sigma_n = V/A + M/(w_{el,y})$, $A = 84.5 \times 10^2$ $\sigma_n = (80 \times 10^3) / (84.5 \times 10^2) + (320 \times 10^3 \times 10^3) / I$ $\sigma_n = 285$ MPa $\geq f_y = 235$ MPa (C.N.V)

Hence stiffeners with a thickness of 14 mm are required.

Column web shear strength: $V_R = 0.58 \times f_y \times h \times t_w / \gamma_{M0}$ $V_R = 0.58 \times 235 \times 400 \times 8.6 / 1.1 = 426.24$ KN. (C.N.V)

requires the installation of a web furring strip (10 mm thick). Column web strength in the shear zone (unstiffened web) $V_r = 0.58 \times f_y \times h \times t_{wc} / \gamma_{M0}$ If the strength proves insufficient, the web must be stiffened, either with a web furring strip (B) or with diagonal stiffeners (C).



CHAPTER II:

Calculation of post

bases

II.1 Introduction

The primary role of a column base is to transmit all vertical and horizontal loads acting on the building structure to the foundations. This transmission is ensured by a base plate welded to the base of the column, which is assumed to be dimensionally stable and of sufficient thickness (thick) or to include stiffeners.

It is assumed that the plates, subjected to the reactions of the foundations, may bend along the lines tangent to the contour of the posts, such as lines 1-1 and 2-2 in the following figure.

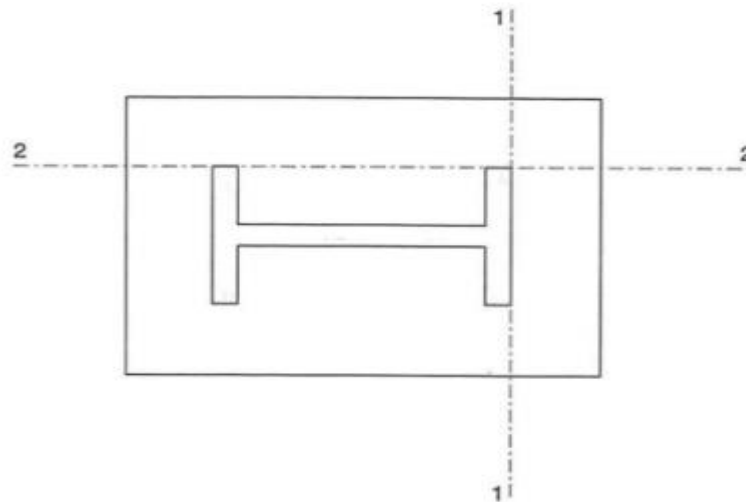


Figure II.1: Base plate [2]

The portions of the plates located outside these lines are then to be calculated as cantilever beams, and it is necessary to verify that the section of plate located at the bending line is capable of resisting the moment of reaction exerted by the foundation mass between this section and the free edge of the base plate. The calculations consist of:

- 1/ Determining the surface area of the base plate, based on the permissible compressive stress σ_b of the concrete in the foundation mass.
- 2/ Determining the thickness of the base plate, based on the bending stress calculated at each pier line.
- 3/ Determining the anchor bolts, based on the tensile forces generated either by a moment at the base (embeddedness) or by wind uplift. The choice of anchor depends on

both the quality of the soil and the type of column-foundation connection. There are two types of joints:

- Articulated or semi-articulated post bases
- Embedded post bases

II.2 Articulated Post Base

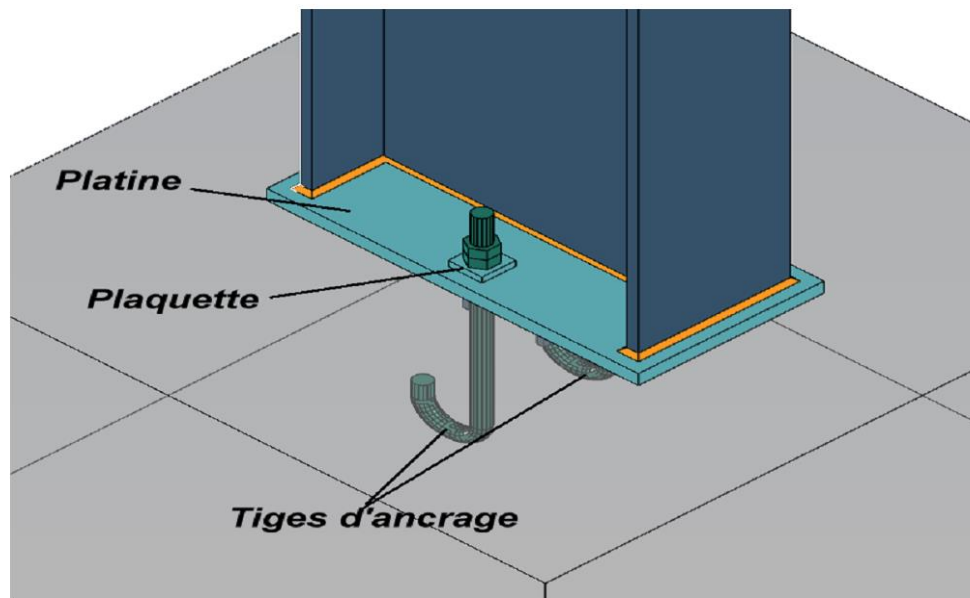


Figure II.2: Articulated Post Base [6]

This type of IPE post base is articulated on a concrete foundation block, shown in gray. The post is welded to a base plate or base plate, shown in light blue. This plate is pierced with two holes located in the IPE shell through which the anchor rods of the fastening system will pass; in this case, curved rods in the shape of a cane. If the holes are not "standard" (do not correspond to the nominal hole diameter d_0), support plates or plates drilled to the nominal diameter will be added between the base plate and the nuts. These plates will be welded to the base plate after precise positioning of the post. Below the base plate and welded to it, there is a spade: a small piece of IPE profile whose role is to absorb most of the shear force that this assembly transmits from the post to the foundation. The base plate has a maximum length of 300 mm; otherwise, the joint cannot be ensured. Therefore, in the case of an IPE with dimensions greater than $h=300$ mm, it is necessary to provide oblique or vertical stiffeners to transfer the forces from the IPE

flanges to the base plate. For hinged or embedded assemblies, the anchor rods can be curved, cane-shaped, straight (smooth or grooved), or straight with anchor plates.

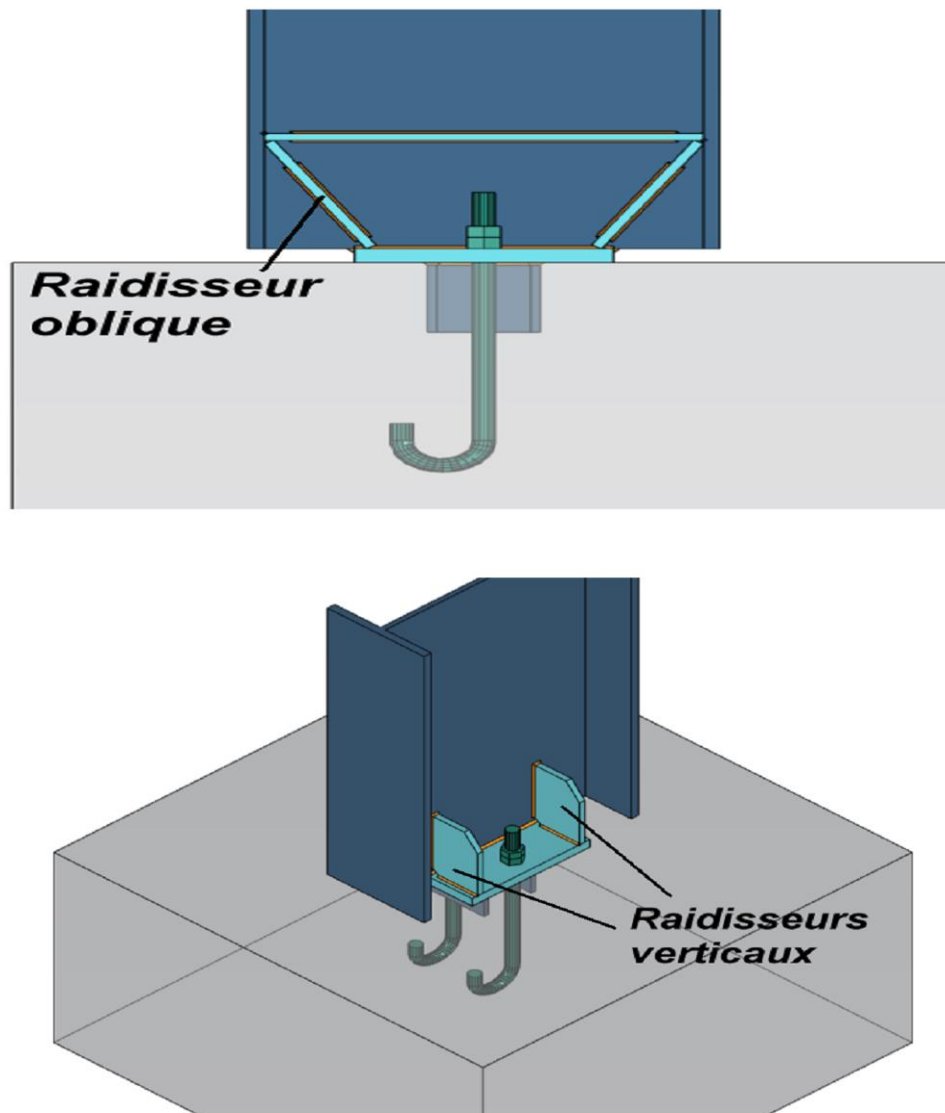


Figure II.2: Articulated column base with oblique and vertical stiffeners [6]

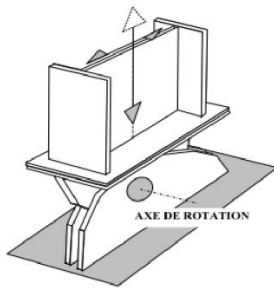
II.2.1 Classification of hinged column bases

II.2.1.1 Rotational rigidity

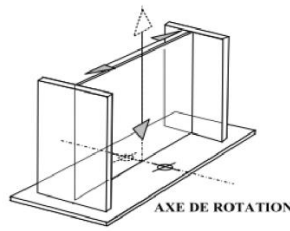
In global analysis, joints are generally considered hinged or fixed. EUROCODE 3 (EN1993-1-8) allows classification of these joints based on their rigidity. A distinction is made between:

- Perfect joints

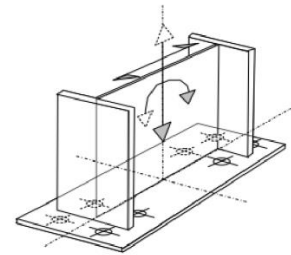
- Nominal joints
- Fixed joints



Articulation parfaite



Articulation nominale



Encastrement

Figure II.3: Classification of column bases [6]

To demonstrate that the column base in question is a hinge, the conventional Lescouarc'h method can be used.

II.2.2 Conventional method of the national annex

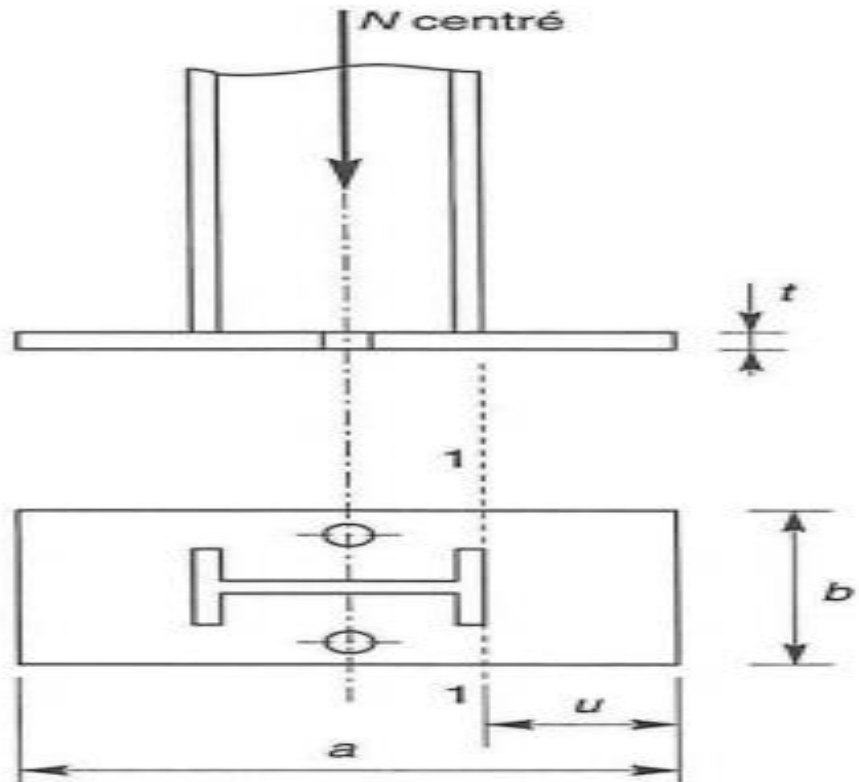
For a rolled column base assembly, with an end plate fixed by two anchor rods on (or very close to) the main axis of the column, to be considered hinged, it must be ensured that:

- Either the height of the plate h_p is less than or equal to 300 mm

Or for $300 \text{ mm} \leq h_p < 600 \text{ mm}$

- $\theta \cdot h_p \leq 3 \text{ mm}$
- $N_{Ed,ser} \cdot \theta \cdot h_c \leq 1,5 \cdot 10^{-6} \text{ N} \cdot \text{mm}$, with h_c the height of the profile with factored loads:
 $N_{Ed,ser}$: the normal force in the column

θ : the local rotation of the column base post, in radians



Surface area of the plate

This is determined by the condition:

$$\sigma = N/(a \cdot b) \leq f_{bu}$$

Thickness of the plate

The force to the right of line 1-1 is:

$$F = \sigma \cdot b \cdot u$$

The corresponding moment has the value:

$$M = F \cdot u/2 = \sigma \cdot b \cdot u^2/2$$

The elastic moment resistance of the plate is:

$$M_{el} = W_{el} \cdot f_y = f_y \cdot b \cdot t^2/6$$

$$\text{Since } W_{el} = b \cdot t^2/6$$

It is therefore necessary to verify that:

$$\sigma \cdot b \cdot u^2/2 \leq f_y \cdot (b \cdot t^2)/6$$

either:

$$t \geq u \sqrt{\frac{3\sigma}{f_y}}$$

i.e.: Conversely, if t is fixed a priori, the problem will be to verify the bending stress σ at the bending line,

either:

$$\sigma \leq \frac{f_y}{3} \left(\frac{t}{u}\right)^2$$

Anchor rods (anchor bolts):

They are straight but generally curved at one end (Figure 3), which allows for greater tensile force to be absorbed and the rod, and therefore the entire frame, to be locked during assembly of the structure. These tensile forces are generated by the uplift wind in general and by moments at the base in the case of embedded posts. The permissible force per anchor, in the case of a dowel with a bracket, set by the CM 66 rules (article 5.123) is:

$$N_a = 0,1 \left(1 + \frac{7 g_c}{1000}\right) \frac{\phi}{\left(1 + \frac{\phi}{d_1}\right)^2} (\ell_1 + 6,4 r + 3,5 \ell_2)$$

g_c : Being the cement dosage of the concrete (kg/m^3). Current values of the other parameters: $r=3\Phi$, $\ell_1=20\Phi$, $\ell_2=2\Phi$;

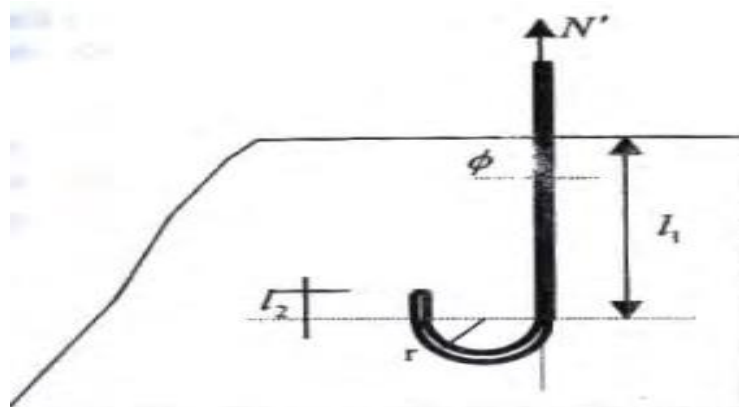


Fig.3 : Goujon d'ancrage

Figure II.4: Anchor stud [2]

II.3 Recessed post base

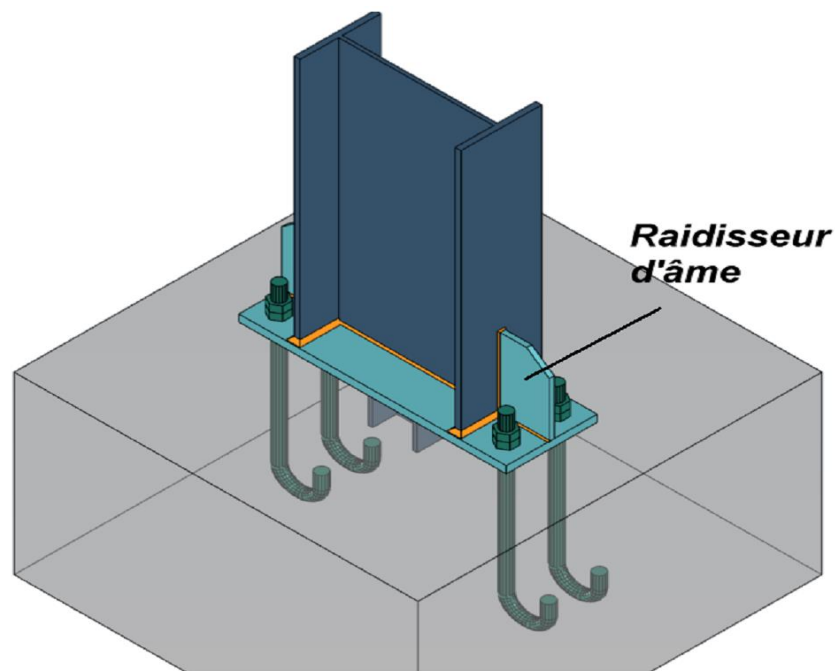


Figure II.5: Embedded column base [6]

In this case, the column is completely fixed to the foundation; no displacement or rotation is allowed between the two elements. The base plate is secured with at least four anchor rods. This is only possible in compact and homogeneous soil. Embedded column bases are necessary when the columns must ensure the vertical stability of the structure and/or when horizontal deformation criteria are imposed on structures containing heavy crane runways and cable car pylons. In this case, the column is loaded at the base by a centered axial force N and a bending moment M , which is equivalent to an eccentric vertical force N of $e = M/N$. The bolts located on the side opposite the force N are subjected to a tensile force, and the concrete on the side of the force N is subjected to a compressive force with a triangular distribution.

- Tensile force acting on the left bolts: $T = A \cdot \sigma_a$.
- Compressive force acting on the concrete under the plate: $C = 0.5 \cdot b \cdot h' \cdot \sigma_b$.
- If n is the steel-concrete equivalence coefficient ($n = E_a / E_b$), we have:

Let's write the force balance: $N + T = C$

and the moment balance: Combining the three previous relationships leads to the following equation, with $n = 15$:

Solving allows us to obtain h' , and subsequently verify σ_a and σ_b :

Exercise 1

Consider an HEB 200 column, hinged at the base (Figure I), subjected to the following loads: Compressive force (under G + Se): $N = 44,000 \text{ daN}$; Wind uplift force (under G + We): $V = 12,000 \text{ daN}$; Concrete containing 350 kg/m^3 of cement; We will:

- 1/ Determine the thickness of the base plate and the diameter of the dowels;
- 2/ Check the compressive stress on the foundation footing

Solution

Determine the thickness of the base plate and the diameter of the dowels. Checking the compressive stress on the foundation footing.

$$\sigma = N/(a.b) \leq f_{bu} \Rightarrow \sigma = 440000/(320 \times 320) = 4.29 < [\sigma_b] = f_{bu} = 8 \text{ MPa}$$

Plate thickness:

$$t \geq u \cdot \sqrt{(3 \cdot \sigma) / f_y} \Rightarrow t \geq 60 \cdot \sqrt{(3 \times 4.29) / 235}$$

$$\Rightarrow t \geq 14.04 \text{ mm or } t = 15 \text{ mm}$$

Stud diameter:

Tensile force per stud:

$$V/2 = 6000 \text{ daN.}$$

Allowable force per stud:

$$N_a = 0,1 \left(1 + \frac{7 \times 350}{1000}\right) \frac{\phi}{\left(1 + \frac{\phi}{250}\right)^2} \cdot (20\phi + 6,4 \times (3\phi) + 3,5 \times (2\phi))$$

$$\text{From which: } \phi^2 - 3\phi - 375 \geq 0 \rightarrow \phi \geq 21 \text{ mm}$$

$$\text{We assume: } \phi = 22 \text{ mm}$$

Exercise 02

Consider the base of an articulated metal column subjected to a weighted compressive force of $20,000 \text{ daN}$ and a horizontal force (wind) of $5,000 \text{ daN}$. Determine the dimensions and thickness of the base plate, given that the column is a HEB160 and the concrete stress is 60 daN/cm^2 , $U = 60 \text{ mm}$, E24.

Solution

Determine the dimensions and thickness of the base plate.

$$\sigma = \frac{N}{a \times b} \leq f_{ub} \rightarrow a \times b \geq \frac{N}{f_{ub}} = \frac{20000}{60}$$

$$\rightarrow a \times b \geq 333,33 \text{ cm}^2.$$

We will use a square base plate $\sigma = \frac{N}{a \times b} = \frac{20000}{25 \times 25} = 32 \text{ daN/cm}^2$

$$\text{Base plate thickness: } t \geq u \sqrt{\frac{3 \times \sigma}{f_y}} = 60 \sqrt{\frac{3 \times 32 \times 10^{-2}}{24}} \rightarrow t \geq 12 \text{ mm},$$

so we take $t = 12 \text{ mm}$

Exercise 03

Consider a HEB 200 metal column hinged at the base and subjected to the following loads: Axial compressive force under wind load $N = 40,000 \text{ daN}$ Wind uplift force $V = 12,000 \text{ daN}$. The foundation concrete is mixed at 350 kg/m^3 . $f_{ub} = 80 \text{ daN/cm}^2$, E24.

1/ Determine the thickness of the base plate.

2/ Determine the diameter of the dowels.

Solution

Verification of the compressive stress on the foundation footing:

$$\sigma = N/(a \times b) = 40000/(30 \times 30) = 44.44 < f_{ub} = [\sigma_b] = 80 \text{ daN/cm}^2$$

$$\text{Plate thickness: } t \geq u \sqrt{\frac{3 \cdot \sigma}{f_y}} \rightarrow t \geq 5 \sqrt{\frac{3 \times 44,44}{2400}}$$

$$\rightarrow t \geq 1,178 \text{ cm} \rightarrow t \geq 11,78 \text{ mm}$$

Therefore, we assume $t = 12 \text{ mm}$

Dowel diameter:

Tensile force per dowel:

$$V/2 = 6000 \text{ daN}$$

Allowable force per dowel:

$$N_a = 0,1 \left(1 + \frac{7 \times g_c}{1000}\right) \frac{\phi}{\left(1 + \frac{\phi}{d_1}\right)^2} \cdot (L^1 + 6,4 \times r + 3,5 \times L^2) > \frac{V}{2}$$

$$N_a = 0,1 \left(1 + \frac{7 \times 350}{1000}\right) \frac{\phi}{\left(1 + \frac{\phi}{250}\right)^2} (20\phi + 6,4 \times 3\phi + 7\phi) \geq 6000$$

$$\phi^2 - 3\phi - 375 \geq 0 \rightarrow \phi \geq 21 \text{ mm} \rightarrow \phi = 22 \text{ mm}$$

Exercise 4

Consider an HEB 200 column, embedded at the base, subjected to a factored centered axial force of $25,000 \text{ daN}$ at a factored moment of $7,500 \text{ daNm}$ (Figure II). Dowel diameter: $\phi = 24 \text{ mm}$ (S.235 steel) · Concrete: $f_{ub} = 8 \text{ MPa}$. Check the tensile stresses

in the dowels and the compressive stresses in the concrete, and determine the thickness of the plate (S.235 steel).

Solution

$e = M/N = 7,500/25,000 = 30 \text{ cm}$. $A_s = 3.53 \times 2 = 7.06 \text{ cm}^2$. $L = 50 \text{ cm}$. $h = 45 \text{ cm}$. $b = 50 \text{ cm}$.

Concrete Compressive Stress Verification: The third-degree equation in h' is then written:

$$\hat{h}^3 + 3 \times 5 \hat{h}^2 + 90 \times 7,06 \cdot \hat{h} - 90 \times 7,06 \times h = 0$$

Therefore: $\hat{h} = 20.7 \text{ cm}$

The compressive stress in the concrete is then:

- $\sigma_b = \frac{2 \times 25000 \times 50}{50 \times 20,7 \times (45 - 6,9)} = 6,33 \text{ MPa} < f_{ub} = 8 \text{ MPa}$

Dowel Tensile Verification:

- $\sigma_a = \frac{250000}{760} \times \frac{(500 - 450 + \frac{207}{3})}{(450 - \frac{207}{3})} = 110,60 \text{ MPa} < f_y = 235 \text{ MPa}$

Exercise 5

Solution

IPE300 Column Permanent Loads G:

$$G = G_c + G_p + G_f + G_{\text{column}}$$

$$G = 17 + 6.4 + 10.20 + 50 \quad G = 83.6 \text{ kg/m}^2.$$

$$\text{Snow load: } N_e = 68 \text{ kg/m}^2; \quad N_n = 40 \text{ kg/m}^2$$

$$\text{Wind load: } V_n = 75 \text{ kg/m}^2; \quad V_e: 131.25 \text{ kg/m}^2$$

$$\text{Column surface area: } 5 \times 8 = 40 \text{ m}^2$$

$$\text{Combinations: } N_1 = G + 5/3 N_e = 83.6 + 5/3 \times 68 = 196.93 \times 5 \times 8 = 7877.33 \text{ kg}$$

$$V = G - V_e \quad 83.6 - 131.25 = -47.65 \text{ kg/m}^2 \times 8 \times 5 = -1906 \text{ kg}$$

$$N_2 = 1.35 g + 1.5 n_n = 1.35 \times 83.6 \times 40 + 1.5 \times 40 \times 40$$

$$N_2 = 6914.4 \text{ kg},$$

$$\text{so } N_{\text{max}} = N_1 = 7877.33 \text{ kg}$$

Verification of the compressive stress on the footing $\sigma = N/(a \times b) = (N_1)/(a \times b) = 7877.33/(40 \times 25) = 7.87 \text{Kg/cm}^2 = 78.7 \text{daN/cm}^2 < [\sigma_b] = F_{bu} = 80 \text{ daN/cm}^2 \text{ (CV)}$

Plate thickness: $t \geq \sqrt[3]{(3 \times \sigma) / V_e} = 50 \sqrt[3]{(3 \times 0.787) / 235} \rightarrow t \geq 5.01 \text{mm}$

The design thickness is small for practical reasons.

For a thickness of $t = 12 \text{mm}$, we adopt Stud diameter: Tensile force per stud:

$$V/2 = (1906 / 2) = 953 \text{Kg}$$

Permissible force per stud $N_a = 0.1 (1 + (7 \times g_c) / 1000) (\phi / (1 + \phi / 250))^2 (L^3 + 6.4 \times h + 3.5 \cdot L^2)$
 $N_a = 0.1 (1 + (7 \times 350) / 1000) (\phi / (1 + \phi / 250))^2 (20\phi + 6.4 \times 3\phi + 3.5 \times 2\phi) \geq V/2 = 953 \text{ N}$
 $N = 0.1 (1 + 0.0000245) (\phi / ((1 + \phi^2 / 62500 + 2 \times \phi \times 0.008))) (46.2\phi) N = 0.1 + 0.00000245 [(46.2\phi^2) / (1 + 0.000016\phi^2 + 0.008\phi)] N = 0.1 + [46.2\phi^2 + 2887500 + 5775\phi] \geq 60000$
 $N = 0.1 + 0.00011319 \cdot 0.345 ((46.2\phi^2) / (1 + \phi / 250))^2 \geq 953$
 $15.939\phi^2 \geq 953 (1 + \phi / 250)^2$
 $15.939\phi^2 \geq 953 (1 + \phi^2 / 250 + 0.008\phi)$
 $15.939\phi^2 \geq 953 + 0.015248\phi^2 + 7.624\phi$
 $15.939\phi^2 - 0.015248\phi^2 - 7.624\phi - 953 \geq 0$
 $15.923752\phi^2 - 7.624\phi - 953 \geq 0$
 $\Delta = b^2 - 4ac = (7.624)^2 - (4 \times 15.923752 \times -953)$
 $\phi \geq (7.624 + 246.49) / (2 \times 15.923752)$
 $\phi \geq 7.97 \text{mm}$ So we take $\phi = 10 \text{mm}$
 $L_1 = 20 \times \phi = 200 \text{mm}$; $L_2 = 20 \text{mm}$; $r = 30 \text{mm}$

**CHAPTER III:
CALCULATION OF
MIXED FLOORS
WITH
COLLABORATING
SLABS**

III.1 Introduction

Composite or composite flooring is the ideal construction solution for all construction sites requiring advanced technical and mechanical performance and requiring rapid, guaranteed implementation. Thanks to its superior characteristics, this flooring is suitable for all types of construction (industrial, commercial, sports, and residential). It also offers significant economic advantages, especially when considered from the very beginning of the project. Its use involves a reduction in the average thickness of the slabs, which translates into a reduction in the structural load-bearing elements (columns, beams, and foundations). In addition to improving the planning of the various implementation phases, the adoption of this technology meets certain essential requirements in modern buildings, such as the incorporation of conduits necessary for the provision of office services and the use of suspended ceilings. Composite (steel/concrete) floors are the most common type used in the construction of office building floors, warehouses, mezzanines, etc. Two scenarios are possible:

The RC slab is non-stressful: it is not connected to the supporting steel frame, and therefore does not contribute to the overall inertia of the floor. In this case, the slab constitutes a permanent load for the supporting frame, which is detrimental due to its heavy weight;

The RC slab is stressful: it contributes to the overall inertia of the floor, which requires it to be perfectly connected to the supporting structure. To achieve this, connecting devices (connectors) must be provided at the steel/concrete interface, which secure the slab and beams together and prevent them from sliding together.

III.2 Composite Flooring

The relevance of composite floors lies in the technology used to strengthen the bond between the worked steel sheet and the concrete. This technology is also known as composite flooring due to the "collaboration" between the two materials forming the floor, which aims to withstand the stresses generated by the loads. The mechanical adhesion of the two components is achieved through the notches machined on the inclined sides of the galvanized steel profile. Chemical adhesion alone is not sufficient. to ensure an effective connection that truly makes the composite floor work as a composite structure.

III.3 Functions and Benefits of the Composite

Composite Floor After installing and securing the floor, it fulfills the following functions:

- It serves as a working platform during construction while also providing protection and safety against falling objects.
- It replaces the traditional permanent wooden formwork used as a support for the concrete pour.
- It contributes to the stabilization of the frame in the case of a metal structure, thus avoiding the need for horizontal bracing.
- It supports loads during the concreting phase in certain span and thickness situations. Above a certain slenderness ratio, it is necessary to shore up the profile before pouring the concrete. It is the responsibility of the person responsible for the calculation to ensure that the necessary intermediate supports are installed when recommended by the live load table.
- It allows free movement of operators on the floors thanks to the absence of the shoring equipment required for conventional formwork.
- It performs a so-called "collaborative" action by closely coupling with the concrete through notches and protruding elements. The metal profile thus makes it possible to completely or partially replace the tensile reinforcement usually placed in the slab. The use of additional round tensile bars is not taken into account in the tables in this manual. However, it is up to the person responsible for the calculation to decide whether or not to include these bars in order to increase the slab's fire resistance.
- The use of connectors allows the composite floor to function as a composite beam. This results in a significant reduction in the floor thickness, consequently reducing the weight of all the metal profiles supporting the slab, the structure, and the building's foundations in general. The economic advantage is therefore obvious, both in terms of materials and execution time.
- The longitudinal ribs of the profiled sheet metal accommodate the building's installations and pipes.
- This construction system offers significant cost savings combined with time savings.

III.4 Method

There are two main types of composite floors because there are two building materials that are highly resistant to tensile forces: steel and wood. As you can see in the article on concrete strength, flexural elements (such as beams and floors) are subjected to tensile forces at the bottom, which is why the materials (steel and wood) that absorb the tension will be located at the bottom of the floor.

III.4.1 Composite Steel Floors

There are three main techniques for composite steel floors:

- The first involves connecting metal beams (typically IPE) to a concrete slab, which will then act in compression. This connection is mainly made using dowels, which prevent one material from sliding over the other and allow for proper load transmission.
- The second technique, certainly one of the simplest in terms of floors, is the steel deck. Corrugated steel trays are laid across the entire surface of the future floor and form the underside. Reinforcements are then placed on top and concrete or mortar is poured to form a smooth slab after troweling. These sheets, which do not exceed 30 kg, are very easy to install and the installation of such a floor takes half as long as that of a solid slab floor. The sheets are easily adaptable and cut to suit the standard and dimensions of the floor you wish to create. A layer of thermal and sound insulation can be easily placed under the steel tray, making this floor optimal.

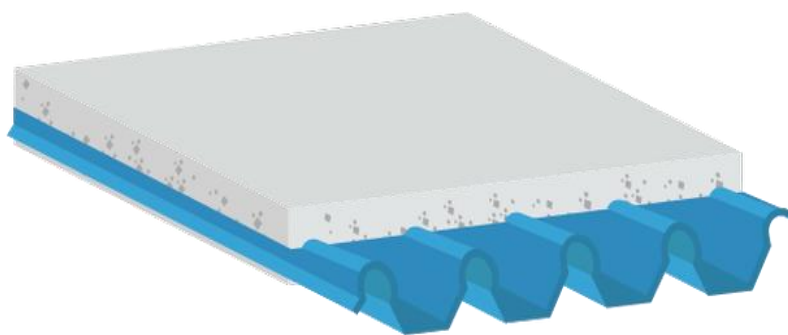


Figure III.1: Collaborative floor [6]

III.5 Components and Installation

III.5.1 Composite Floor Components

The composite floor consists of:

- A corrugated steel plate profile (composite plate), the identification of which allows for a plate unified with the concrete.
- An anti-cracking mesh reinforcement positioned in the same manner as for solid slab floors. Its main purpose is to prevent cracking caused by shrinkage and temperature.
- A negative reinforcement placed on the upper portion of the slab and centered in the valleys. Its purpose is to absorb the tensile forces generated on the supports of the continuous slabs.
- A fire reinforcement, placed in the same manner as the negative reinforcement but on the lower portion of the valley. This reinforcement is also called positive reinforcement. It serves to resist the forces caused by fire in compliance with the standards.
- Concrete poured directly onto the composite floor.

III.5.2 Installing the Composite Floor

The groups of panels must be placed on the main beams of the structure. Once they are installed, each panel must be positioned, paying attention to the layout plans and checking for overlaps, alignment, voids, etc. The most common option is to use an INCO 70.4 Composite profile. It is fixed directly on the head of the beams, and it will rest on them.

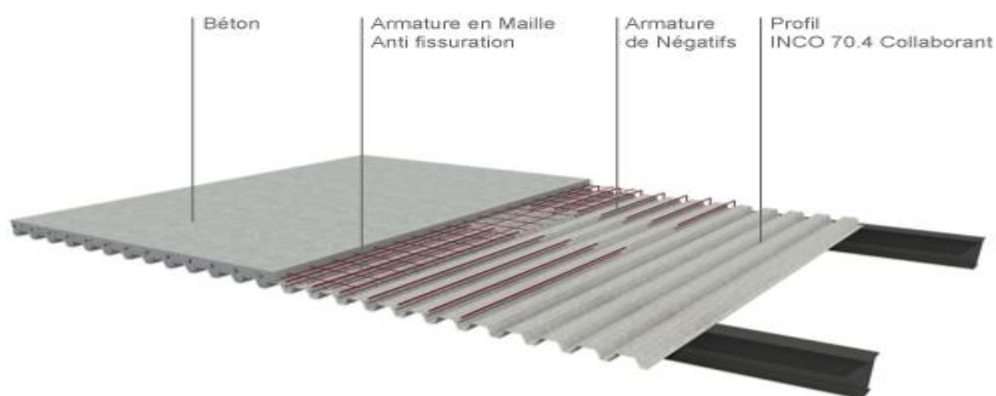


Figure III.2: Composite Floor [7]

Another option, in case the surface area of the composite floor and beam assembly needs to be reduced, is to place the angular supports on the beam webs. In addition, sufficient space must be left for the negative reinforcement and the anti-cracking mesh reinforcement to pass over the beam next to the compression layer, while ensuring the reinforcement overlaps.

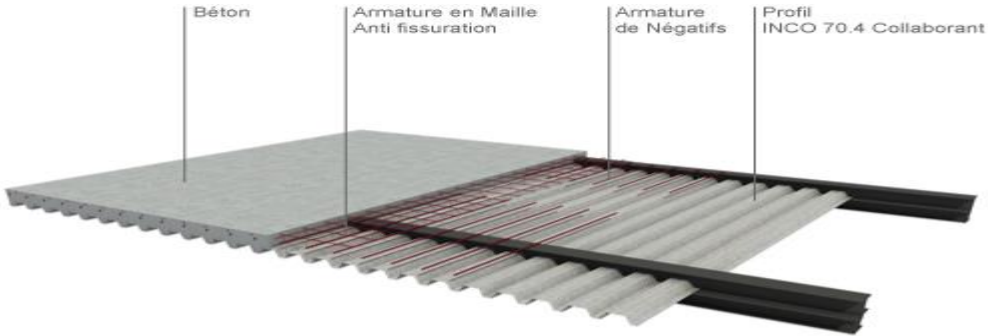
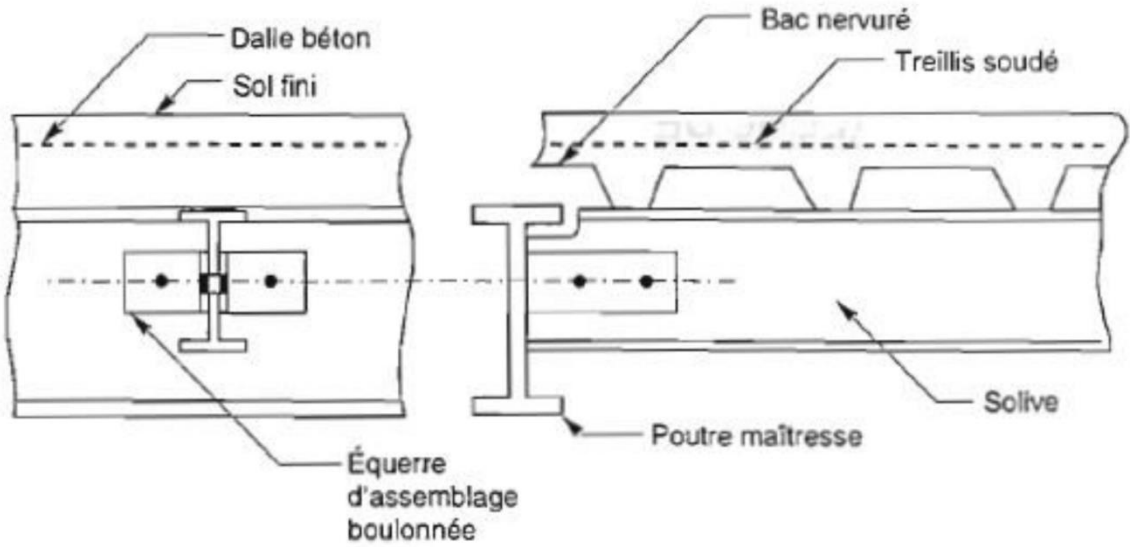


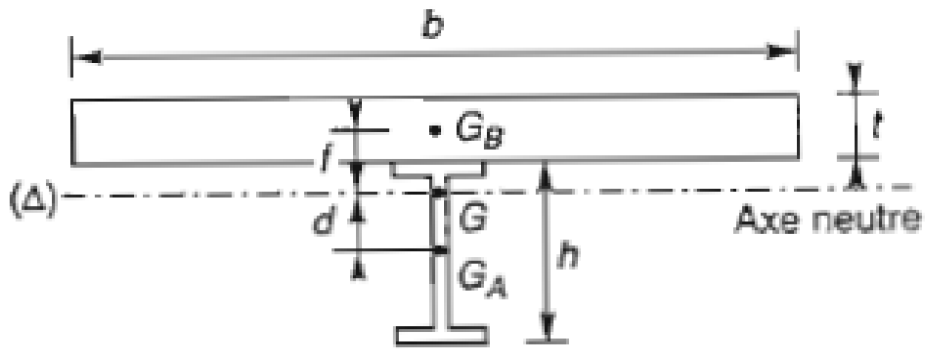
Figure III.3: Composite beam floor [7]

Since composite beam floors are the most economical and technically sound solution, we will now develop the calculation method for this type of floor.



III.6 Calculation of a composite floor with a composite slab

III.6.1. Inertia of the beam/slab assembly



$$S = A + B/n \text{ with } B = bt$$

n: steel/concrete equivalence coefficient

The position of the neutral axis (Δ) of the composite section relative to G_A , the center of gravity of the steel beam, is d and is obtained by writing the equality of the static moments with respect to (Δ):

Beam: $\mu A = A \cdot d$

Slab: $\mu B = B/n \cdot f$

Let

$$A \cdot d = B/n \cdot f$$

Now: $f + d = t + h/2$

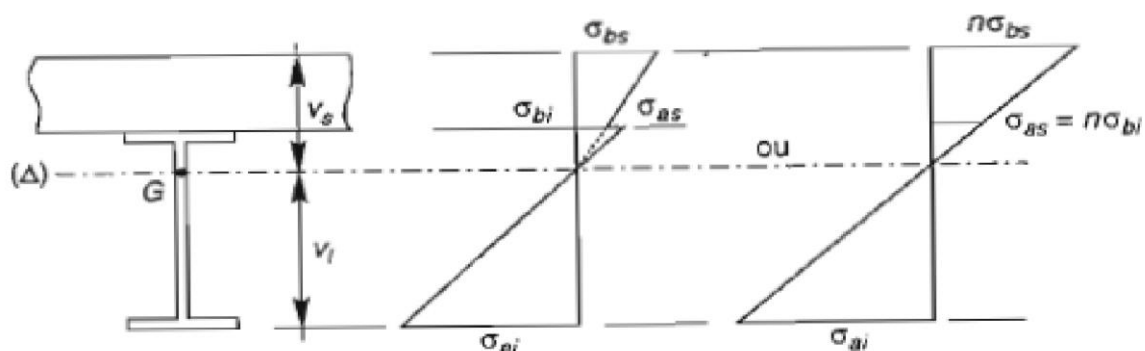
Hence:

$$d = bt/n \times t + h/2S$$

The moment of inertia of the composite section with respect to the neutral axis (Δ) is: I_A and I_B being the natural inertias of sections A and B Let:

$$I = I_A + Ad^2 + \frac{bt^3}{12n} + \frac{bt}{n} \left(\frac{t + h}{2} - d \right)^2$$

III.6.2 Simple bending stresses



M being the maximum bending moment in the composite section, of inertia I, the various extreme stresses are:

Stresses in the steel beam:

Tension: $\sigma_{ai} = M/I \times v_i$

Compression: $\sigma_{as} = M/I(v_s - t)$

Compression (upper fiber) $\sigma_{bs} = M/nI \times v_s$

Compression (lower fiber) $\sigma_{bi} = M/nI \times (v_s - t)$

With: $v_i = h/2 + d$

and $v_s = h/2 + t - d$

III.6.3 Additional Stresses

Due to Concrete Shrinkage After the slab is poured, the concrete, as it hardens, should shrink (shortening ϵ). However, since the slab is secured to the steel beams, this shrinkage is counteracted by the steel, which opposes the shortening of the slab, at the steel/concrete interface.

The shrinkage effect can also be compounded by the effect of the lowering temperature (thermal gradient). These effects cause:

- A shortening ϵ_a of the steel beam
- An elongation ϵ_b of the concrete slab (relative to its equilibrium position, because the concrete cannot freely shrink, and in fact stretches, which is equivalent to an elongation), and we have: $\epsilon = \epsilon_a + \epsilon_b$

Setting: $K = M/I$, the stresses are written:

- $\sigma'_a = K y_1 = E_a \epsilon_a$
- $\sigma_a = K (h - y_1)$
- $\sigma'_{b1} = E_b \epsilon_b = \frac{E_a}{n} (\epsilon - \epsilon_a) = \frac{1}{n} (E_a \epsilon - K y_1)$
- $\sigma'_{b2} = \sigma'_{b1} - K (y_2 - y_1) = \frac{1}{n} (E_a \epsilon - K y_2)$

Let's write the equilibrium of the system:

$\Sigma F = 0$ et $\Sigma M/O = 0$ soit:

- Tensile force in the concrete (at the Σ axis):

$$F_B = \frac{B}{n} \frac{n\sigma'_{b1} + n\sigma'_{b2}}{2}$$

$$F_B = \frac{B}{n} \left(E_a \varepsilon - K \frac{y_1 + y_2}{2} \right)$$

$$F_B = \frac{B}{n} (E_a \varepsilon - K[\alpha + \beta])$$

III.6.4 Compressive force in the steel (at the Σ axis):

$$F_A = \int \sigma' a. ds = K \mu A$$

The static moment μA of the steel section A with respect to 0 is: $\mu A = A. \alpha$, d'où $F_A = K. A. \alpha$ By setting $F_B = F_A$, we obtain:

$$K A \alpha = B/n [E A \varepsilon - K(\alpha + \beta)] \text{ Moment due to FB in the concrete:}$$

$$M B/0 = F_B(\alpha + \beta) = K A \alpha (\alpha + \beta) \text{ Moment due to FA in the beam:}$$

$$M A/0 = \int y \sigma' a. dS = K I \text{ With: } I = I_A + A \alpha^2 \text{ Let } M_B = M_A, \text{ we obtain:}$$

$$K A \alpha (\alpha + \beta) = K (I_A + A \alpha^2) \text{ From which: } \alpha = I_A / A \beta$$

By subtracting this value of α from the previous equation, we obtain the value of K, which allows us to calculate the values of different constraints:

$$K = \frac{B E_a \varepsilon \beta A}{n I_A A + B I_A + B A \beta^2}$$

III.6.5 Deflections

Regulatory limits are:

- to 1/400 of the span, for floors supporting walls, partitions, or glazing.
- to 1/250 of the span, for standard floors

III.6.6 Connectors

In the absence of a connection, the two elements act separately with two neutral axes. The strengths and stiffnesses are combined: it is the steel beam that acts primarily. With composite behavior, the concrete, being entirely compressed, contributes fully to the strength and stiffness of the assembly. Thus, to obtain an optimal composite floor, it is advisable to connect the two materials to have not two, but a single elastic neutral axis, and thus benefit from a stable construction with a compressed concrete slab and steel beams subjected to tension.

III.6.6.1 Connection Design

Connectors are elements that provide the connection between the concrete slab and the joist. They must be able to resist slab uplift and can also prevent slippage between the two elements being joined.

III.6.6.2 Connector Shear Resistance

Connection design is based on the assumption of a complete connection. The design shear resistance of a connector is given by:

$Prd = \min (Prd1, Prd2)$ Where:

Prd1: Shear resistance of the dowel itself.

Prd2: Shear resistance of the dowel cover.

f_u : The specified ultimate tensile strength of the stud steel

γ_v : Partial safety factor equal to 1.25

a : Correction factor \Rightarrow h : Stud height.

d : Stud diameter.

E_{cm} : The average value of the secant modulus of the concrete.

($E_{cm} = 30.5 \text{ kN/mm}^2$).

III.6.6.3 Beam Connection Design

Calculation of the longitudinal shear force:

$$V_{lf} = \min \left(A_a \cdot \frac{f_y}{\gamma_a}; b_{eff} \cdot 0,85 \cdot \frac{h_c \cdot f_{ck}}{\gamma_a} \right)$$

Number of studs $N_f = V_{lf} / Prd$

Exercise 01

Calculate the bending stresses of a composite floor with the following characteristics:

- Storage load: 15 kN/m²
- RC slab, cast on steel decks, with an average thickness of $t=8 \text{ cm}$
- Joist spacing: 1.50 m
- Allowable material stresses:
- For steel: E28

- For concrete: $f_{c28}=25$ MPa
- Steel/concrete equivalence coefficient: $n=15$ Solution

Solution

Calculation of joists:

$$S = A + B$$

$$S = 28,5 \times 10^2 + \frac{1500 \times 80}{15} = 10850 \text{ mm}^2 = S = 108,5 \text{ cm}^2$$

$$d = \frac{1500 \times 80}{15} \times \frac{80 + 200}{2 \times 10850} = 103,2 \text{ mm} = 10,32 \text{ cm} = d$$

$$V_i = 100 + 103,2 = 203,2 \text{ mm} = 20,32 \text{ cm} = v_i$$

$$V_s = 100 + 80 - 103,2 = 76,8 \text{ mm} = 7,68 \text{ cm} = V_s$$

$$I = 6488,37 \text{ cm}^4$$

$$M = 1,35 \text{ MG} + 1,5 \cdot \text{MQ}$$

$$G = G_{\text{joist}} + G_d = 22,4 \text{ Kg} + 25 \times 1,5 \times 0,08 = 0,224 + 3 = 3,224 \text{ KN}$$

$$G = 3,224 \text{ KN/m}$$

$$P = 1,35G + 1,5\emptyset$$

$$P = 1,35 \times 3,22 + 1,5 (15 \times 1,5) \rightarrow P = 38,09 \rightarrow M = \frac{PL^2}{8} = 171,40 \text{ KN.m}$$

Bending stresses

Stresses in the steel beam

$$\sigma_{ti} = \frac{M}{I} \cdot V_i = \frac{171,40 \times 10^6}{6488,37 \times 10^4} \times 203,2 = +536,78 \text{ MPa} = \sigma_{ti}$$

$$\sigma_{cs} = \frac{171,40 \times 10^6}{6488,37 \times 10^4} (76,8 - 80) = -8,45 \text{ MPa}$$

Constraints in the concrete slab

$$\sigma_{ds} = \frac{M}{n \cdot I} \cdot V_s = \frac{171,40 \times 10^6}{15 \times 6488,37 \times 10^4} \times 76,8 = +13,52 \text{ MPa} = \sigma_{ds}$$

$$\sigma_{di} = \frac{171,40}{15 \times 6488,37 \times 10^4} (76,8 - 80) = -0,563 \text{ MPa}$$

Exercise 02

Calculate the bending stresses of a composite floor with the following characteristics:

- Storage load: 20 kN/m^2
- RC slab, cast on steel decks, with an average thickness of $t=8 \text{ cm}$

- Joist spacing: 2 m

- Allowable material stresses: For steel: E30

For concrete: $f_{c28}=25$ MPa

- Steel/concrete equivalence coefficient: $n=15$ $L=5$ m, IPE220 joist

Solution

Joist calculation

$$S = 33,4 + \frac{200 \times 8}{15} = 140,06 \text{ cm}^2$$

$$D = \frac{200 \times 8}{15} \cdot \frac{8+22}{2 \times 140,06} = 11,42 \text{ cm}$$

$$V_i = \frac{h}{2} + d = \frac{22}{2} + 11,42 = 22,42 \text{ cm}$$

$$V_s = \frac{h}{2} + t - d = \frac{22}{2} + 8 - 11,42 = 7,58 \text{ cm}$$

The internals of sections A and B (Beam + slab).

$$I = 2772 + 33,4 \times (11,42)^2 + \frac{200 \times (8)^3}{12 \times 15} + \frac{200 \times 8}{15} \left(\frac{8+22}{2} - 11,42 \right)^2$$

$$I = 9063,87 \times 10^4 \text{ mm}^4$$

$$M = 1,35 \text{ MG} + 1,5M\emptyset$$

$$G = G \text{ solive} + Gd = 0,262 \text{ KN/m} + 25 \times 2 \times 0,08 = 4,262 \text{ KN/m}$$

$$P = 1,35 \times 4,262 + 1,5 (20 \times 2) = 65,75 \text{ KN/m} \rightarrow M = \frac{P \cdot L^2}{8} = 205,48 \text{ KN.m}$$

Bending stresses:

Stress in the steel beam

$$\sigma_{pi} = \frac{M}{I} \cdot V_i = \frac{205,48 \times 10^3 \times 10^3}{9063,87 \times 10^4} \times 224,2 = +508,26 \text{ MPa} = \sigma_{ti}$$

$$\sigma_{ps} = \frac{M}{I} (V_s - t) = \frac{205,48 \times 10^6}{9063,87 \times 10^4} \times (75,8 - 80) = -9,52 \text{ MPa}$$

Concrete slab stress:

$$\sigma_{ds} : \frac{M}{h \times I} \cdot V_s = \frac{205,48 \times 10^6}{15 \times 9063,87 \times 10^4} \times 75,8 = +11,45 \text{ MPa}$$

$$\sigma_{di} = \frac{205,48 \times 10^6}{15 \times 9063,87 \times 10^4} (75,8 - 80) = -0,635 \text{ MPa}$$

Exercise 3

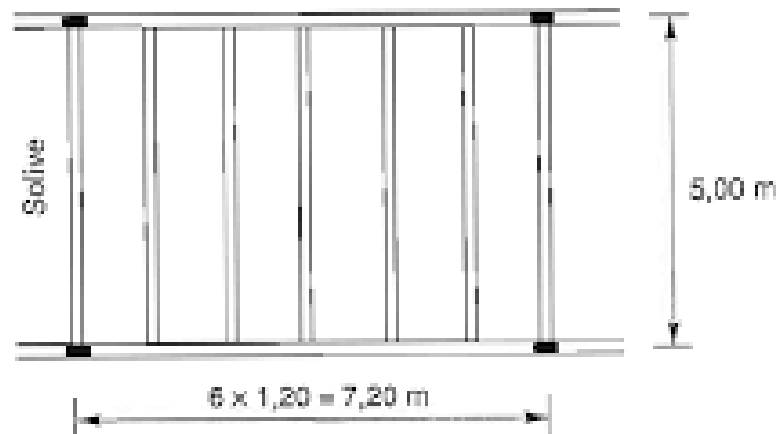
Calculate a warehouse floor with the following characteristics:

- Frame: 7.20m \times 5.00m

- Storage load: 10kN/m²

- RC slab, cast on steel decks, with an average thickness of $t = 8$ cm

- Joist spacing: 1.20m
- Allowable material stresses: For steel: $f_y = 235$ MPa and $f_y = 0.58$. For concrete: $f_{c28} = 25$ MPa
- Steel/concrete equivalence coefficient: $n = 15$
- Concrete shrinkage coefficient: $\epsilon = 2 \times 10^{-4}$



CHAPTER IV:

Design of industrial hall-type buildings

IV.1 Introduction

The load-bearing system of a metal-framed hall consists of an assembly of elements in the form of profiled bars or flat products. The primary function of this framework is to support the loads and forces acting on the hall and transmit them to the foundations. It must then allow for the attachment of the envelope elements (roof, facade, and interior partitions). Due to the volume they delimit, load-bearing structures constitute spatial systems whose behavior under load is three-dimensional. Industrial buildings are found in exhibition halls, railway stations, airports, and sports stadiums, etc. Structure of industrial buildings: They are constructed with a steel frame and primarily use beams and merchant bars.

- Steel structures are of three types:
 - Portal structures
 - Lattice structures
 - Three-dimensional structures made from hollow round, square, and rectangular tubes.

IV.2 Type of structure

A simple-shaped hall is considered a box (Figure VI.1) formed of six flat load-bearing surfaces. The frame and envelope of the hall constitute the lid and the four walls of the box, the bottom being represented by the foundation soil or infrastructure. The supporting structure of the box is broken down into three spatial directions into flat substructures: the transverse frames and gables, the roof and the long-span facades. To ensure the stability of this box, it is necessary that each of the three directions be rigid in its plane.

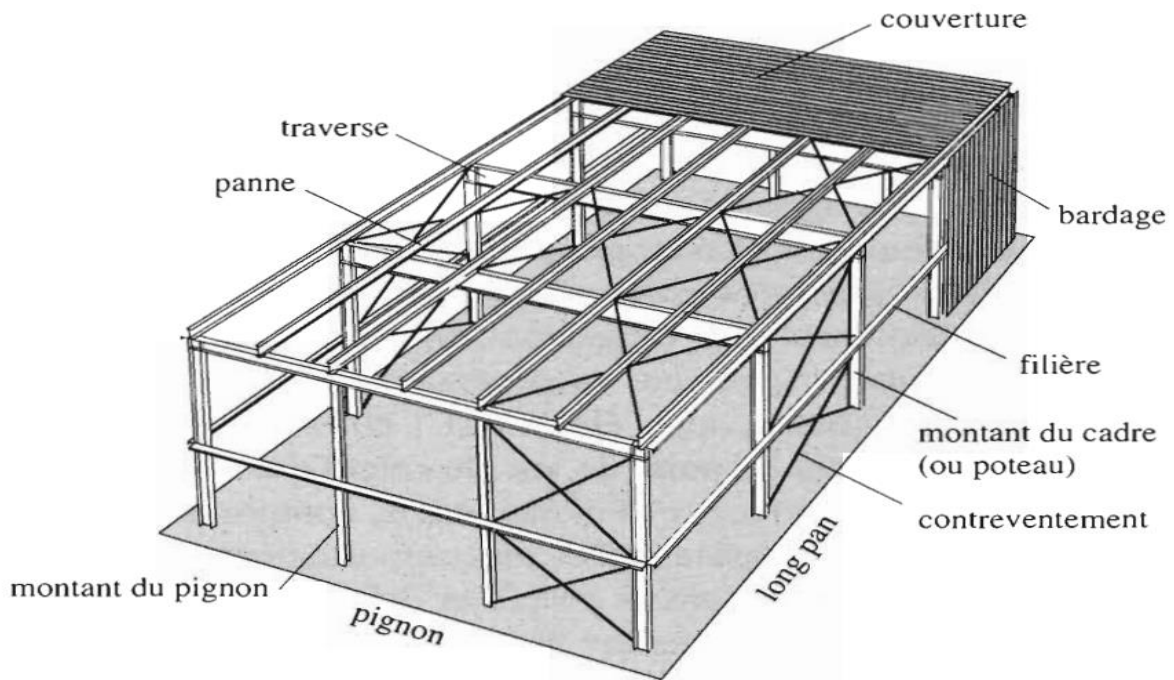


Figure IV.1: Example of a simple hall [1]

The different types of supporting system that can be designed for a hall in the shape of a rectangular parallelepiped based on the basic hall.

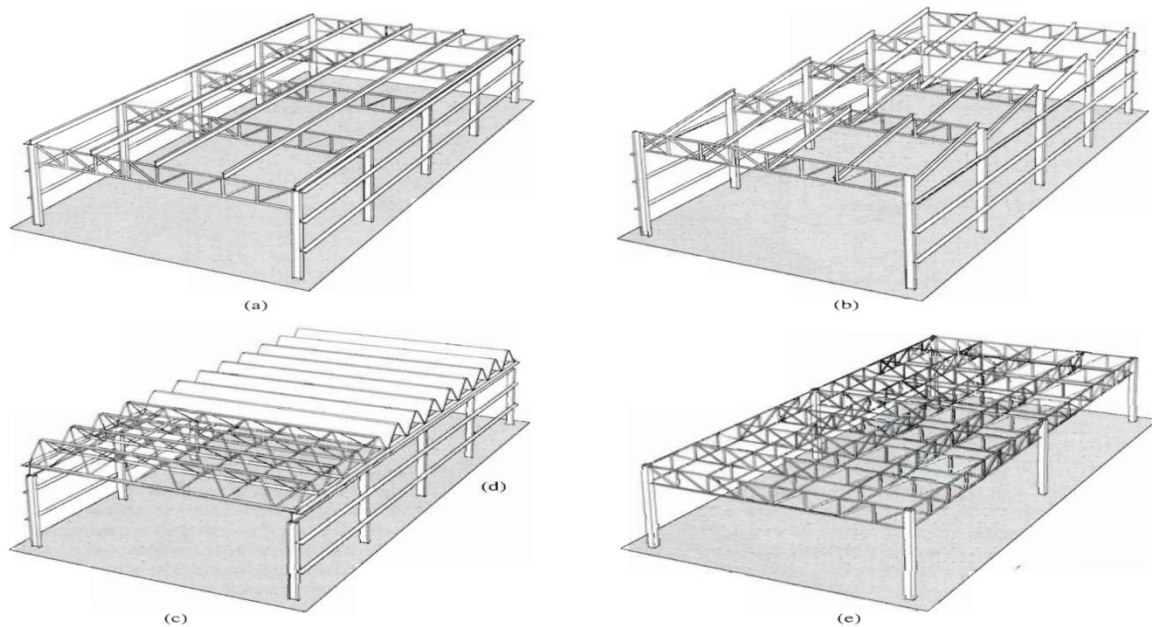


Figure IV.2: Different types of hall support system [1]



Halle Tony Garnier – Lyon



Ateliers industriels (Renault, Citroën)



Halles Baltard – Paris

IV.3 Load Path

Let us first consider the vertical loads acting on the roof, with the distributed load q representing the snow. This load will initially act on the roofing element resting on the purlins, which will in turn transmit the reactions to the frame crosspieces, which, resting on the uprights, will thus transmit the load to the foundations, thus fulfilling the vertical load-bearing function of the system. The wind, coming laterally from the left, exerts pressure on the long-span façade. The cladding rests on the horizontal wales, which will transmit their reaction to the frame uprights. The vertical and horizontal loads are therefore transmitted through elements located in planes according to the three main dimensions of the hall:

Lengthwise (purlins, wales),

Widthwise (frame crosspiece),

Height wise (uprights or posts).

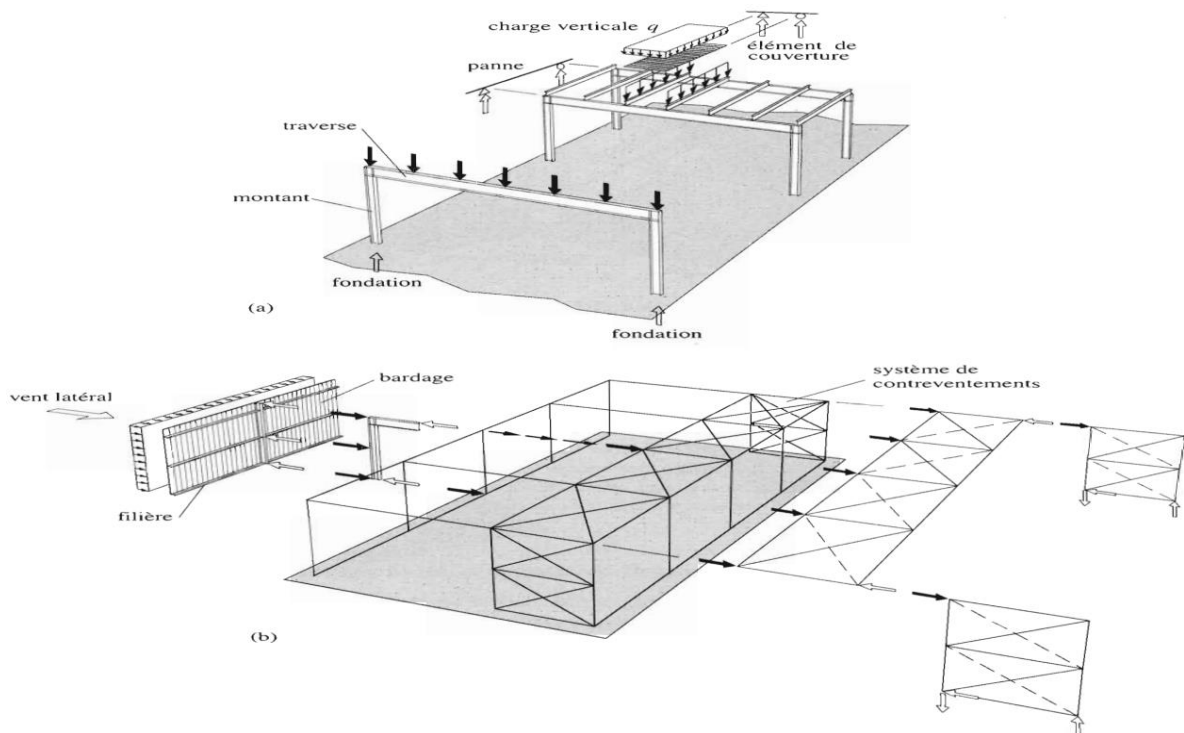


Figure IV.3: Vertical and horizontal load path [1]

IV.4 Portal Frame Structures

The most commonly used structural types in industrial buildings are portal frames with hinged bases and column-and-beam structures with embedded or hinged column bases. Portal frames provide sufficient in-plane stability and only require bracing for out-of-plane stability.

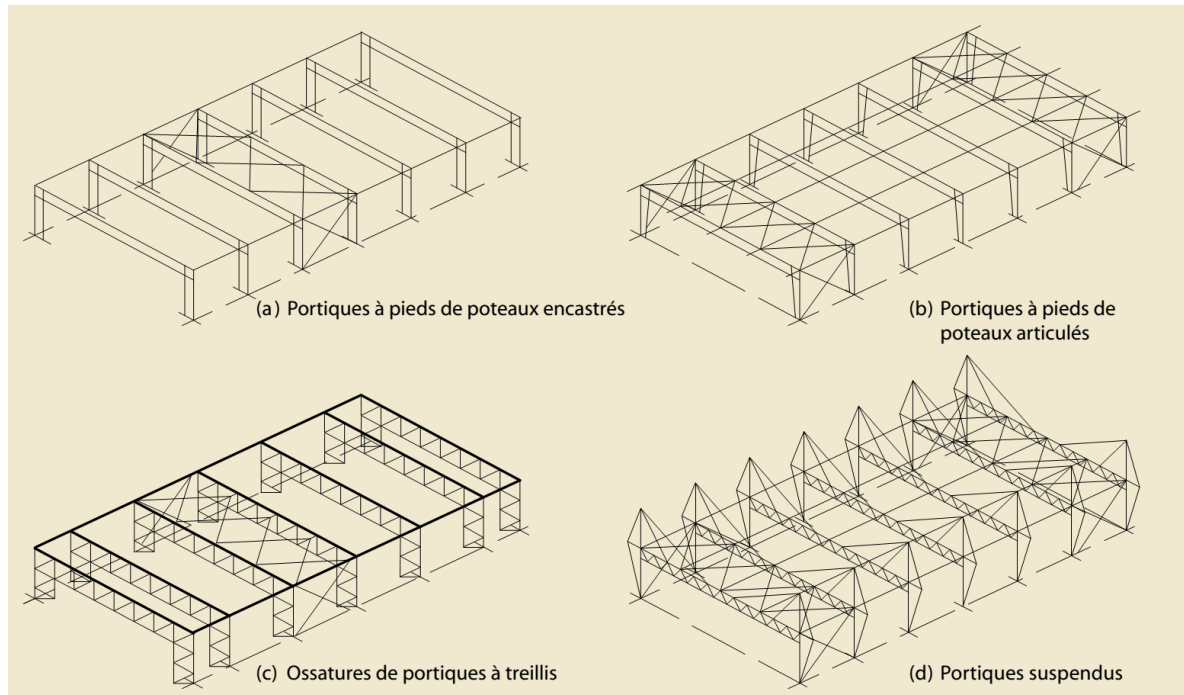


Figure IV.4: Examples of portal frames [8]

IV.5 Truss structures

The use of truss structures achieves relatively high stiffness and load-bearing capacity while minimizing material usage. In addition to their ability to create long spans, truss structures are aesthetically pleasing and simplify the integration of technical equipment. Industrial buildings with long spans can be designed using truss trusses. Installation process of a modern portal frame Barrett Steel Buildings Ltd. Lattice trusses are more likely to be post-and-beam structures and are rarely portal frames. Figure IV.3 shows various configurations of truss trusses. The two generic forms feature W- or N-shaped trusses. In these cases, stability is generally provided by bracing rather than rigid frame behavior.

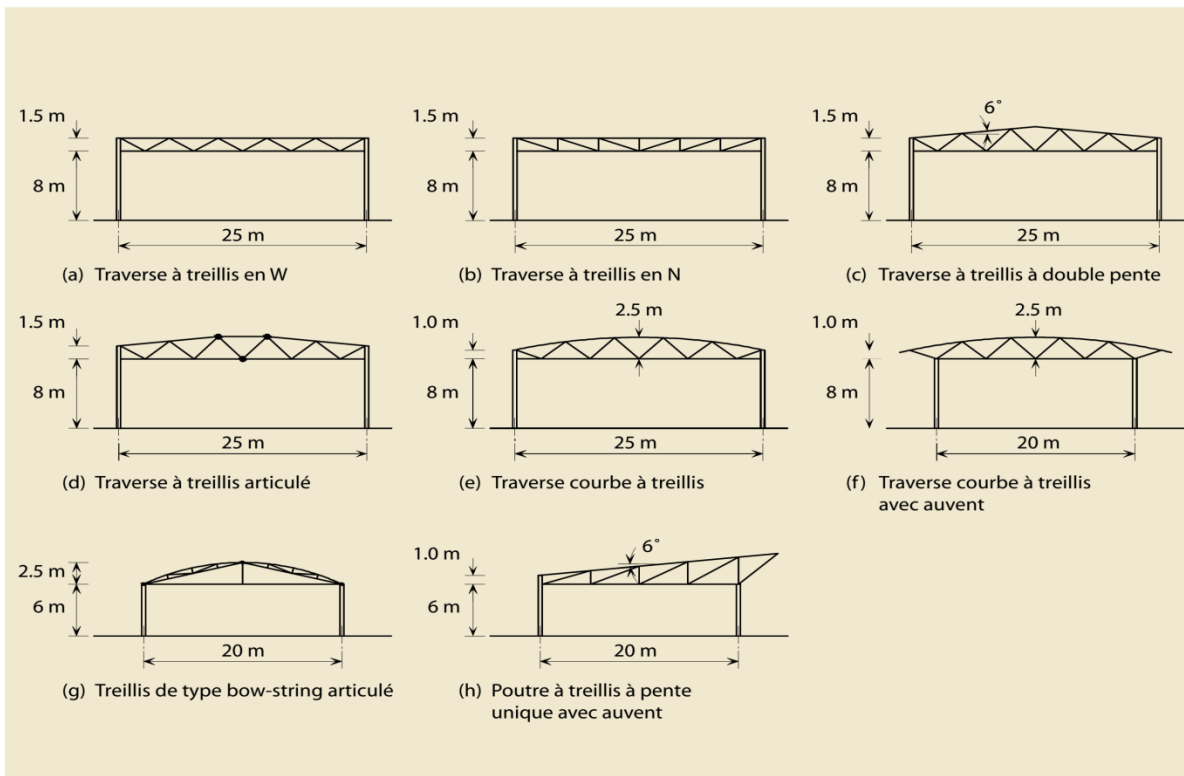


Figure IV.5: Different trusses used in industrial buildings [8]

IV.6 Arched Structures

Arched structures offer more advantageous load-bearing properties and a pleasing visual appearance. One solution may be to raise the structure on posts or integrate it into a truss, as shown in Figure IV.4.

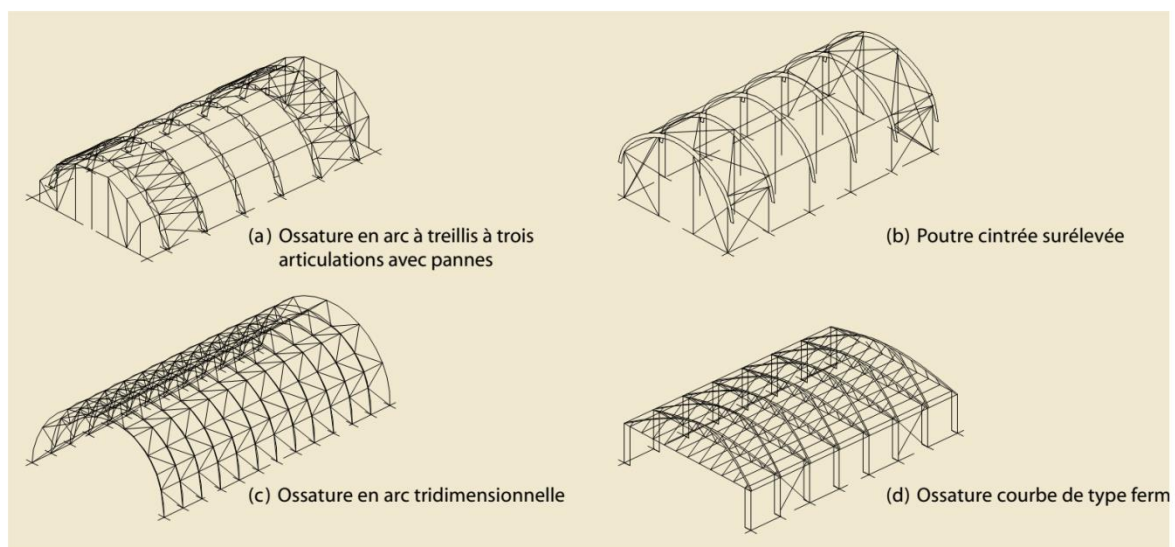


Figure IV.6: Examples of arch structures [8]

IV.7 Design of Steel Halls

IV.7.1 Stringers

Stringers are beams that are arranged horizontally at constant centers, supporting the portal posts or intermediate posts. (Wind pressure or depression).

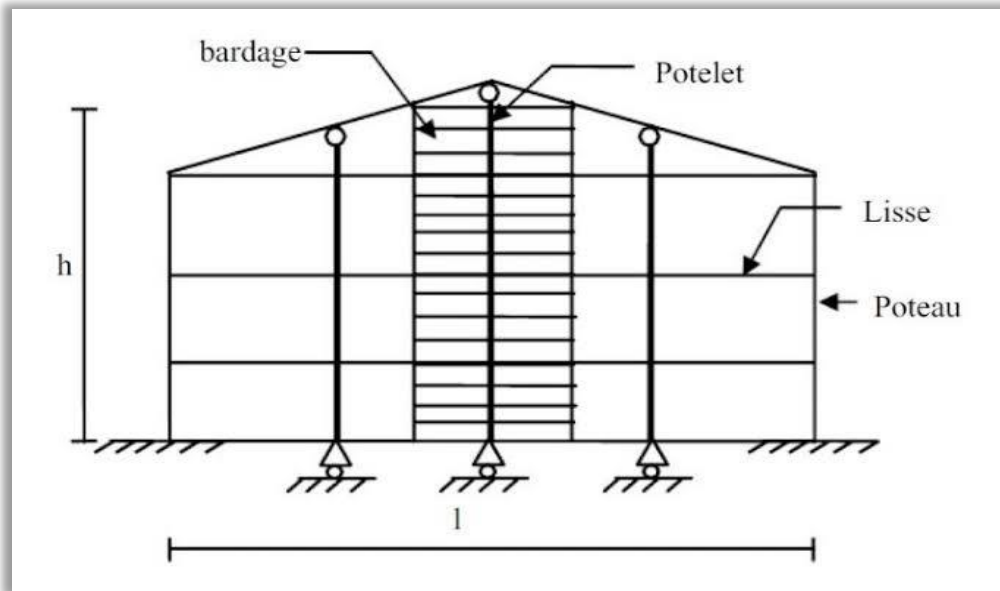


Figure IV.7: Facade with rails [7]

Their function is to support the facade coverings (cladding) and the loads applied to these claddings (wind pressure or depression).

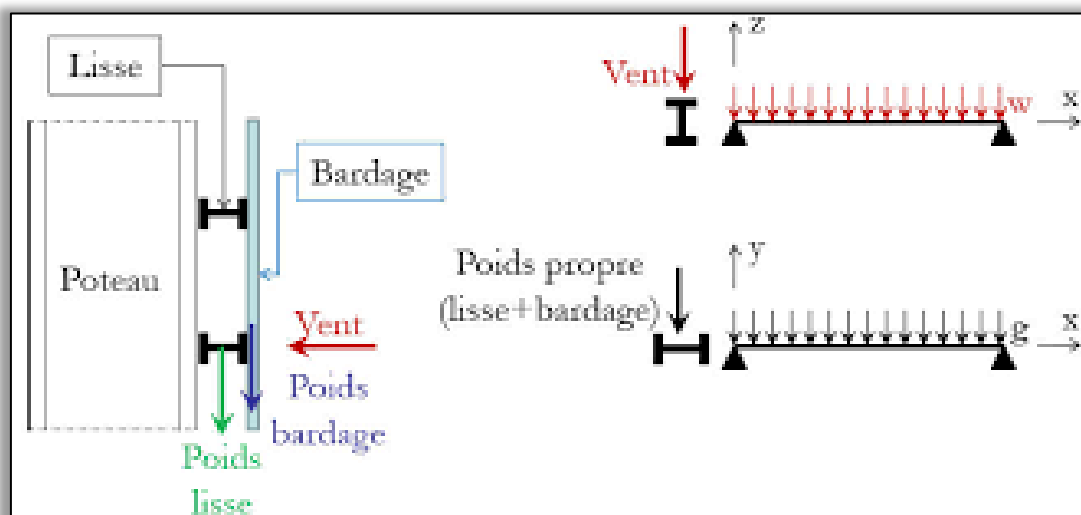


Figure IV.8: Loads applied to cladding [6]

The rails are made either from rolled profiles (IPE, UAP) or from elements formed from folded sheets. They are attached to the posts or bollards with bolts, and in certain situations (in the case of UAP rails), angle irons or folded sheets are used.

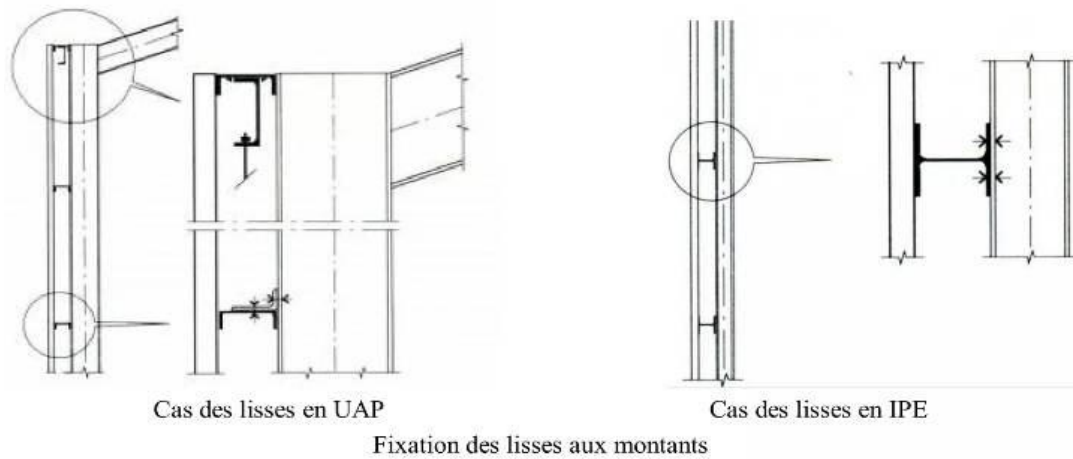


Figure IV.9: Fixing the stringers [6]

IV.7.1.1 Principle of sizing and calculation of stringers

Stringers are sized and calculated to simultaneously satisfy:

- Strength conditions (ULS: bending + shear)
- Stability condition (ULS: lateral buckling)
- Deflection condition (ELS).

IV.7.2 Purlins

Purlins are structural elements on which the roof of a building is placed. Their main role is to support the roof and transmit forces to the frames. They can be made of hot-rolled sections (IPE) or cold-formed sections (C, Z, etc.), or even lattices in cases where the spans are very long.

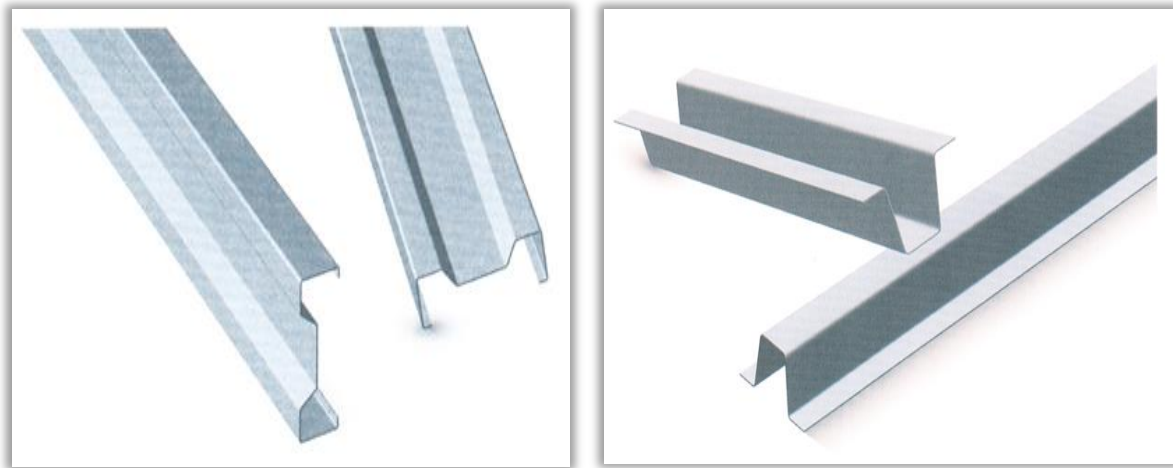


Figure IV.10: Cold-formed thin profile sigma and omega [7]

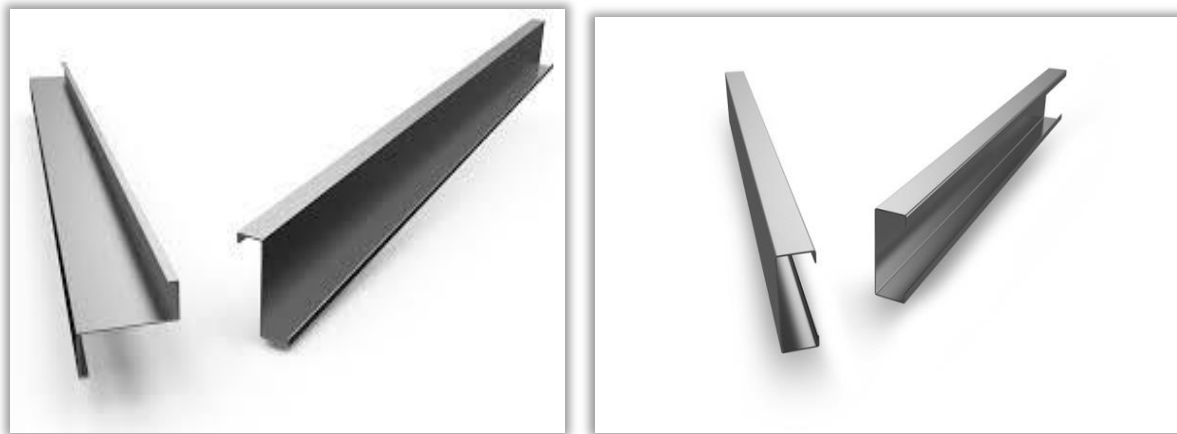


Figure IV.11: Cold-formed thin section Zed and Cé [7]

Depending on its position in the frame, the purlin has a specific name: The ridge purlin, located at the top of the frame of a pitched roof; the wall purlin, located at the bottom of the slope. It can rest on the top of the wall. The intermediate purlins, located between the wall purlin and the ridge purlin, are called belly purlins.

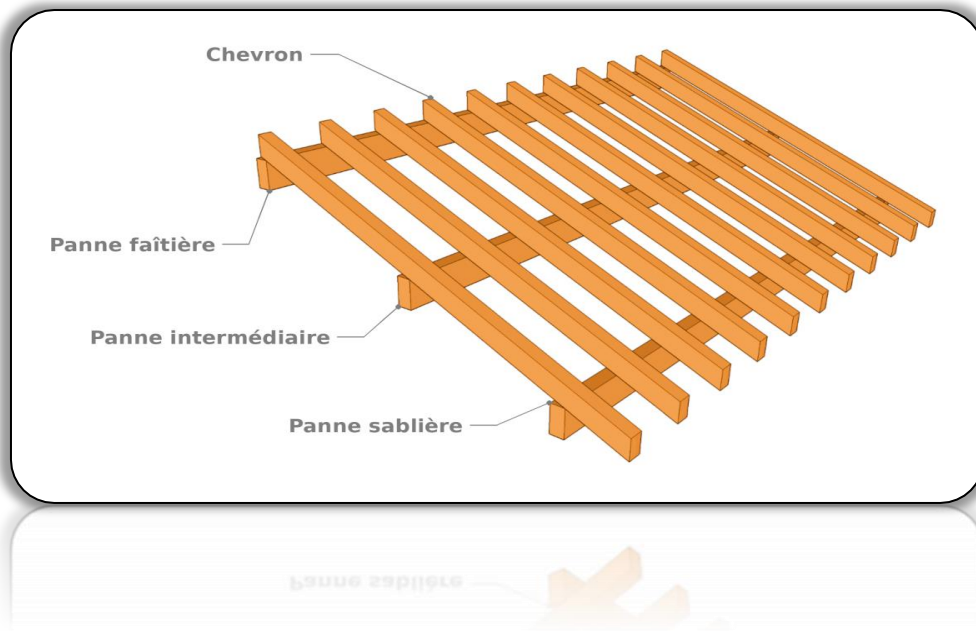


Figure IV.12: Types of purlins [5]

Purlins are bolted to the frame cross members or trusses. On sloping roofs, to prevent slipping and tilting during installation, the purlins are secured with shims.

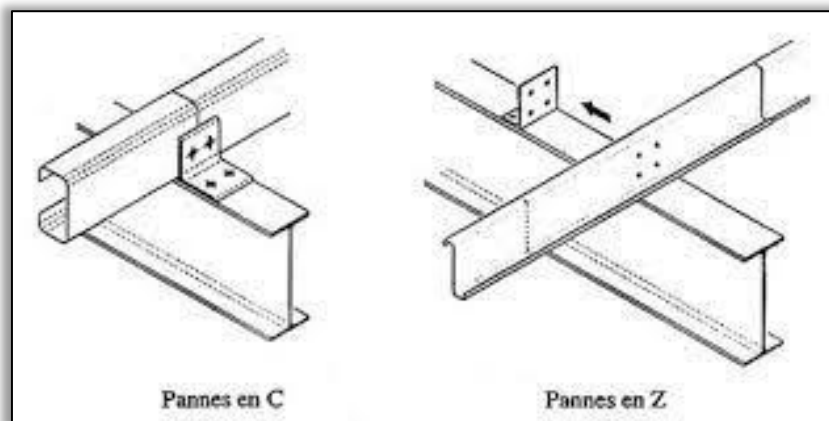


Figure IV.13: Joint with joists [2]

Joists provide the connection between the purlins and the main structure. The purlins are suspended to prevent local compression of the web. Therefore, there is a clearance of approximately 10 mm between the frame and the profile. Standard joists are manufactured from high-strength steel and hot-dip galvanized. The joists are attached to the trusses either by bolting or welding. The purlins are attached to the joists by bolting.

IV.7.3 Bracing Design

A brace is a static system designed to ensure the overall stability of a structure against the horizontal effects of possible actions on it (e.g., wind, earthquake, impact, braking, etc.). It also serves to locally stabilize certain parts of the structure (beams, columns) against instability phenomena (buckling or lateral buckling). Bracing ensures the horizontal and vertical stability of the structure during tremors, which, as a reminder, have components in all three directions.

The role of horizontal bracing is to transmit lateral forces to the vertical elements called stability beams. To ensure horizontal bracing, floors and roofs acting as rigid diaphragms should not be weakened by excessively large or poorly placed openings that could compromise their strength and rigidity. Flexible diaphragms should be avoided to combat lateral buckling of walls, particularly masonry walls.

Vertical bracing using joists should meet specific criteria such as: Number: at least three non-parallel and non-intersecting joists per floor. Arrangement: they should be located as symmetrically as possible relative to the center of gravity of the floors, preferably at the corners, with sufficient width.

Vertical distribution: should be regular; the joists should preferably be stacked to provide the different levels with comparable rigidity, both in translation and torsion. Bracing is a construction principle used to stabilize and stiffen a structure.

It is used: On roofs in sloping sections: these are referred to as wind beams; On facades in vertical sections: these are referred to as stability joists. The term "bracing" is used in various construction trades: timber framing, metal framing, carpentry, and even masonry.

IV.7.3.1 Different Types of Bracing

There are three types of bracing: Triangulation: Triangulation (truss) is the most common type: K, V, and X trusses are the most effective. The term bracing refers to diagonal elements found in a frame, such as the braces in a traditional frame, which reinforce the rafters and triangulate the frame.



Figure IV.14: Types of triangulation bracing [5]

Nodal stiffening (portal frame) avoids bulky or even unsightly diagonals. This type of bracing allows for more structural modifications and more opportunities to create openings. In a metal structure such as a hangar or industrial building, portal frames are used, consisting of posts and beams supported on them to support the building's envelope. This principle thus creates large spaces for large-scale storage.



Figure IV.15: Types of bracing, stiffening of nodes [7]

Infill: (wall, concrete or wooden wall) with a load-bearing wall on the facade or a partition wall perpendicular to a facade.

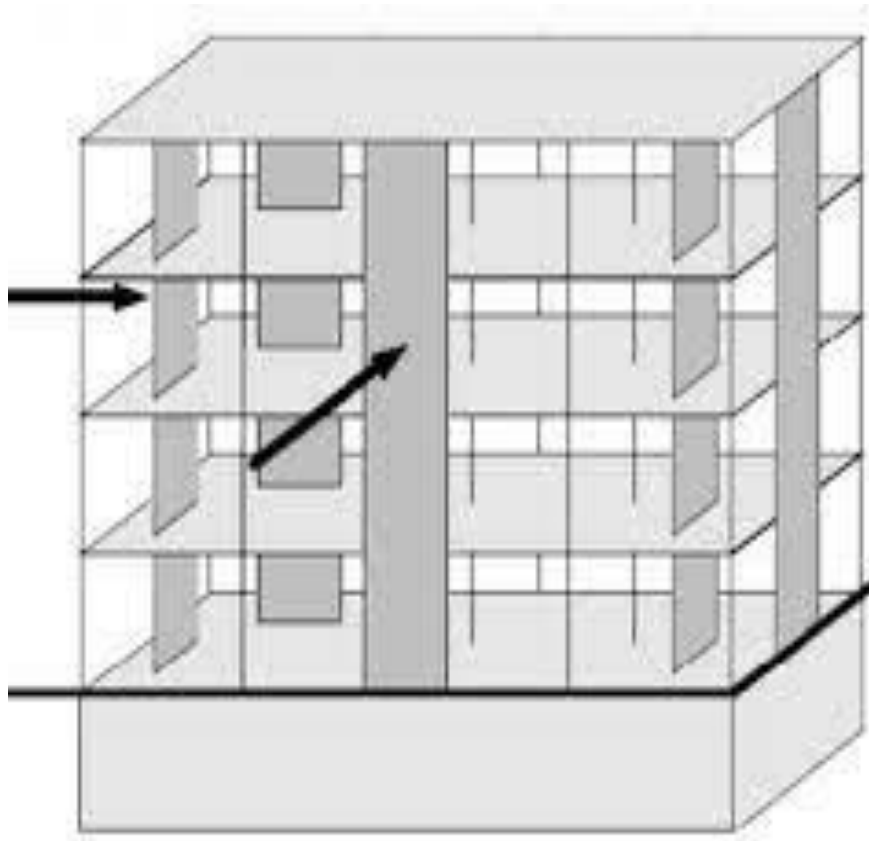


Figure IV.16: Infill [7]

IV.7.4 Stability Bevel

A well-designed structure must be able to ensure its stability under the action of any external load, particularly horizontal loads (wind pressure on the facades, seismic shock, vehicle impacts, etc.). In other words, the overall design of a structure, as well as that of each of its components, must ensure the downward movement of horizontal loads from their points of application to the foundations. A stability bevel is a structural element inscribed in a vertical plane and capable of resisting a horizontal force parallel to its plane.

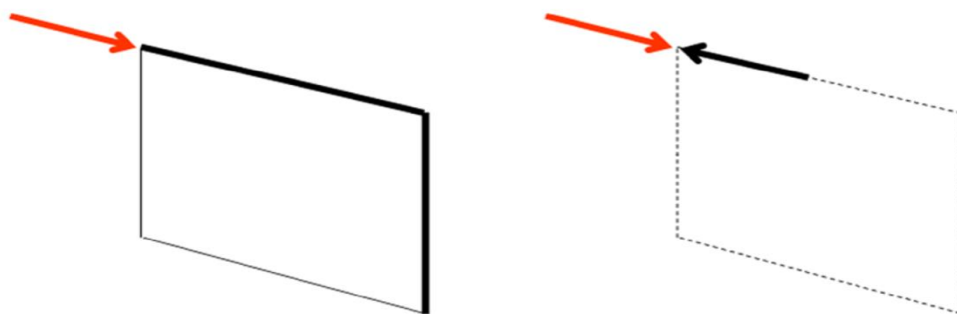


Figure IV.17: Stability beam [1]



Figure IV.18: Single-skin cladding [5]

Double-skin cladding Since single-skin cladding lacks insulation, double-skin cladding overcomes this drawback. This type of cladding consists of:

- An interior facing, called a plate, attached tightly to the exterior face of the main frame posts. They are installed horizontally (spans of 5 to 6 m).
- Thermal insulation, the choice of which depends on the desired thermal transmission coefficient.
- An exterior facing consisting of vertically arranged ribbed profiles attached to the interior facing.

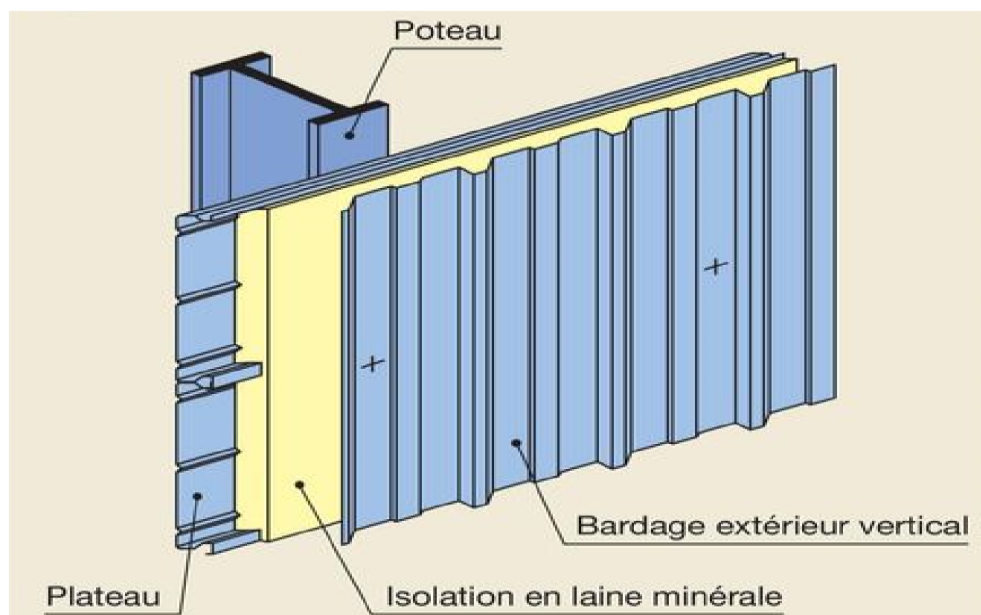


Figure IV.19: Double-skin cladding [7]

Sandwich panel cladding

In this type of cladding, the components (outer skin, insulation, inner skin) are pre-assembled in the workshop to form single-piece panels. These prefabricated panels consist of two facings, most often made of treated sheet metal, enclosing mineral insulation or joined by injected synthetic foam, forming the insulating core of the element. The insulation thickness can range from 30 to 80 mm. The K coefficient of the panel ranges from 0.69 to 0.23 W/m², depending on the thickness of the insulation.

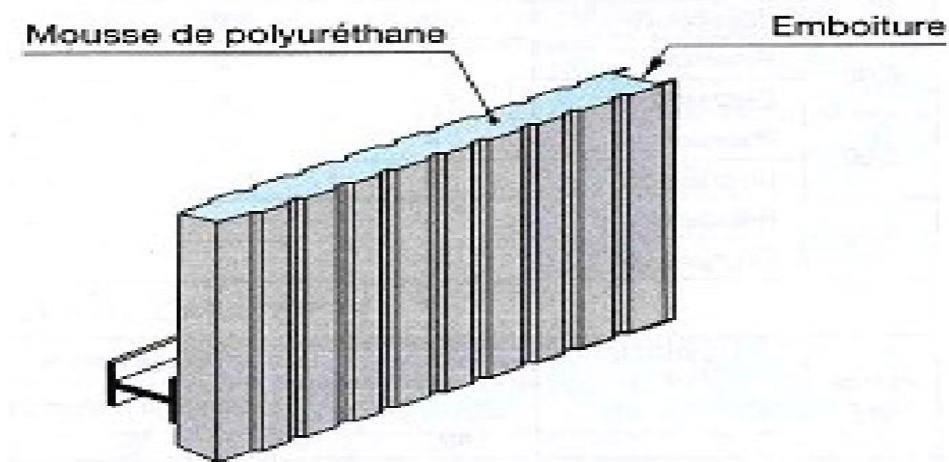


Figure IV.20: Sandwich panel cladding [7]

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