People’s Democratic Republic of Algeria
Ministry of Higher Education and Scientific Research
Ahmad Zabana University of Relizane
Faculty of Science and Technology
Department of Civil Engineering and Public Works

Steel Structures Course

Presented for Third-Year Civil Engineering and Public Works (L3 CE & PW)

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Strive relentlessly to gain wisdom, as if there is

always more to learn.

Cherish and protect what you have already acquired.

For those who do not advance each day,

inevitably fall behind.

Confucius

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Preface

This polycopy, extracted from the Steel Structures course taught at the Department of Civil Engineering at Ahmed Zabana University, serves as an introductory guide to familiarize students with the vocabulary and calculation techniques in this field. It is intended for third-year Bachelor's degree students in Civil Engineering and Public Works, who have steel structures in their curriculum and wish to acquire the basic calculation methods used in the design of steel structures.

The methods presented in this booklet are based on the Algerian Steel Structures Design Code CCM97, as well as Eurocode 3.

The first chapter of this booklet is devoted to general information about steel, the processes used to produce steel products, and the vocabulary of steel construction.

The second chapter introduces basic concepts necessary to ensure the safety of structures, which must be calculated taking into account all the differences that may exist between the actual structure and the theoretical calculation model.

The third chapter addresses assembly methods, including bolts and welds.

The fourth chapter is devoted to tension members subjected to simple axial tension.

Finally, the fifth chapter deals with the verification of the strength and stability of flexural members.

The booklet concludes with an appendix containing tables of profile characteristics and a list of bibliographical references.

Introduction

General Introduction

Steel structures form a cornerstone in modern and industrial construction. Their ability to provide lightweight yet highly durable frameworks enables the design of buildings with long spans and diverse architectural forms, while minimizing the self-weight of the structure. Steel is the preferred material due to its outstanding mechanical properties, including **high tensile strength**, **ductility**, and **homogeneity**, which ensure both safety and longevity.

The study of steel structures covers several complementary aspects:

1. **Design of structural members**: beams, columns, girders, and frames, taking into account permanent and variable loads.
2. **Connections and joints**: welding, bolting, or riveting, which directly influence the overall strength and stiffness of the structure.
3. **Analysis of internal forces**: calculation of bending moments, shear forces, axial forces, and torsion to properly size each member.
4. **Verification according to standards**: compliance with **Ultimate Limit States (ULS)** for safety and **Serviceability Limit States (SLS)** for performance and comfort.

This pedagogical lecture notes aim to provide students with an integrated approach to the **design and sizing of steel structures**, combining theoretical fundamentals of structural mechanics with practical applications according to **Eurocode 3**. Numerical examples, explanatory diagrams, and tables of steel section properties are also included to facilitate learning and practical implementation.

General Overview of Steels

Chapter 01

General Overview of Steels

1.1 Introduction

The design of the load-bearing structure for a hall or building is determined by its intended use, emphasizing two primary aspects: resistance (to ensure adequate structural safety) and deformability (to guarantee proper serviceability). This design is significantly influenced by the properties of the materials that make up the structure. Consequently, the development of a steel framework must be approached with a focus on optimizing the inherent qualities of steel, such as its exceptional mechanical strength, high ductility, and weldability.

The formulation of design criteria for a structural framework necessitates a comprehensive understanding of steel, encompassing its production processes, fundamental properties, and the fabrication techniques of its products.

Iron, the primary component of steel, naturally occurs in oxide form. It stands out as the hardest, most prevalent, and widely utilized metal compared to other materials.

Historically, iron has been extensively used by early civilizations across various domains, including weaponry and agriculture, and has eventually found its way into the field of interest here: civil and industrial construction.

1.2 Evolution of Steel Structures :

Iron began to emerge as a construction material in the mid-17th century, during an era when wood and stone were the predominant building materials. Initially, its primary functions were ornamental and structural reinforcement. Many metallic components were used to secure stones in place through clamps and fastenings. By the late 17th century, metallic elements were no longer concealed within stone walls but instead became integral to the primary framework of constructions, marking the beginning of a new architectural style.

In 1750: The industrialization of cast steel began.

In 1779: The first metal bridge, designed by Abraham Darby, was constructed. The Coalbrookdale Arch Bridge over the Severn River (England) was made of cast iron and spanned 31 meters.

In 1786 : Victor Louis designed the first structure composed entirely of a metal framework—the iron roof of the French theater in Paris.

In 1801 : The first true metal framework for a building, featuring beams and columns, was constructed in England.

In 1881 : Discovery and development of electric arc welding.

In 1889: Construction of the Eiffel Tower in Paris, a riveted structure 300 meters tall, for the Universal Exposition.

In 1931 : Introduction of cold-drawn steel wires (with a tensile strength of 1520 N/mm^2) in construction by Swiss engineer O.H. Ammann. This innovation was used in the George Washington Bridge in New York, which features a central span of 1067 meters.

In 1931 : Completion of the Empire State Building in New York, a steel-framed structure reaching 380 meters in height.

In 1973 : Construction of the World Trade Center in New York, comprising two towers, each 110 stories tall and 410 meters in height.

In 1974: Construction of the Sears Tower in Chicago, a 109-story building with a total height of 442 meters.

In 1981: Completion of the Humber Bridge in Hull, Great Britain, a suspension bridge with a central span of 1410 meters.

In 1998: Anticipated completion of the Akashi Kaikyo Bridge in Japan, a suspension bridge with a central span of 1990 meters.

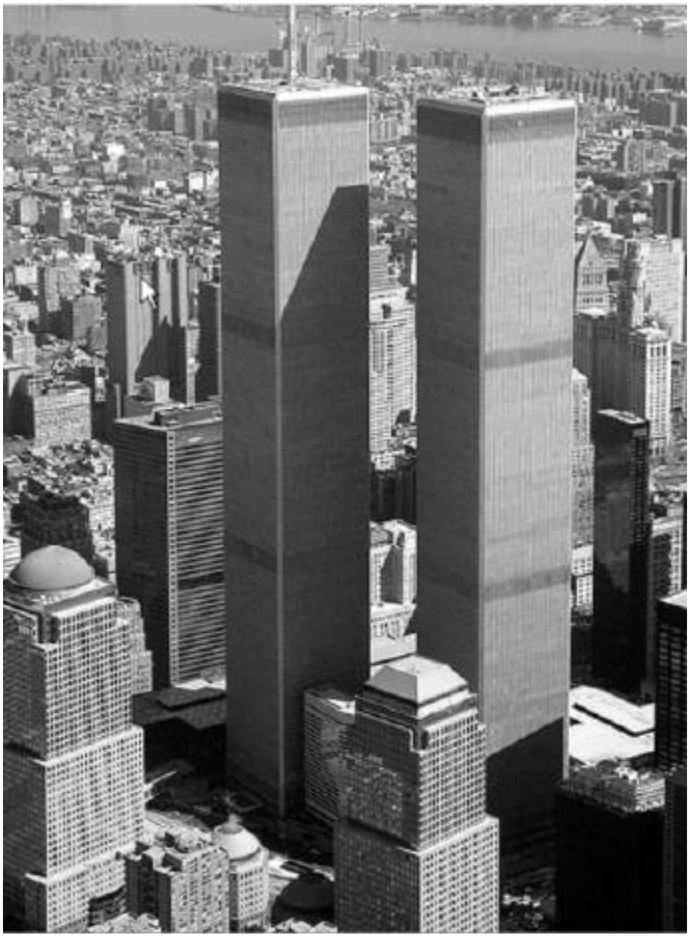


Figure (1.1) **Construction of the World Trade Center in New York (1973)**

1.3 Advantages and Disadvantages of Steel Construction

1.3.1 Advantages :

- Mechanical Strength

  - High tensile strength allows for significant spans and heights.

  - Plastic adaptability enhances structural safety.

- Full Industrialization : Prefabrication is feasible, streamlining construction processes.
- Ease of Transformation or Modification : Steel structures can be altered or adapted with relative simplicity.
- Architectural Possibilities : Allows for more intricate architectural designs with larger spans.
- Recyclability : Components can be recovered and reused after decommissioning.
- Seismic Resistance : Steel's ductility provides better performance during seismic events.
- Lightweight Nature : Reduces the impact of seismic forces due to lower structural weight.

1.3.2 Disadvantages :

- High Cost : Steel construction can be expensive, particularly when compared to reinforced concrete for larger spans.

- Poor Fire Resistance : Steel performs poorly under fire exposure, necessitating costly protective measures.

  - *Solutions* : - Application of specialized fire-resistant paints.

    - Use of insulating materials, such as plaster, or combining steel with concrete to slow fire propagation.

- Corrosion : Surface rust layers develop due to atmospheric humidity. Rust is an oxide formed under such conditions.

  -*Solutions :* - Application of anti-corrosion paints.

  -Use of metallic coatings, such as chromium plating or galvanization with zinc.

1.4 Steel :

Steel is a material primarily composed of iron and carbon. It is produced from natural raw materials extracted from iron and carbon mines.

The carbon content in steel is minimal, typically less than 1.7%.

1.4.1 Steel production :

Steel is typically produced through a two-phase process:

- *First phase :* Iron ore is introduced into a blast furnace, where it undergoes combustion and is heated until it melts. This process produces pig iron, a material containing more than 1.7% carbon.
- *Second phase :* The molten pig iron is converted into steel at a temperature of approximately 1500°C. This step involves decarburizing the pig iron to reduce its carbon content. The resulting steel contains less than 1.7% carbon.

At the conclusion of the steelmaking process, the molten steel is poured into molds (usually copper molds) to form solid bars with square or rectangular cross-sections.

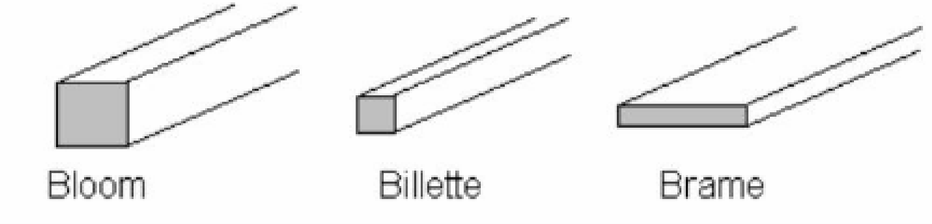
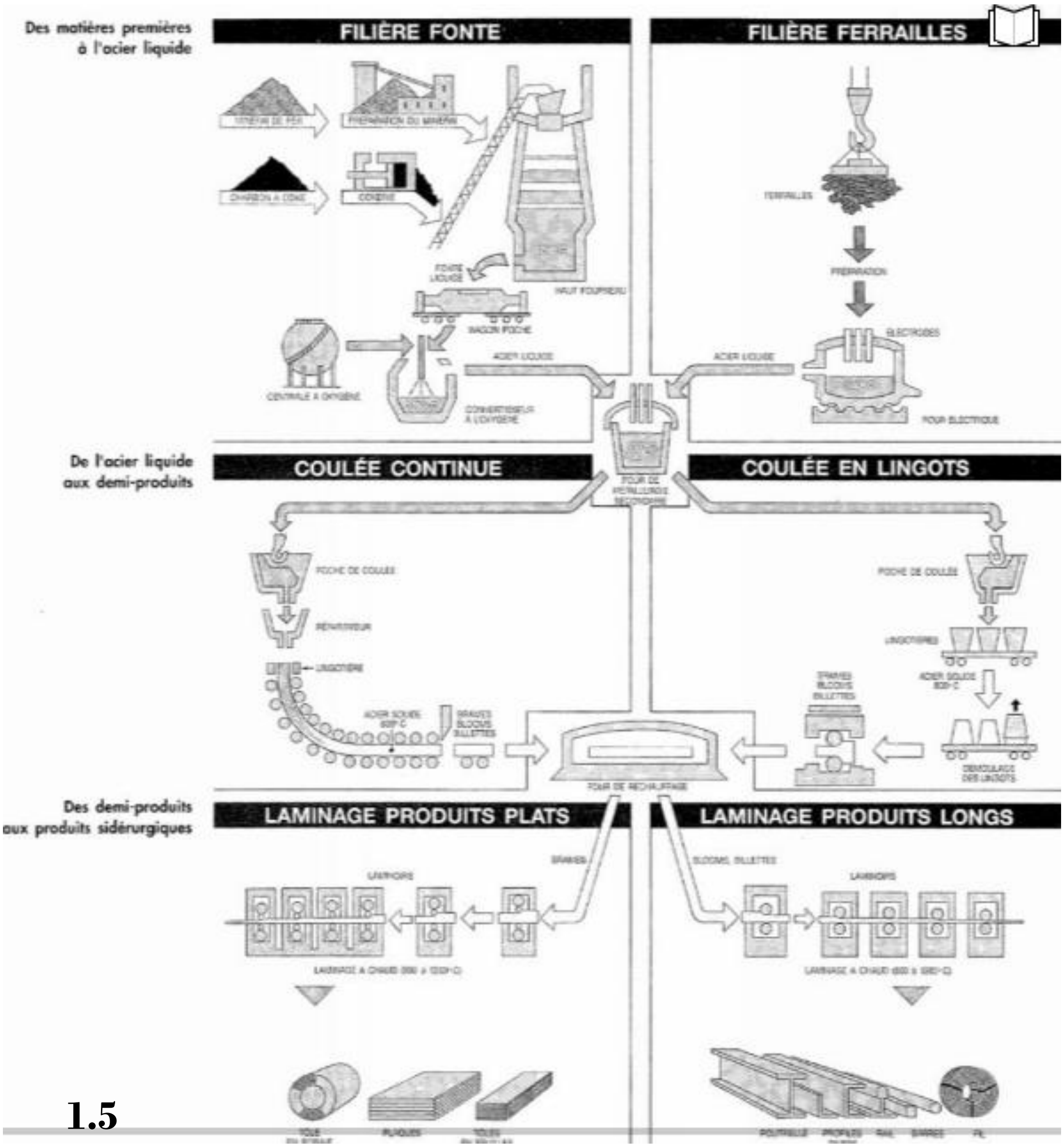


Figure (1.2) **Semi-Finished Products (Bloom, Billet, and Slab)**

- *Finale phase :* The final phase involves rolling the semi-finished products, which means stretching and compressing the metal to achieve the desired dimensions and shapes. This process is carried out at high temperatures, approximately 1000°C.



1.5

Figure (1.3) **Steel Production Processes**

1.5 Main Products Used as Structural Elements

1.5.1 Hot-Rolled Products:

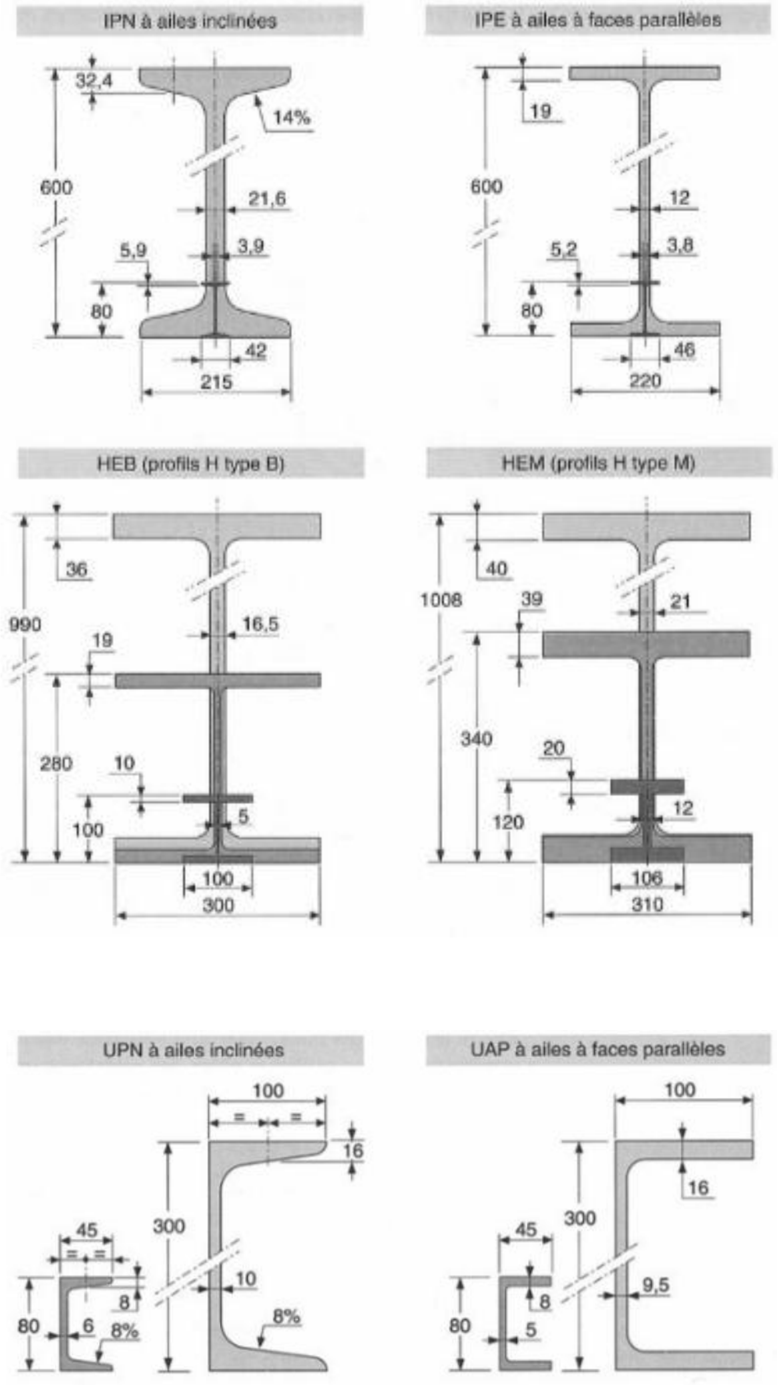


Figure (1.4) **Hot-Rolled Products**

1.5.2 Cold-Formed Products

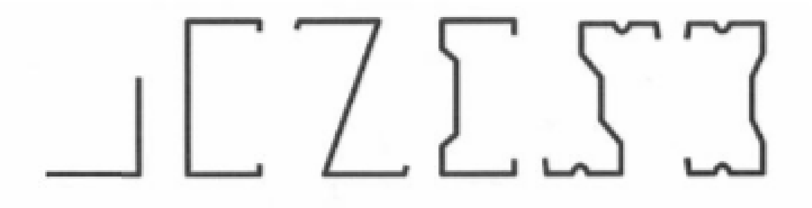


Figure (1.5) **Cold-Formed Long Products (Examples of Sections T and S)**

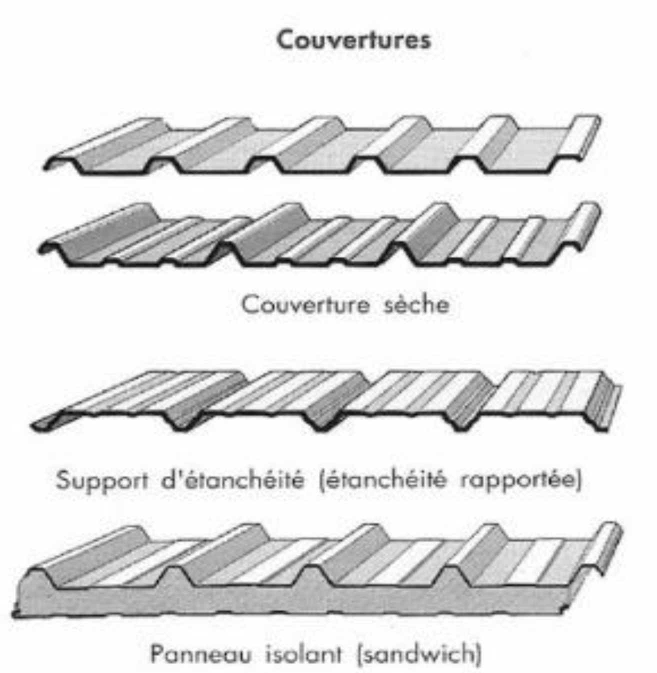


Figure (1.6) **Cold-Formed Flat Products**

1.5.3Products Derived from Rolled Profiles and Welded Reconstituted Profiles

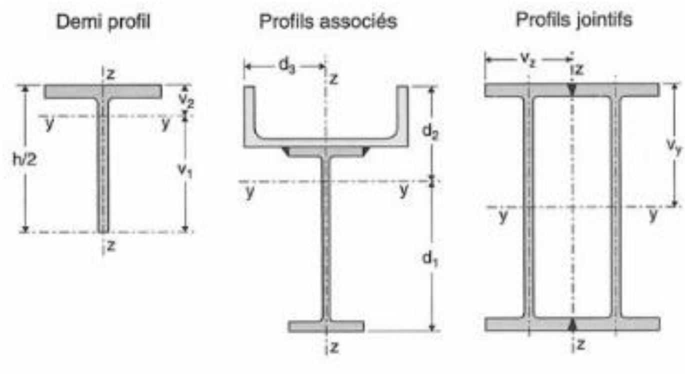


Figure (1.7) **Derived Products**

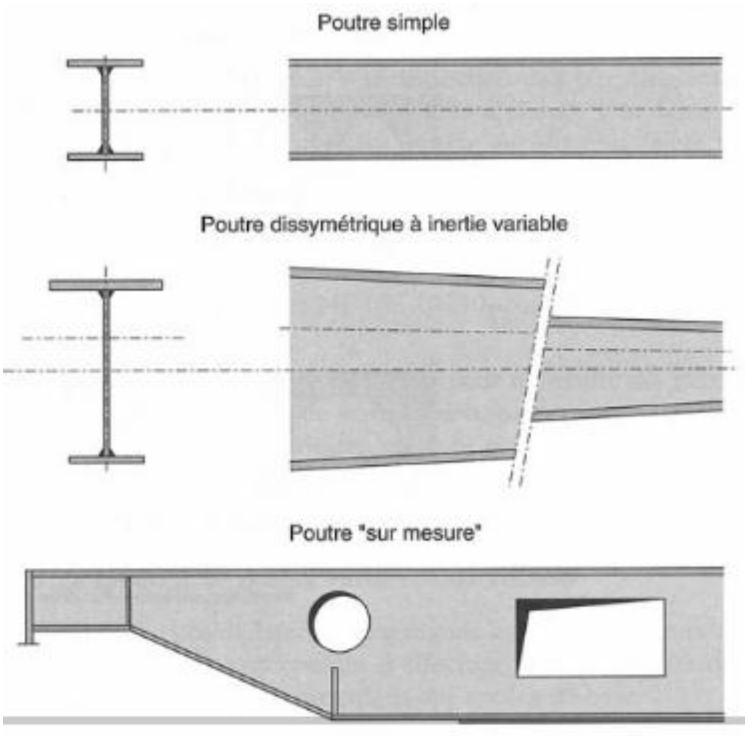


Figure (1.8) **Welded Reconstituted Profiles**

1.6 Steel Classification :

Table (1.1) Steel Classification.

	Material	Carbon Content	Application Domain
Steel	*Mild Steel*	*0.05< C<0.30*	*Frameworks, Fasteners*
	Medium-Hard Steel	*0.30< C<0.60*	*Rails*
	Hard Steel	*0.60< C<0.75*	*Tools*
	Extra-Hard Steel	*0.75< C<1.20*	*Tools, Punches*
	Wild Steel	*1.20< C<1.70*	*Special Parts*
Cast Iron	*Hypoeutectic Cast Iron*	*1.70< C<4.50*	*Engine Heads*
	Hypereutectic Cast Iron	*4.50< C<6.30*	*Machine Bases*

1.7 Mechanical Tests :

1.7.1 Tensile Test:

This test is conducted on a cylindrical specimen subjected to an increasing tensile force, starting from zero until fracture occurs. By measuring the elongation of the specimen as a function of the applied force, a diagram is obtained representing either the Force-Elongation or the Stress-Strain relationship.

$$\sigma = \frac{F}{S_0}$$

Where : $$\Delta l = L - L_0$$

$$\Delta\varepsilon = \frac{\Delta L}{L_0}$$

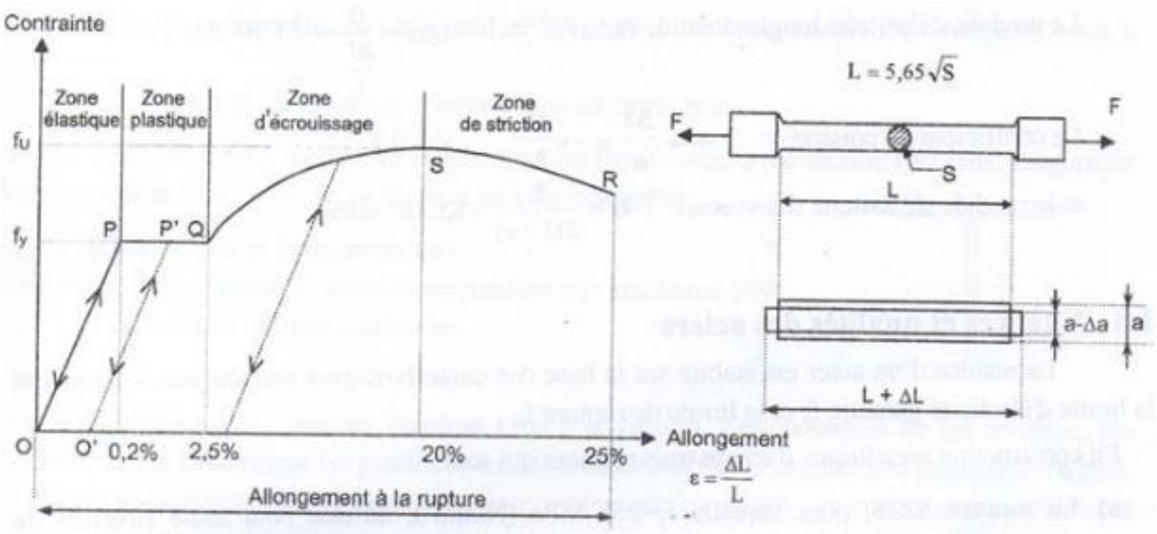


Figure (1.9) **Tensile Test**

This diagram can be divided into four phases:

- *Elastic Domain (Straight Line OP)*: The variation σ(ε) is linear (following Hooke's Law) and reversible. The elongation is proportional to the applied forces. If the force is removed, the bar returns to its original length.
- *Plastic Domain with Small Deformations (Plateau PQ)*: The elongation continues at a constant stress value σ_e. A release of the force from point P on this plateau results in elastic shortening along a line O'P' // OP. However, a permanent deformation OO' (residual strain) remains. This phenomenon is referred to as the flow of steel.
- *Plastic Domain with Large Deformations (Work-Hardening Zone → Curve OS)*: From point Q, the curve σ(ε) continues to rise but with a much lower slope compared to the elastic phase, and the slope gradually decreases until it reaches a peak at point S. This phase is called work-hardening, as it reflects the steel's adaptation, where, when loaded beyond f_y , it becomes more rigid.
- *Necking Domain (Curve SR)*: Beyond the peak at point S, the metal continues to elongate while its resistance decreases. The metal ultimately

fractures after a reduction in cross-section within a zone known as the "necking" region.

This diagram allows for the measurement of :

- The yield strength of steel f_y ;
- The ultimate tensile strength f_u ;
- The elongation at fracture A ;
- The Longitudinal Elastic Modulus of Steel:

  E= tg α = $\frac{\sigma}{\frac{\Delta L}{L}} = 2.1\ 10^5\ MPa$
- Poisson's Ratio: **ν** $\rightarrow \frac{\Delta a}{a} = -\frac{\Delta L}{L}$ **ν=0.3**
- The Transverse Elastic Modulus (Shear Modulus):

  $G = \frac{E}{2(1+\nu)} = 8.1\ 10^4\ MPa$

1.7.2 Impact Bending Test (Resilience Test):

The purpose of this test is to measure the energy absorbed by a simply-supported specimen with a V-shaped notch at its midpoint during fracture in bending caused by the impact of a pendulum hammer. This energy characterizes the ductility of the steel and its susceptibility to brittle fracture as a function of temperature.

$$\text{Fracture Energy} = m\ g\ (h_0 - h)$$

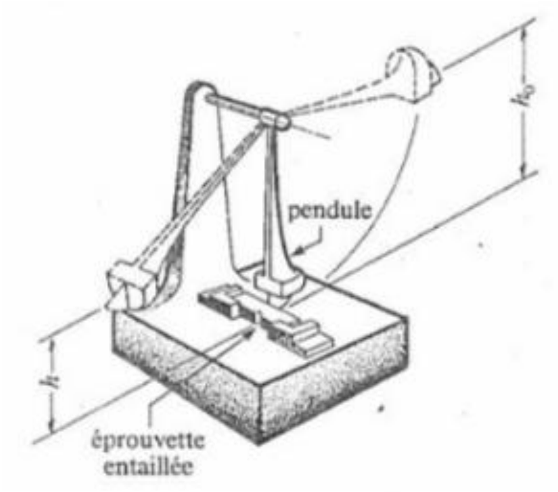


Figure (1.10) **Principle of the Impact Bending Test**

1.7.3Hardness Tests :

Hardness tests involve measuring the penetration of a standardized tool into the test specimen under a predetermined load.

1.7.4 Bending Tests :

This test qualitatively evaluates the ductility of steel and its suitability for cold forming by bending sheets or bars made from the material.

1.8 Steel Grades and Properties :

The grade of steel is determined based on its mechanical properties, particularly the guaranteed yield strength (fy) and ultimate tensile strength (fu).

In steel construction, there are three main grades:

a) **Grade S235**, for wich f_y=*235 MPa (N/mm²)*, used for all building structures. Ultimate tensile strength : f_u=*360 MPa*.

b) **Grade S275,** for wich f_y=*275 MPa,* Likely to be required by public project owners (e.g., hospital, etc.). For this grade, f_u = *430 MPa* is given.

c) **Grade S355,** for wich f_y=355 *MPa*, Used for bridges and civil engineering structures, and possibly in buildings when large spans are involved. f_u = 510 *MPa* is given.

Table (1.2) *Steel grades and properties*

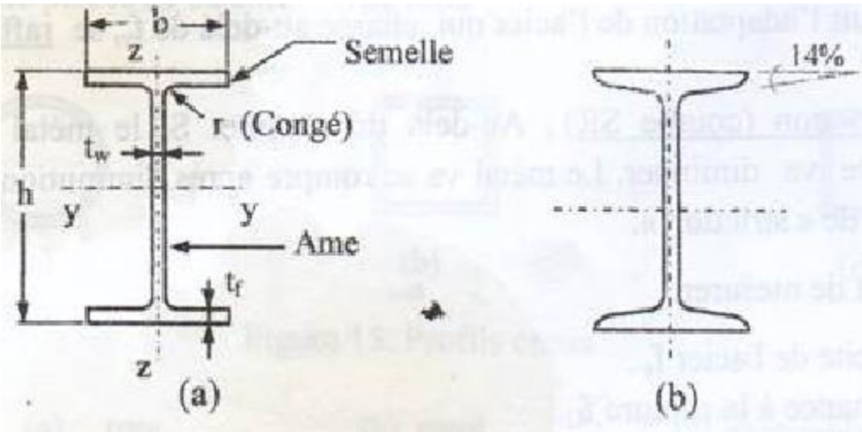
Steel Grade NF		Thickness t en mm			
		t ≤ 40 mm		40 mm < t ≤ 100 mm	
EN 10025	EN 10027	f_y [N/mm²]	f_u [N/mm²]	f_y [N/mm²]	f_u [N/mm²]
Fe360	S235	235	360	215	340
Fe430	S275	275	430	255	410
Fe510	S355	355	510	335	490

1.9 Characteristics and Applications of Common Profiles:

Steel profiles are produced through hot rolling, and their characteristics are standardized. They are listed in catalogs and include the following:

a) I-Beams : These are primarily used to resist simple or deviated bending. The types include:

- **IPN** (standardized sections): The inner faces of the flanges are inclined at 14% relative to the outer faces.
- **IPE** (Extra-Light I-Profile): The inner faces of the flanges are parallel to the outer faces.



(a) IPE h= 80 ÷ 600 mm (b) IPN h= 80 ÷ 200 mm

Figure (1.11) **I-Beams**

b) H-Beams : The cross-section of H-beams approximates a square, with height h ≈ b.

They are used to resist buckling or combined buckling and bending (e.g., in columns, diagonal braces for stability frameworks). These are divided into three categories:

- HEA: With reduced flange and web thickness.
- HEB: Standardized profiles corresponding to the former HN profiles.
- HEM: With reinforced flange and web thickness.

c) U-Beams : These profiles are well-suited for bending resistance, which makes them ideal for applications such as purlins and roof bracing in industrial buildings. Additionally, they are used as bracing diagonals (in pairs) or as components of lattice columns.

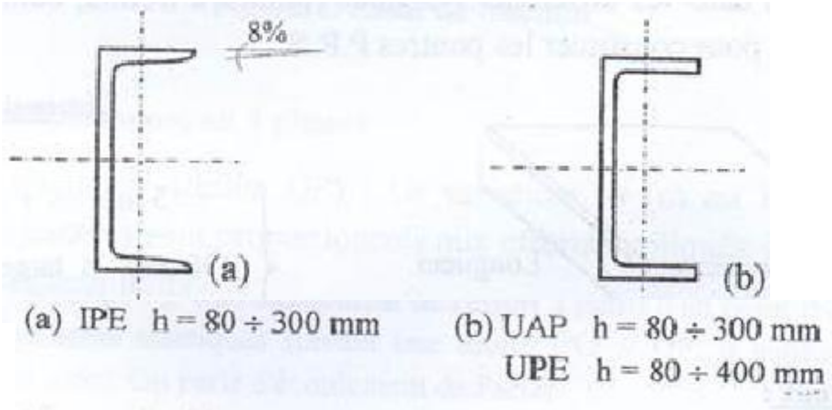


Figure (1.12) **U-Beams**

d) Angles and T-Beams : Equal and unequal angle sections are used to form the following elements:

- Truss bars: lattice beams, bracing diagonals.
- Assembly or fastening elements.

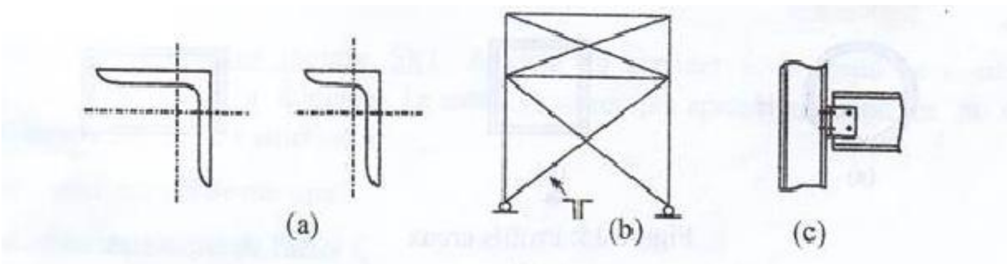


Figure (1.13) **Use of Angles**

T-Beams Equal and unequal-leg T-beams are generally used in truss beams (trusses) where they serve as the upper and lower chords.

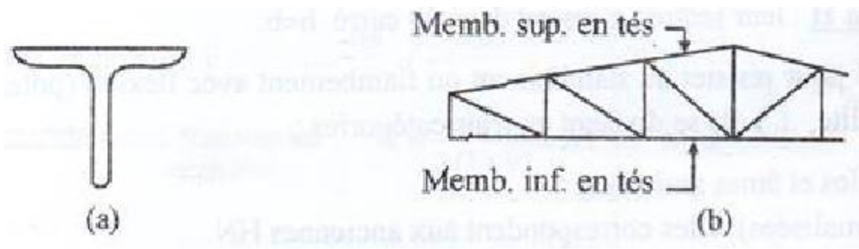


Figure (*1.14*) **T-Beam Profile**

e) Plates and wide flats: Plates are typically used as base plates for beams and columns or as connecting elements in various systems (e.g., lattice beams, bracing). They can also be used to form P.R.S. beams.

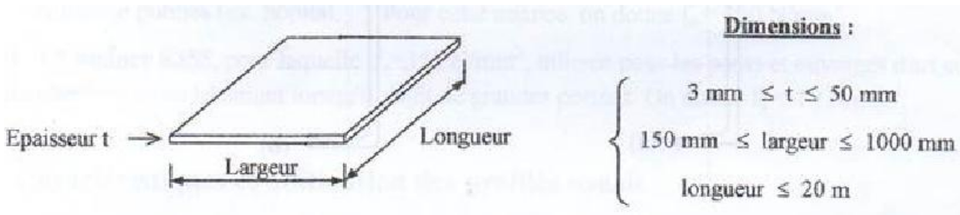


Figure (1.15) **Plates**

f) Hollow Profiles : Hollow profiles are commonly used to form the following elements:

- Columns: Round, square, or rectangular types.
- Lattice Beams: Round or square types.
- Stability Bracing Diagonals: Round types.

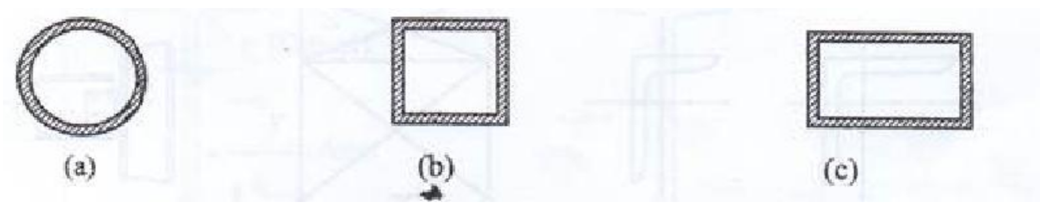


Figure (1.16) **Hollow Profiles**

Fundamentals and Safety Concepts

Chapter 2. Fundamentals and Safety Concepts

2.1 Concept of Safety

All structural design or verification calculations are based on numerous mathematical or physical assumptions, which are generally modeled and entirely theoretical.

These assumptions often deviate significantly from reality due to the multitude of inaccuracies, imperfections, and even errors that can affect calculations, manufacturing, assembly, and the use of the structures in question. These factors are highly variable and inherently random in nature.

This set of inaccuracies and imperfections can affect :

2.1.1 The Design of a Structure

- Underestimation of loads, both permanent and especially variable ones.
- Incorrect assumptions about joint connections (modeled as either perfectly fixed or hinged for calculation purposes, whereas, in reality, a joint is only partially fixed or hinged).
- Poorly designed connections (concepts of rigidity and rotational capacity are often misunderstood or entirely ignored).
- Effects of steel expansion due to thermal dilation not accounted for.
- Excessive deformations under serviceability limit states (deflections f, rotations $\boldsymbol{\theta}$, displacements Δ…etc.).

2.1.2 The Manufacturing of Elements

- The yield strength f_y of steel is not precisely determined.
- Residual stresses from rolling, which are poorly understood, distort the calculations of resulting stresses.
- The modulus of elasticity EEE of steel is not uniform across a section.
- Contrary to assumptions in material strength, steel is not a perfectly elastic, homogeneous, and isotropic material.
- Rolling tolerances are significant and can disrupt inertia and stress calculations by up to 15% with ease.

2.1.3 The Transformation of Components

Errors during factory transformations, whether due to mistakes in execution plans, incorrect dimensions, omitted stiffeners, oversized drill holes, defective welds, or other factors, can significantly impact the final product.

2.1.4 On-Site Assembly

- Calculation methods typically account for structures in their final state, rarely considering the assembly phase, which can lead to various issues: beam lateral-torsional buckling during lifting, collapse due to omitted temporary bracing, etc.
- Improper bolt tightening (especially for HR bolts), non-compliance with bolt diameter and steel grade specifications, insufficient friction coefficient μ of base plates, and similar issues.

2.1.5 Operation by the client may prove harmful

- Changes in the intended use of spaces, leading to significantly higher loads on floors.

- Addition of loads that were not originally planned, such as equipment or furniture.

- Lack of maintenance and upkeep, resulting in steel corrosion, oxidation, and loss of load-bearing section.

Considering, contrary to the assumptions of material strength theories, that loads are never perfectly centered, columns are rarely perfectly vertical, beams are seldom straight, and stresses are not always confined to the principal planes of inertia, it is certain that throughout its lifespan, a structure will be subjected to loads greater than those accounted for in calculations.

Therefore, to ensure the safety of a construction, two approaches are possible :

- The first approach involves calculations based on permissible stresses, where the goal is to verify that the service stress remains below a fraction of the material's ultimate stress.
  This method is a deterministic approach, which assumes that all calculation parameters are known and not subject to variability.

- The second approach uses calculations based on "limit states", where it is necessary to verify that the factored (or weighted) service stress remains below the material's ultimate stress.

This method is a probabilistic approach, incorporating variable weighting factors, and thus allowing for randomness in the parameters.

It appears that the current and future trend in regulations and standards under development is moving towards "semi-probabilistic" methods, as is the case with Eurocode

2.1.6 Limit States

A limit state is a specific condition beyond which a structure no longer meets the requirements for which it was designed and sized.

Two types of limit states are distinguished:

- **Serviceability Limit State (SLS)**, which corresponds to the normal, everyday use of the structure and limits its deformations in order to prevent secondary damages and ensure the proper functioning of the structure (limitation of deflections, concrete cracking, etc.);
- **Ultimate Limit State (ULS)**, which corresponds to an exceptional, ultimate load case (for example: a 30-year snow load, a 100-year flood, etc.), for which the stability of the structure must be ensured, even though it is at the verge of failure. A ULS is reached when there is a loss of equilibrium, a form of instability, a failure of an element, excessive plastic deformation, and so on.

2.2 Regulations

For many years, French regulations concerning the design and calculation of steel structures were widely used in Algeria, specifically the "CM 66 Rules," which governed all steel buildings. These rules, based on the assumption of linear elastic behavior of steel (Hooke's Law: stresses proportional to deformations), allowed for straightforward calculations of the resistance of structural elements.

For bridges and civil engineering structures, Title V of Booklet 61, titled "Design and Calculation of Steel Bridges and Structures", was also applied.

These regulations were later supplemented (in the 1970s) by NF standards, which governed the calculations for connections :

- For riveted connections : Standards NFP 22410 and P22411.

- For connections using standard bolts: Standards NFP 22430 and P22431.
- For connections using preloaded bolts: Standards NFP 22460 and P22469.
- For welded connections: Standards NFP 22470 and P22472.

The development of plasticity theory and the research findings in this field led to the creation of a supplementary document to the previous rules known as "Additive 80" (introduced in the 1980s). This document allowed for the utilization of the elasto-plastic properties of steel, thereby enabling the reduction of structural weight.

Since 1993, with the aim of standardizing the various regulations across the European Union in the field of construction, a new set of regulations came into effect: the Eurocodes, specifically Eurocode 3 for steel structures.

In 1997, Algeria introduced the "CCM 97" as a replacement for the "CM 66" rules. This document incorporated the various sections of Eurocode 3.

2.2.1 Eurocode 3 and the Adopted Calculation Approach

Eurocode 3 has been significantly enriched by experimental and theoretical advancements in steel construction. It is a highly innovative standard, introducing concepts that previously did not exist, such as the classification of cross-sections. It establishes very detailed calculation rules and offers several design alternatives (elastic or plastic calculations, and calculations based on a global structural analysis in the first or second order).

All calculations are designed with a highly safety-oriented approach, supported by the calibration of numerous resistance calculation formulas.

Instead of relying on a single traditional safety factor, Eurocode 3 introduces multiple partial safety factors applied to both loads and resistances.

2.2.2 Algerian Regulation CCM 97

CCM 97 is the Algerian technical regulation that replaces CM 66 as the accepted practical guideline for the calculation of steel structures. Derived from the unified European regulation in this field, Eurocode 3, it incorporates specific recommendations for Algeria, particularly in the areas of seismic design and climatic overloads as outlined in RNVA 99.

2.3 Actions and Calculation Combinations

The actions acting on a structure are categorized into three types:

- **Permanent Actions (G):**
  - Self-weight; prestressing forces.
  - Differential displacement of supports.
  - Imposed deformation on the structure.
- **Variable Actions (Q):**
  - Operational loads.
  - Wind action.
  - Snow action.
  - Effects of thermal gradients.
- **Accidental Actions (A):**
  - This type is rarely considered and only included if specified in the project's consultation terms.
  - Examples: Explosions, vehicle impacts.

2.3.1 Action Combinations at the Ultimate Limit State (U.L.S)

The combinations of actions are as follows:

With a single variable action : $\mathbf{1.35\ G_{max}+G_{min}+1.50Q}$

Where:

- $\mathbf{G_{max}}$: unfavorable permanent action.
- $\mathbf{G_{min}}$: favorable permanent action.
- **Q**: unfavorable variable action.

With multiple variable actions : $\mathbf{1.35\ G_{max}+G_{min}+1.35\sum Q_i}$

2.3.2 Action Combinations at the Service ability Limit State (S.L.S)

These combinations are used exclusively for the calculation or verification of deformations (deflections and displacements).

- With a single variable action : **G+Q**
- With multiple variable actions : **G+0.9∑Qi**

2.3.3 Limit Values for Deformations

The limit values for deformations in steel structures are not strictly and universally regulated, as they depend on various criteria specific to each construction. For instance, installations such as overhead cranes, elevators, or glazed facades require very limited deformations and high structural rigidity to ensure proper functionality. Conversely, a simple warehouse can accommodate significantly greater deformations.

The responsibility for setting these limits lies with the designers, project owners, or end users, who are expected to understand the factors impacting both the construction itself and its intended use or final purpose.

If these preferences are not explicitly specified in the project specifications, Eurocode 3 provides recommended deformation limits, which, although approximate, serve as useful guidelines.

- Roofs in general: $f < L/200$

- Floors in general: $f < L/250$
- Floors supporting columns: $f < L/400$
- Portal frame columns in general: $f < L/300$
- Portal frame columns with overhead cranes : $f < L/500$

2.3.4 Partial Safety Factors

a. Loads

Limit state calculations introduce numerous partial safety factors that connect the characteristic values of loads and resistances to their design values.

Limit state calculations introduce numerous partial safety factors to relate the characteristic values of loads and resistances to their corresponding design values.

Eurocode 3 defines three partial safety factors for actions as follows:

- γ_G : Permanent actions.
- γ_Q : Variable actions.
- γ_A : Accidental actions.

The factor γ_G can take two values: $\gamma_{G,sup}$ and $\gamma_{G,inf}$, representing the "upper" and "lower" values, respectively.

- When permanent actions have an unfavorable effect on the calculation conditions considered, the upper value of the partial safety factor is used.
- Conversely, when the effect of a permanent action is favorable (e.g., in the case of loads applied to a cantilever when considering the adjacent span calculation), the lower value of the partial safety factor is applied.

In principle, the characteristic values (F_k) are multiplied by partial safety factors for actions (γ_F) to obtain the design loads (F_d), as expressed by:

$$\mathbf{F_d = \gamma_F\, F_k} \qquad (1)$$

The consequences of applying design loads to the structure, such as bending moments and shear forces, are referred to as the "design effects" and are denoted by E_d.

The design resistance R_d is calculated by dividing the characteristic resistance R_k by the partial safety factor for the material γ_M.

For the calculation to be satisfactory, the design resistance must be greater than or equal to the "design effect," expressed as:

$$(E_d \leq R_d)$$

b. Partial Safety Factors for Materials

The partial safety factors introduced by Eurocode 3 are denoted by $\boldsymbol{\gamma_M}$. These factors depend on the section class when calculating the resistance of cross-sections.

- $\boldsymbol{\gamma_{m0} = 1.0}$: Applied to account for the resistance of straight sections classified as Class 1, 2, or 3. **($\boldsymbol{\gamma_{M0}}$ = 1,0** is used for elements conforming to NF standards; for others, **$\boldsymbol{\gamma_{M0}}$ = 1,1)** is applied.
- $\boldsymbol{\gamma_{m1} = 1.1}$: Applied to account for the resistance of straight sections classified as Class 4 and resistance to instability.
- $\boldsymbol{\gamma_{m2} = 1.25}$: Applied to account for the resistance of straight sections at bolt hole locations.

For the calculation of metallic elements subject to elastic instability :

$$\left.\begin{array}{l}\text{Buckling}\\ \text{Lateral-Torsional Buckling}\\ \text{Plate Buckling}\end{array}\right\}\ \gamma_{m1} = 1.1$$

For the calculation of connections, $\boldsymbol{\gamma_m}$ takes different values depending on the type of connection and the stress it is subjected to :

Connection with standard bolts:

– Subjected to shear: $\boldsymbol{\gamma_{mb} = 1.25}$

– Subjected to tension: $\boldsymbol{\gamma_{mb} = 1.50}$

Connection with high-strength (HR) bolts:

– At the ultimate limit state: $\boldsymbol{\gamma_{ms} = 1.25}$ or **1.40**, depending on the hole type (standard or slotted).

– At the serviceability limit state: $\boldsymbol{\gamma_{ms} = 1.1}$ for standard tolerance holes.

Connection with welds: γM depends on the steel grade:

– Steel S235 : $\boldsymbol{\gamma_{mW} = 1.25}$

– Steel S275 : $\boldsymbol{\gamma_{mW} = 1.30}$

– Steel S355 : $\boldsymbol{\gamma_{mW} = 1.35}$

2.4 Classification of Cross-Sections

For the calculation of a framework and its components, the designer must select an appropriate structural model. The choice of model determines:

- The structural analysis, aimed at determining internal forces (stresses).
- The calculation of the resistance of cross-sections.

For the ultimate limit state, various combinations of global analysis methods and cross-section calculation methods are available, involving either an elastic approach or a plastic approach. The possible combinations are presented in Table 01.

Table (2.01): *Ultimate Limit State Calculations - Definition of Models*

Type of Modeling	Global Analysis Method (Calculation of Stresses)	Element Calculation (Resistance of Cross-Sections)
I	Plastic	Plastic
II	Elastic	Plastic
III	Elastic	Elastic
IV	Elastic	Elastic Local Buckling

2.4.1 Conditions for the Classification of Cross-Sections

According to Eurocode 3, Table 2 provides a summary of the conditions that cross-sections must satisfy in terms of mechanical behavior, bending resistance capacity, and rotational capacity. The bending resistance moments for the four defined classes are as follows:

- Classes 1 and 2: Plastic moment ($M= W_{pl}.f_{pl\,y}$)
- Class 3: Elastic moment ($M=W_{el}\,.f_{ely}$)
- Class 4: Local buckling moment ($M_o < M_{el}$)

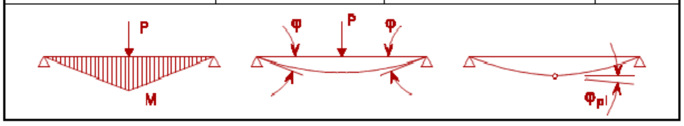
2.4.2 Criteria for the Classification of Cross-Sections

The classification of a cross-section depends on the width-to-thickness ratio (b/t) of each of its compressed plates. The compressed plates to be considered include any plate that is fully or partially compressed by axial forces and/or bending moments present in the section under the given loading condition.

The classification of a cross-section, therefore, partially depends on the type of loading to which the section is subjected.

Table 2.02 *Classification of Cross-Sections*

Behavior Model	Bending Resistance	Rotation Capacity	Class
M, M_{pl}, Local Buckling, φ	f_y, Plastic behavior over the entire section	M/M_{pl}, 1, $\varphi_{rot}/\varphi_{pl}$, 1, φ/φ_{pl}, Significant	1
M, M_{pl}, Local Buckling, φ	f_y, Plastic behavior over the entire section	M/M_{pl}, 1, 1, φ/φ_{pl}, Limited	2
M, M_{pl}, M_{el}, Local Buckling, φ	f_y, Elastic behavior over the entire section	M/M_{pl}, 1, M_{el}/M_{pl}, 1, φ/φ_{pl}, None	3
M, M_{pl}, M_{el}, Local Buckling, φ	f_y, Elastic behavior over an effective section	M/M_{pl}, 1, 1, φ/φ_{pl}, None	4



Class 1 : Cross-sections capable of forming a plastic hinge with the required rotational capacity for plastic analysis.

Class 2 : Cross-sections capable of developing a plastic moment of resistance but with limited rotational capacity.

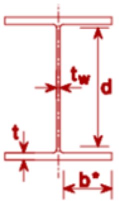
Class 3: Cross-sections where the calculated stress in the extreme compressed fiber of the steel element can reach the yield strength, but local buckling may prevent the development of the plastic moment of resistance.

Class 4 : Cross-sections where the bending or compressive resistance must explicitly account for the effects of local buckling.

2.5 Limiting Slenderness Ratios of Plates

Table 2.3 provides some of the b/t ratio values for the plates of rolled I-section cross-sections under compression or bending.

Table 2.3. *Maximum Slenderness Ratios for Plates of Rolled Cross-Sections under Compression or Bending*

	Wall	Class 1 Cross-section	Class 2 Cross-section	Class 3 Cross-section: Expression	k_σ	b^*/t ou d/t_w
	Flange (1) (b^*/t)	9 ε	10 ε	$21\,\varepsilon\,\sqrt{k_\sigma}$	0,43	14 ε (1)
	Web in compression d/t_w	33 ε	38 ε	$21\,\varepsilon\,\sqrt{k_\sigma}$	1,0	42 ε
	Web in pure bending d/t_w	72 ε	83 ε	$25{,}4\,\varepsilon\,\sqrt{k_\sigma}$	23,9	124 ε
	f_y	235		275		355
$\varepsilon=\sqrt{f_y/235}$	ε	1,0		0,92		0,81

(1) b represents the half-width of the flange, instead of b*.
For this reason, the values provided in the 'Eurocode 3 version' are =15>b=15ε>b*.

Assemblies

Chapter 3. Assemblies

3.1 General Overview

Metal structures consist of a framework made up of bar elements (columns and beams) that are assembled together. Consequently, the joints play a critical role in this type of construction.

A joint is a connecting mechanism that links and secures multiple metallic elements together, ensuring the transmission and distribution of various forces (Nsd, Msd, and Vsd) between the connected elements without inducing unwanted stresses.

In most metal structures, the various types of joints encountered are illustrated in the following figure:

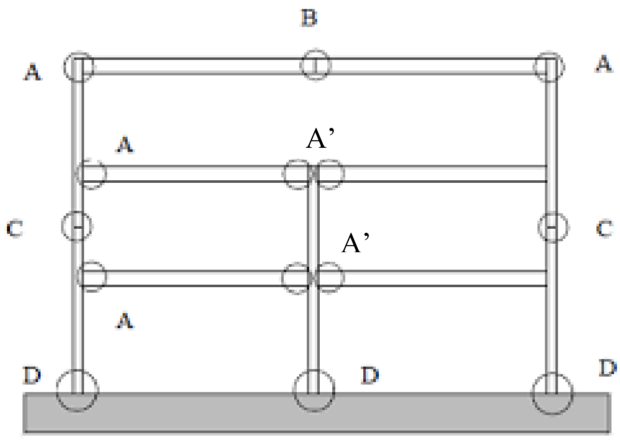


Figure (3.1) *Different Types of Joints in a Metal Structure*

The types of joints include:

- Unilateral beam-column joint configuration (A);
- Bilateral beam-column joint configuration (A');
- Beam continuity joint (B);
- Column continuity joint (C);
- Baseplate joint (D).

3.2 Joint Mechanisms

3.2.1. Mechanism by Obstruction

This applies to ordinary bolts, which are non-preloaded. In this case, the shafts of the bolts bear the forces and operate under shear stress.

3.2.2. Mechanism by Adhesion of Assembled Components

Here, the transfer of forces occurs through the adhesion of the contact surfaces between the assembled components. This mechanism includes welding, bonding, and bolting with high-strength bolts (HR bolts).

3.2.3. Mixed Mechanism

This applies to riveting (and, in extreme cases, high-strength bolts). Rivets initially transmit forces through adhesion between the components up to a certain threshold. Once this threshold is exceeded, the rivets function by obstruction, engaging in shear stress.

3.3 Bolted Joints

3.3.1. Joints with Ordinary Bolts

The load transfer between components is achieved through the bolt shaft, which acts as an obstruction between the assembled components, thereby functioning under shear stress.

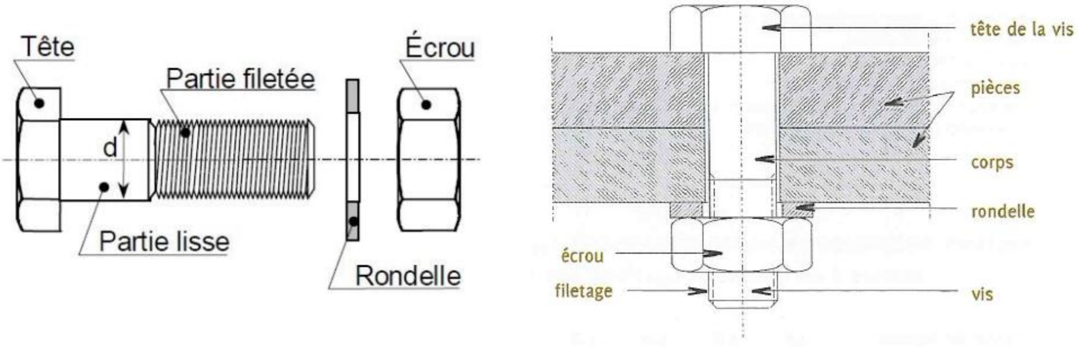


Figure (3.2) *Components of a bolt*

3.3.2. Bolt Layout

The spacing between bolt axes (pitch *p*) and between bolt axes and the edges of the component (edge distance *e*) must adhere to specific limits for the following reasons:

- Minimum values are required to facilitate bolt installation and to prevent sheet failure when the edge distance is too small.
- Maximum values ensure continuous contact between the assembled components (to limit corrosion risks) and prevent overly elongated joints.

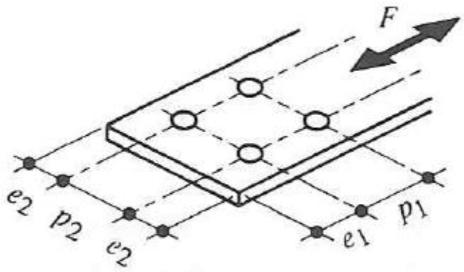


Figure (3.3) *Representation of Pitch and Edge Distances Based on Force Direction*

The arrangement of bolts in a component is regulated by standard **E 3**, depending on the nature of the applied load and the configuration of the holes (in parallel or staggered).

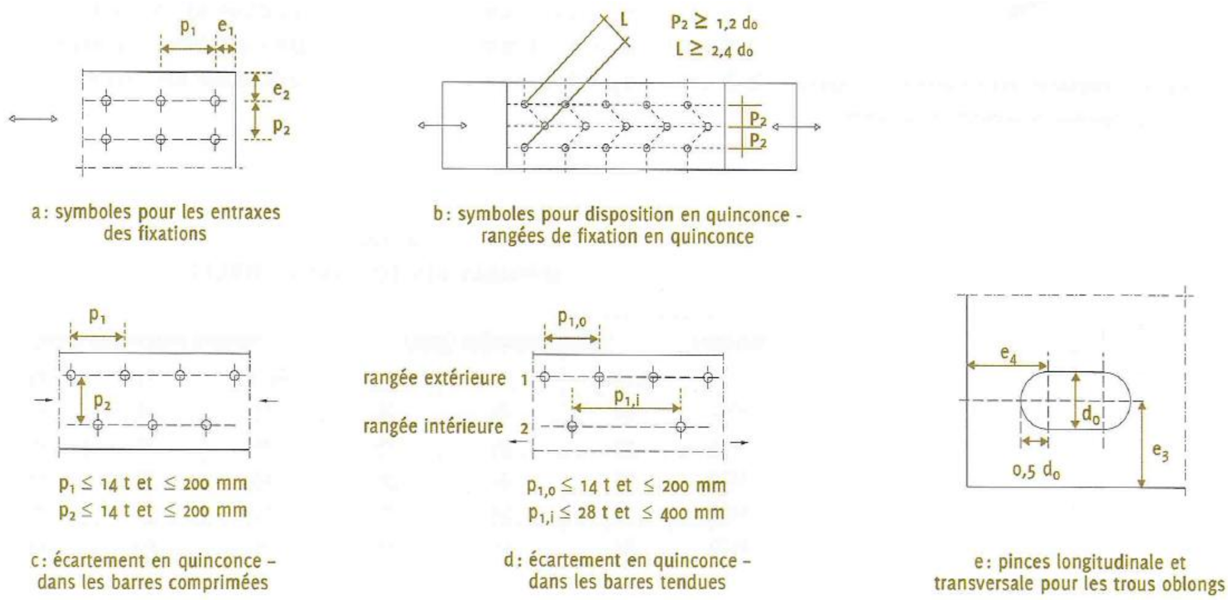


Figure (3.4) *Symbols for Transverse and Longitudinal Edge Distances and Fastener Spacing*

3.3.3. Clearances in Bolt or Rivet Holes

Bolt holes are produced by punching and reaming, or by drilling. They have been standardized according to **C.C.M.97**, depending on the nominal diameter of the bolt and the type of hole provided.

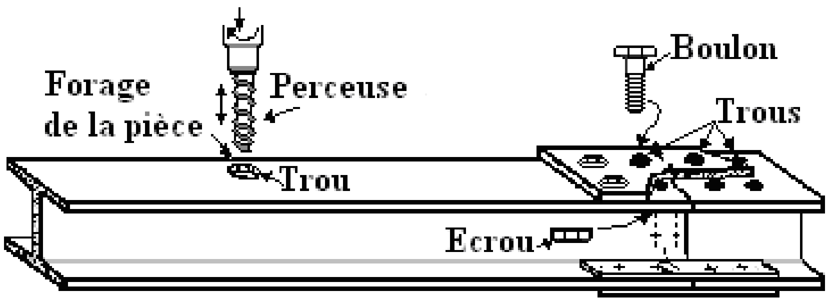


Figure (3.5) Drilling and Bolting of Components

- **Standard holes:** for connections of all categories.
- **Oversized holes**: for connections of categories A, B, and C (shear-loaded connections).
- **Short slotted holes**: for connections of categories A, B, and C.
- **Long slotted holes**: for connections of categories A, B, and C.

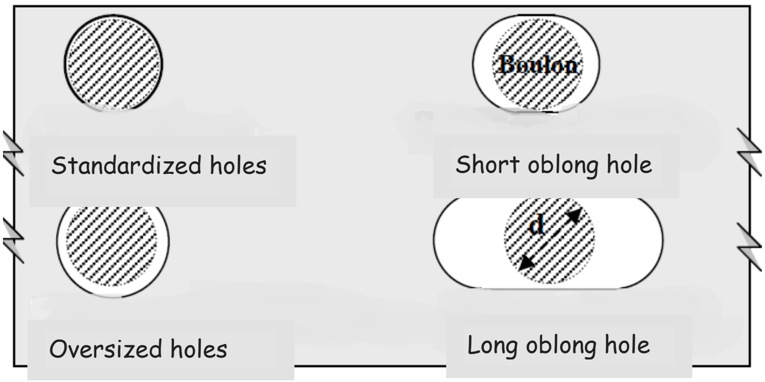


Figure (3.6) Different types of holes for an M12 bolt.

3.3.4. Tensile Resistance Section of Bolts

Several geometric characteristics are essential for the design and verification of bolted joints. For the most commonly used diameters, the main characteristics are provided in the table below.

Table (3.1) *Geometric Characteristics*

Designations	M8	M10	M12	*M14*	M16	*M18*	M20	*M22*	M24
d (mm)	8	10	12	*14*	16	*18*	20	*22*	24
d_0 (mm)	9	11	13	*15*	18	*20*	22	*24*	26
A (mm²)	50,3	78,5	113	*154*	201	*254*	314	*380*	452
A_s (mm²)	36,6	58	84,3	*115*	157	*192*	245	*303*	353
Ø rondelle (mm)	16	20	24	*27*	30	*34*	36	*40*	44
Ø clé (mm)	21	27	31	*51*	51	*51*	58	*58*	58
d_m (mm)	14	18,3	20,5	*23,7*	24,58	*29,1*	32,4	*34,5*	38,8

d: Diameter of the unthreaded part of the bolt.
d_0: Nominal diameter of the hole.
A: Nominal section area of the bolt.
A_s: Tensile section area of the threaded part of the bolt.
d_m: Mean diameter between the circumscribed circle and the inscribed circle at the bolt head.

Note: In italics, the bolts most commonly used are indicated.

3.3.4. Mechanical Properties of Bolts

To prevent the proliferation of bolt types, Eurocode 3 allows only a finite series of mechanical bolt classes. The mechanical properties of bolts necessary for calculations are the yield strength (f_{yb}) and the ultimate tensile strength

(f_{ub}). Each of the seven authorized classes is designated by two numbers (e.g., class 6.8).

Table (3.2) provides the values of f_{yb} and f_{ub} for each class. These values should be adopted as characteristic values in dimensioning calculations.

Preloaded Bolts: Only bolts of classes 8.8 and 10.9 may be used as high-strength preloaded bolts for construction. These are the only ones authorized for use in preloaded bolted joints.

Table (3.2) *Nominal values of yield strength and ultimate tensile strength for bolts.*

Class	4.6	4.8	5.6	5.8	6.8	8.8	10.9	6.6
f_{yb} (Mpa)	240	320	300	400	480	640	900	360
f_{ub} (Mpa)	400	400	500	500	600	800	1000	600

For a given class XY: $f_{yb}=10.X.Y$

$f_{ub} = 100.X$

3.3.5. Assembly Subjected to Shear Forces

In this case, it is necessary to verify:

- On one hand, the shear resistance of the bolts,
- On the other hand, the bearing resistance of the components.

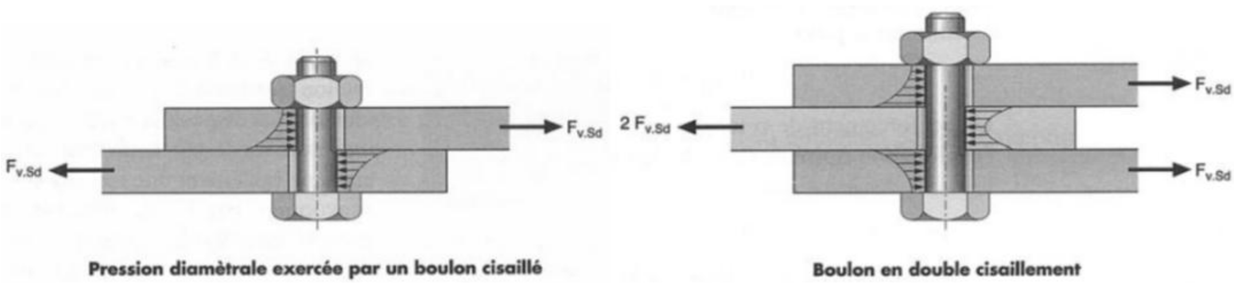


Figure (3.7) *Assembly Subjected to Shear*

a) *Shear Resistance of Bolts per Shear Plane*

– *If the shear plane passes through the threaded portion of the bolt:*

- For the bolt classes : **4.6 ; 5.6 ; 6.6 et 8.8**

$$F_{v,rd} = 0.6 f_{ub} \frac{A_s}{\gamma_{Mb}}$$

- For the bolt classes : **4.8 ; 5.8 ; 6.8 et 10.9**

$$F_{v,rd} = 0.5 f_{ub} \frac{A_s}{\gamma_{Mb}}$$

– *If the shear plane passes through the smooth (unthreaded) portion of the bolt shank:*

$$F_{v,rd} = 0.6 f_{ub} \frac{A}{\gamma_{Mb}}$$

b) *Bearing Resistance of Assembled Components*

At the contact point between the bolt and the component, the bolt applies a pressure to the component, which must have sufficient resistance to prevent ovalization of the holes and tearing of the clamped parts.

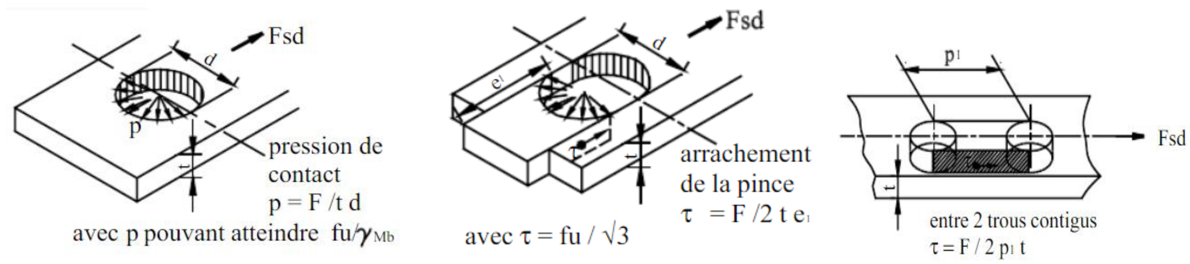


Figure (3.8) *Assembly Subjected to Bearing Pressure*

The resistance is:

$$F_{b,rd} = 2.5\alpha . f_u . \frac{d.t}{\gamma_{Mb}}$$

Where:

α : is the smallest of the following values : $\alpha = min\left\{\frac{e_1}{3d_0}; \frac{P_1}{3d_0} - \frac{1}{4}; \frac{f_{ub}}{f_u}; 1\right\}$

d : diameter of the bolts;

d_0 : diameter of the hole;

f_u : ultimate tensile strength of the component ;

f_{ub} : ultimate tensile strength of the bolt;

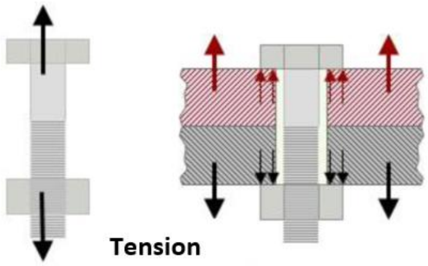
A_s : area of the resistant section in the threaded part ;

A : area of the gross section of the bolt.

γ_{Mb} : partial safety factor (**γ_{Mb} = 1,25**) ;

t : thickness of the assembled component.

c) Tensile Resistance

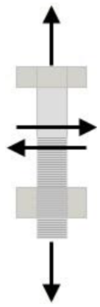


The design tensile strength of a bolt is expressed as :

$$F_{v,rd} = 0.9 f_{ub} \frac{A_s}{\gamma_{Mb}}$$ where : (γ_{Mb} = 1,5)

d) Resistance of Bolts Subjected Simultaneously to Shear and Tension

$$\frac{F_{V,sd}}{F_{V,Rd}} + \frac{F_{t,sd}}{1.4 F_{t,Rd}} \leq 1$$



$F_{V,sd}$: Design shear force,

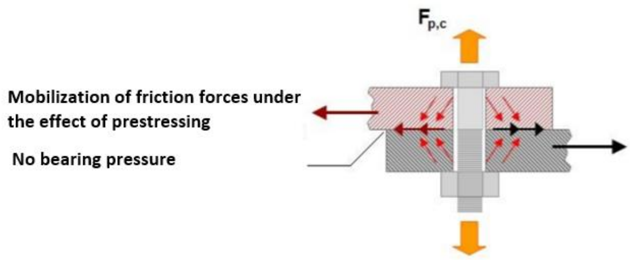
$F_{t,sd}$: Design tensile force,

$F_{V,Rd}$: Shear resistance force,

$F_{t,Rd}$: Tensile resistance force.

3.4 Assembly Using Preloaded Bolts

Although similar in appearance to an ordinary bolt, an HS (high-strength) bolt is made of high-yield-strength steel of grade 8.8 or 10.9 and includes a washer integrated into the head. During bolting, it is tightened strongly, which applies a preload force acting along the bolt axis. This preload generates, through mutual friction between the connected parts, a high resistance to their relative slipping. Unlike ordinary bolts, HS bolts do not work in shear; instead, they transmit forces through friction.



However, in some cases, the rods may come into contact with the parts, either due to improper assembly or accidentally due to sliding of the parts (insufficient friction coefficient μ or excessive tangential force). In such cases, HS bolts will act in shear.

3.4.1. Mechanical Properties of Preloaded Bolts

There are two classes of HS bolts, defined by their yield stress f_{yb} and ultimate tensile strength f_{ub}:

- HS 1 bolts or HS 10.9
- HS 2 bolts or HS 8.8

Class Designations:

- The first digit corresponds to $f_{ub}/100$
- The second digit corresponds to $10\times f_{yb}/f_{ub}$

Table (3.3) *Main Mechanical Properties of HS Bolts*

Reference	Designation	f_{ub} (MPa)	f_{yb} (MPa)
HS 1	HS 10.9	1000	900
HS 2	HS 8.8	800	640

3.4.2. Slip resistance of preloaded bolts (HS)

The design slip resistance of a prestressed bolt is:

$$F_{s,Rd} = \frac{k_s . n . \mu . F_{p,c}}{\gamma_{m3}}$$

Where :

n: Number of friction surfaces.

$\boldsymbol{\gamma_{m3}}$: Safety factor, equal to :

At the ultimate limit state (ULS) :

1.25 : for holes with normal tolerances, as well as for slotted holes whose long axis is perpendicular to the direction of the force.
1.40 : for oversized holes, as well as for slotted holes whose long axis is parallel to the direction of the force.

At the serviceability limit state (SLS)

1.10 : for holes with normal tolerances, as well as for slotted holes whose long axis is perpendicular to the direction of the force.

$\boldsymbol{F_{p,c}}$: Preload force, expressed as: $F_{p,c} = 0.7 \times f_{ub} \times A_s$

$\mathbf{k_s}$: Coefficient depending on the size of the bolt hole, given in the following table:

Table (3.4) *Values of the coefficient K_s*

Description	K_s
Bolts used in standard holes.	**1.0**
Bolts used either in oversized holes or in short slotted holes whose longitudinal axis is perpendicular to the direction of the forces.	**0.85**
Bolts used in long slotted holes whose longitudinal axis is perpendicular to the direction of the forces.	**0.7**
Bolts used in short slotted holes whose longitudinal axis is parallel to the direction of the forces.	**0.76**
Bolts used in long slotted holes whose longitudinal axis is parallel to the direction of the forces.	**0.63**

ks = 1.0 for holes with normal tolerances, namely:

1 mm for bolts φ12 and φ14

2 mm for bolts φ16 and φ 24

3 mm for bolts φ 27 and above.

μ : friction coefficient, which takes the following values :

- **μ = 0.50** for surfaces cleaned by shot blasting or sandblasting and then metallized by aluminum spraying or a zinc-based coating.

• **μ = 0.40** for surfaces cleaned by shot blasting or sandblasting and coated with an alkaline zinc silicate paint layer with a thickness of 50 to 80 μm.

• **μ = 0.30** for surfaces cleaned by flame or brushed with a wire brush, removing all non-adherent rust flakes.

• **μ = 0.20** for untreated surfaces.

3.4.3. Tensile Strength of HS Bolts

For a force applied parallel to the bolt axis, an HS bolt connection is designed so that the joined components remain in contact, provided that the applied force does not exceed the bolt's pre-tension.

3.4.4. Strength of HS Bolts Subjected Simultaneously to Shear and Tension

When bolts are subjected simultaneously to shear and tensile forces, the applied force can be resolved into components perpendicular and parallel to the bolt axis. The axial component reduces the clamping pressure on the connected members. Consequently, the slip resistance per bolt should be taken as the following value :

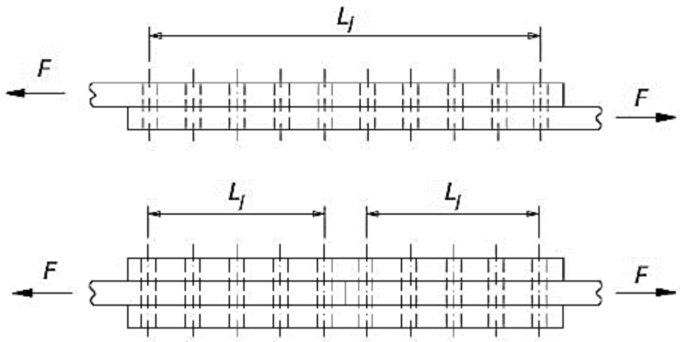
$$F_{s,Rd} = \frac{k_s.\eta.\mu\left(F_{p,c} - 0.8F_t\right)}{\gamma_{m3}}$$

3.4.5. Effect of Joint Length:

Studies on shear-loaded connections have highlighted the influence of the connection length on the ultimate load. The largest deformations occur at the ends of the joint ; therefore, the end bolts are more heavily loaded than the bolts located near the center. Not all bolts within a shear-loaded connection transmit the same force.

When checking a connection in which the distance between the first and the last bolt in a plate exceeds 15 times the bolt diameter *d*, the design shear resistance $F_{v,Rd}$ of all fasteners must be reduced by multiplying it by a reduction factor β_{Lf} given by:

$$\beta_{Lf} = 1 - \frac{L_j - 15d}{200d} \qquad \text{where} \qquad 0.75 \leq \beta_{Lf} \leq 1.0$$



3.5 Welded Assemblies

Welding is a process used to join parts by creating a strong bond in the material, either through fusion or plastic deformation.

Compared to bolted connections, welding offers several advantages:

- It provides continuous material, ensuring effective transmission of loads;
- It eliminates the need for secondary components such as gussets or brackets;
- It results in more compact and aesthetically pleasing assemblies than bolting.

However, welding also has some drawbacks :

- The base metal must be suitable for welding;
- Inspection of welds is necessary and can be costly;
- The quality of welders' work may vary;
- Welding requires skilled labor and specialized equipment.

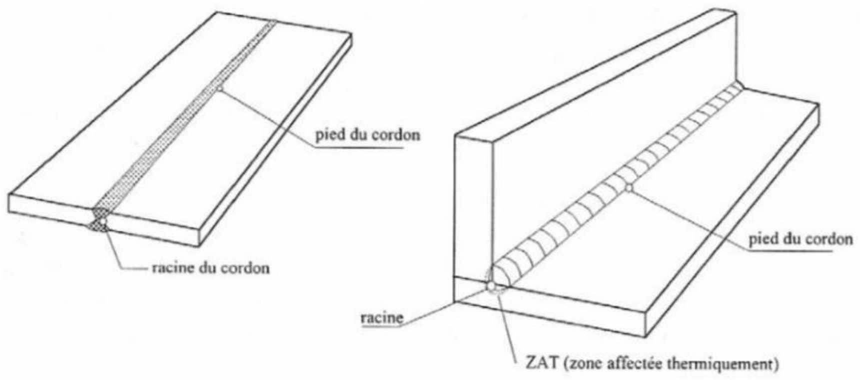


Figure (3.9) *Connection of two parts by welding*

3.5.1. Design Provisions

It is generally distinguished:

- **Butt welds**: either full penetration or partial penetration. Parts with a significant thickness t >6 mm must be beveled.
- **Fillet welds:** the fillets can be flat and/or convex on the outside or inside.

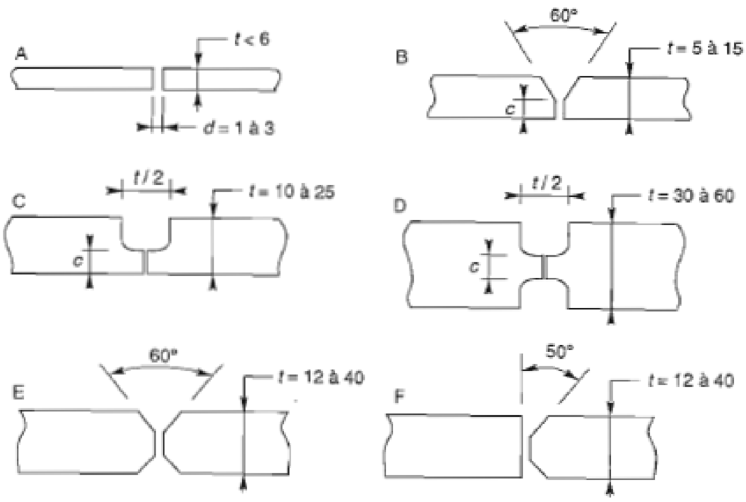


Figure (3.10) *Butt Weld*

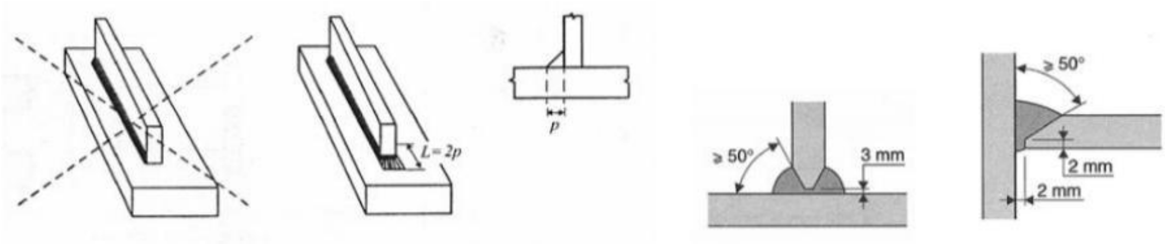


Figure (3.11) *Fillet Weld*

Fillet welds should not be stopped at the ends of the assembled elements. They must wrap around the corners of the joints without interruption, maintaining their size over a length equal to twice the leg of the weld, wherever such continuity is possible within the same plane.

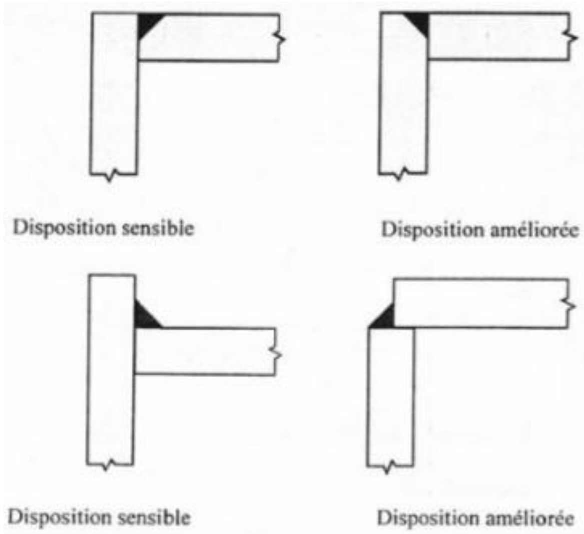


Figure (3.12) *Construction Details to Prevent Lamellar Tearing*

- **Other types of welded joints :**

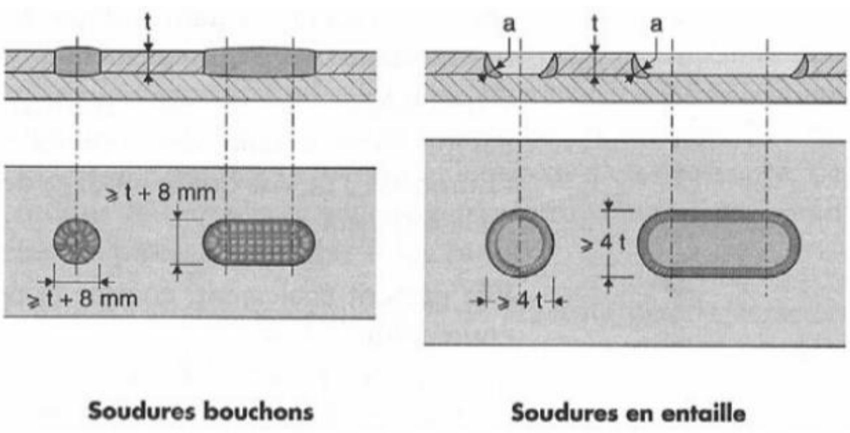


Figure (3.13) *Other Types of Welded Joints*

3.5.2. Calculation of Weld Seam Strength

a) Preliminary Sizing of the Fillet

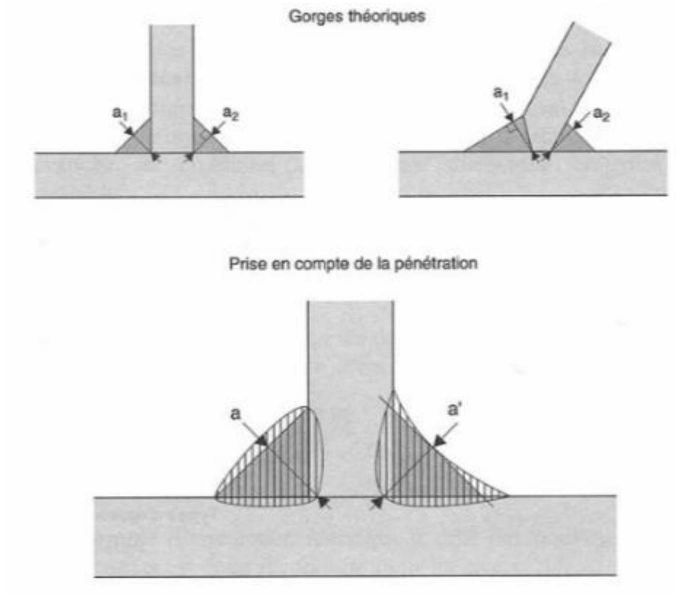


Figure (3.14) *Definition of the Throat of a Fillet Weld*

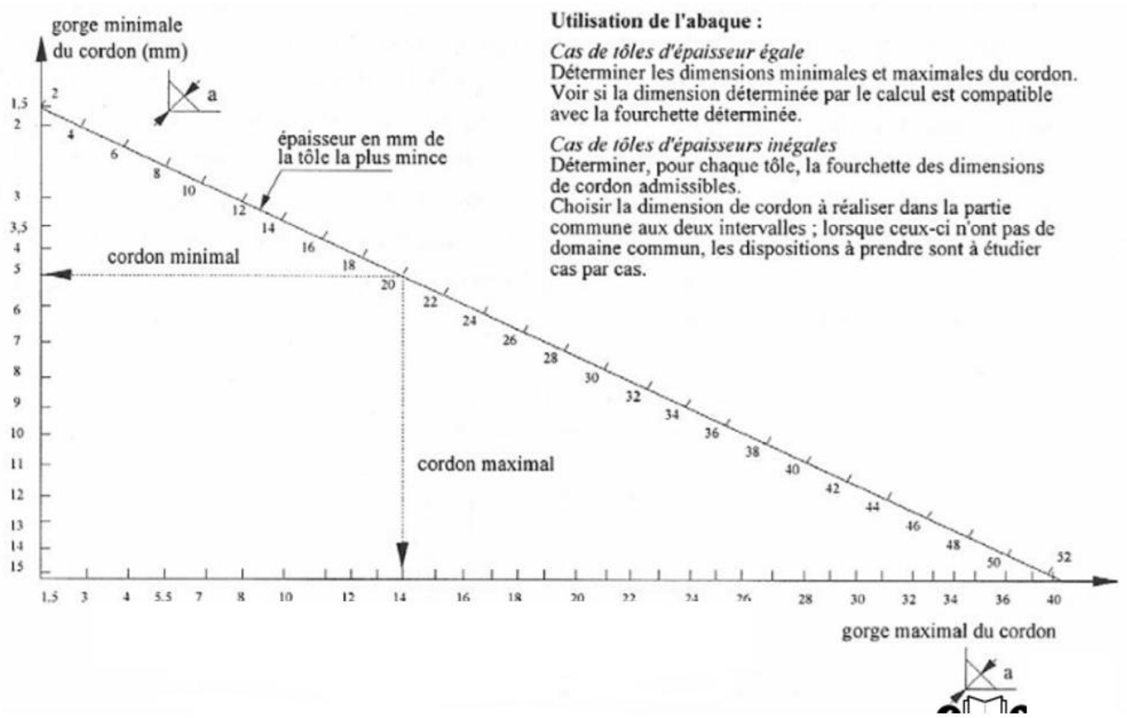


Figure (3.15) *Chart for Preliminary Sizing of the Throat "a"*

b) Butt weld

This type of weld is not calculated. It is assumed that there is material continuity, and therefore continuity of the parts, under the two following conditions: the weld thickness must be at least equal to the thickness of the thinner of the assembled parts, and the filler metal must have mechanical properties at least equal to those of the base metal.

c) Fillet weld

A fillet weld is characterized by:

- A **length L** equal to the length of the assembled part ;
- A **throat thickness a**, which is the minimum distance from the root to the surface of the weld, corresponding to the largest triangle inscribed within the weld thickness.

Forces in a fillet weld are transmitted from one part to another through the weld, passing through the throat section with a minimum area equal to (**a.L**).

The fillet weld is considered sufficient if the following condition is satisfied:

$$\sqrt{\sigma^2 + 3\left(\tau_{\perp}^2 + \tau_{\parallel}^2\right)} \leq \frac{f_u}{\beta_w \gamma_{Mw}} \qquad \text{(fundamental equation)}$$

$$\sigma \leq \frac{f_u}{\gamma_{Mw}}$$

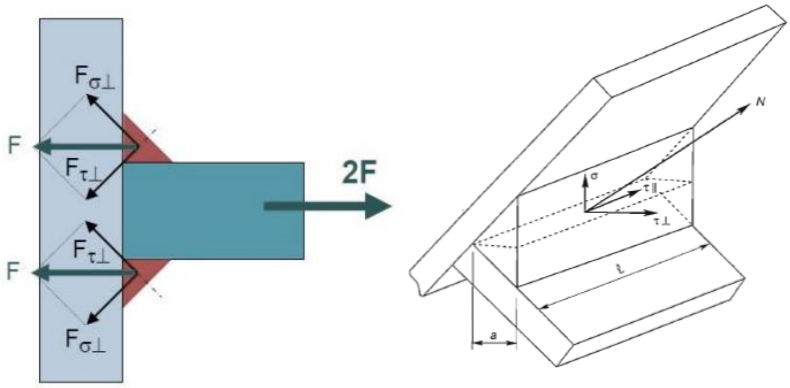


Figure (3.16) *Stress State in the Throat Section.*

N: the applied resultant force on each weld, assumed to be centered along the middle of the weld length.

The normal and shear stresses resulting from the applied force are decomposed along the transverse and longitudinal directions of the weld as follows:

- $\sigma_{\perp}$ **:** component perpendicular to the throat section;
- $\tau_{\perp}$ **:** component of the shear stress in the throat plane and perpendicular to the longitudinal axis of the weld;
- $\tau_{\parallel}$:component of the shear stress in the throat plane and parallel to the longitudinal axis of the weld.

Table (3.5) *Coefficients β_{Mw} and γ_{Mw} Based on Steel Grade*

Steel Grades		γ_{Mw}	β_w	$\beta_w . \gamma_{Mw}$
f_y (MPa)	f_u (MPa)			
235	360	1.25	0.8	1
275	430	1.3	0.85	1.1
355	510	1.35	0.9	1.2

c) Calculation of Weld Beads Connecting Orthogonal Components

- *Frontal Welds*: In this type of weld, the applied force is perpendicular to the longitudinal axis of the weld.

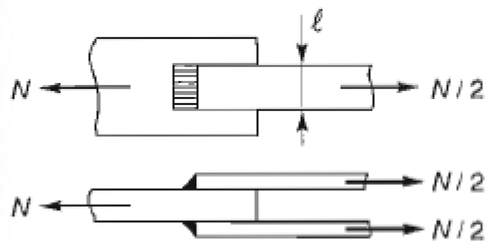


Figure (3.17) *Assembly with Frontal Welds*

The stress state in this case is simplified and becomes:

$$N_{\parallel} = \frac{N\sqrt{2}}{2} \qquad \text{From which} \qquad \sigma = \frac{N_{\parallel}}{a\sum L} = \frac{N\sqrt{2}}{2a\sum L}$$

$$N_{\perp} = \frac{N\sqrt{2}}{2} \qquad \text{From which} \qquad \sigma = \frac{N_{\perp}}{a\sum L} = \frac{N\sqrt{2}}{2a\sum L}$$

$N_{\parallel} = 0$ hence $\tau_{\parallel} = 0$; and the fundamental equation is written by replacing σ and τ with their respective values.

$$\sqrt{\frac{2N^2}{4a^2(\sum l)^2} + \frac{2N^2}{4a^2(\sum l)^2}} \leq \frac{f_u}{\gamma_{Mw}.\beta_w}$$

Thus : $$a\sum l \geq \beta_w.\gamma_{Mw}.\frac{N\sqrt{2}}{2}$$

- *Lateral Weld Beads:* When the applied force is parallel to the longitudinal axis of the weld bead, the stress state becomes:

$$\sigma = \tau_{\perp} = 0 \text{ and } \tau_{\parallel} \neq 0 \text{ ; From which } \tau_{\parallel} = \frac{N_{\perp}}{a\sum L}$$

And the fundamental equation is expressed as:

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

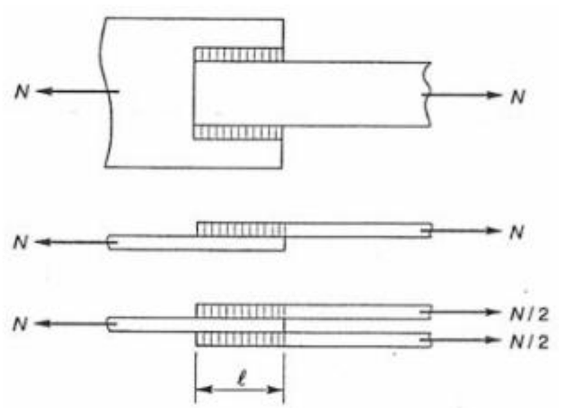


Figure (3.18) *Assembly with Lateral Weld Beads*

- *Oblique Weld Beads*: For this type of assembly, either the applied force is at an angle, or the weld bead itself is oblique.

The stress state is:

$$\sigma = \tau_{\perp} = \frac{N.\,sin\alpha}{\sqrt{2}.\,a.\sum L}$$

And $$\tau_{\parallel} = \frac{N.sin\alpha}{a.\sum L}$$

The fundamental equation is expressed by substituting the stress components with their respective values:

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

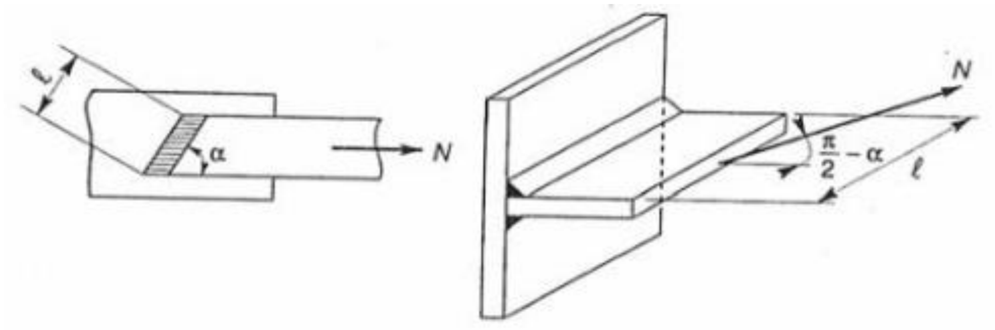


Figure (3.19) *Assembly with Oblique Weld Beads*

3.6 Applications

3.6.1. Application 01 (ordinary bolts)

Consider a lap joint between two plates (a single shear plane passes through the threaded part) made with three ordinary bolts of grade **(4.6)** en boulonnage ordinaire.

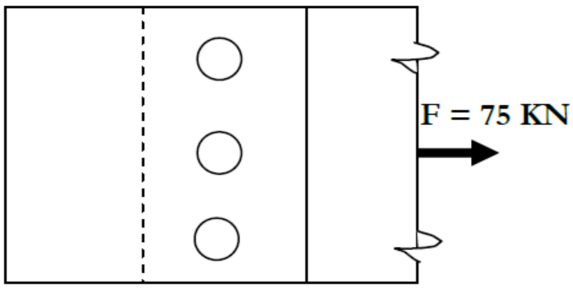
Given: Steel **S235**, **e_1 = 3.d_0** ; edge distance : **t = d_0 +2 mm**

Required:

1° What is the primary load acting on this joint ?

2° Determine the required bolt diameter.

3° Perform the necessary verifications.



Solution :

1° The bolts are loaded in shear.

2° Calculation of the bolt diameter :

The shear force carried by each bolt :

$$F_{vsd} = \frac{F}{n_B} = \frac{75}{3} = 25\ KN$$

For a single shear plane, we have:

$$F_{vsd} \leq F_{vRd} = \beta f_{ub} \frac{A_b}{\gamma_{mb}} \text{..............................(01)}$$

Given:

Bolt grade 4.6 → β=0.6 et f_{ub}=400 Mpa

Shear plane passes through threaded part → A_b=A_s

Bolts subjected to shear → γ_{mb}=1.25

$$25000 \leq 0.6x400\frac{A_s}{1.25}$$

$$A_s \geq 130.208\, mm^2$$

Choose

M16 bolts → A_s=157 mm^2

→ d=16 mm

→ d_0=18 mm

→ t= d_0+2mm=20 mm

3° Verification of bearing (diametral) stress:

$$F_{vsd} \leq F_{b,Rd} = 2.5\ \alpha f_u \frac{dt}{\gamma_{mb}}$$

α = minimum value$\left\{\frac{e_1}{3d_0}, \frac{P_1}{3d_0} - \frac{1}{4}, \frac{f_{ub}}{f_u}, 1\right\}$ with: $e_1 = 3.d_0$=54 mm

α = minimum value$\{1, 1.11, 1\} \rightarrow \alpha = 1$

$$F_{b,Rd} = 2.5x360\frac{16X20}{1.25x10^3} = 230.4\ KN$$

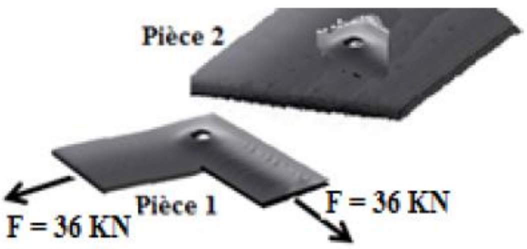
$$F_{vsd} = 25\ KN < F_{b,Rd} = 230.4\ \text{KN (OK)}$$

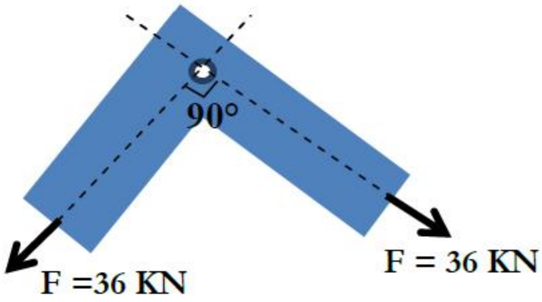
The assembled plates resist the diametral bearing stress.

3.6.2. Application 02 (Ordinary bolts)

Consider a lap joint between two plates (fastening plate 1 to plate 2) using a grade 6.8 bolt, as shown in the figure below.

Determine the required bolt diameter. (*The verification of the connected plates is not required*)



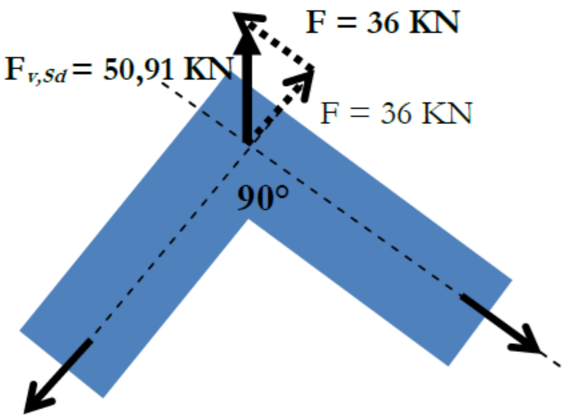


Solution :

1° Calculation of the force applied to the bolt:

$$F_{vsd} = \sqrt{F^2 + F^2} = \sqrt{36^2 + 36^2}$$

$$F_{vsd} = 50.91KN$$



2° Calculation of the bolt diameter:

For a single shear plane, we have:

For bolt classes **6.8** :

$$F_{vRd} = 0.5\mathrm{f_{ub}}\frac{\mathrm{A_s}}{\gamma_{\mathrm{mb}}}$$

$$F_{vsd} \leq F_{vRd}$$

$$F_{vsd} \leq 0.5\mathrm{f_{ub}}\frac{\mathrm{A_s}}{\gamma_{\mathrm{mb}}}$$

$$\mathrm{A_s} \geq \gamma_{\mathrm{mb}}\frac{F_{vsd}}{0.5\mathrm{f_{ub}}}$$

$$\mathrm{A_s} \geq 1.25\frac{50910}{0.5\mathrm{x}600}$$

$A_s \geq \mathbf{212,125\ mm^2}$ An **M20** bolt is adopted (As = 245 mm²).

3.6.3. Application 03 (Preloaded bolts)

Consider a preloaded bolted joint between two tensioned plates. The joint is made of grade 8.8 bolts and transmits a force at the ultimate limit state (ULS) N_{sd} = 187,68 KN. Between two plates, each having a thickness t=15 mm.

– The surface treatment is of class B.

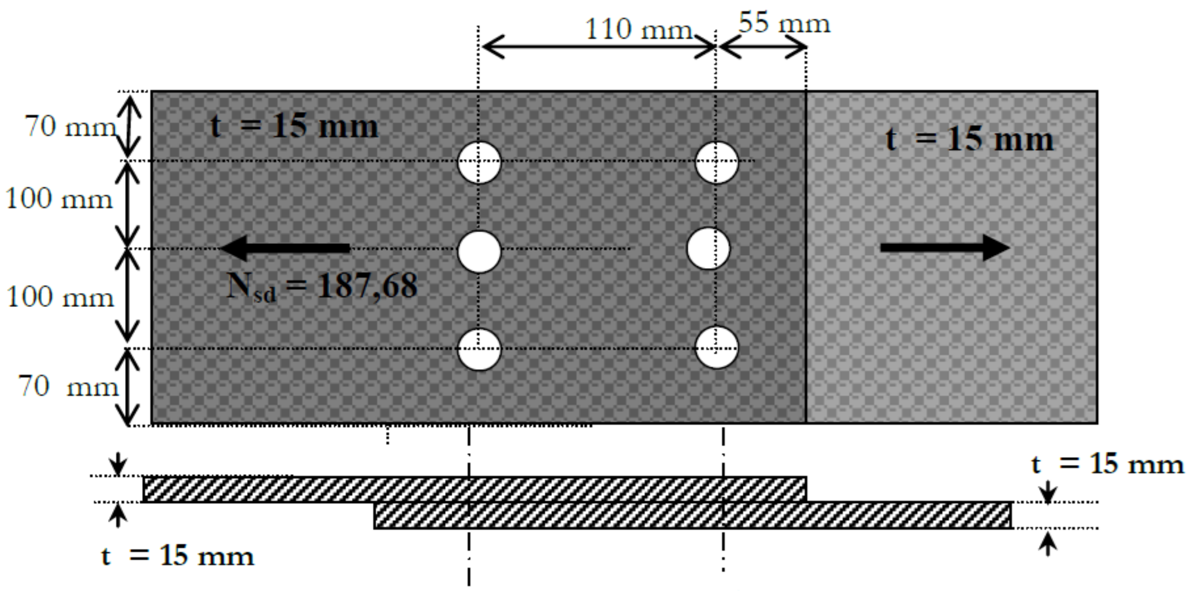
– The bolt holes are oversized.

– The steel grade of the plates is S235.

Required:

1° Determine the required bolt diameter.

2° Verify the resistance of the connected plates.



Solution :

1° Shear force carried by each bolt:

$F_{Vsd} = \frac{N_{sd}}{n_B} = \frac{187.68}{6} = 31.28\ KN$ (n_B=number of bolts)

For a single shear plane, we have :

$$F_{vsd} \leq F_{s,Rd} = \frac{k_s.\mu.F_{p,Cd}}{\gamma_{Ms}}$$

Where:

$\mathbf{F_{p,Cd}}$: preload force, calculated as:

$$F_{p,Cd} = 0{,}7.f_{ub}.A_s$$

Coefficient of friction **(surface treatment class B) → μ=0.4**.

Bolt holes are oversized → **ks=0.85**

Partial safety factor for bolts in shear : $\boldsymbol{\gamma_{Ms}=1.4}$

Number of slip planes : **n=1**

$$F_{v,sd} \leq \frac{0{,}7.k_s.n.\mu.f_{ub}.A_s}{\gamma_{Ms}}$$

$$A_s \geq \frac{F_{v,sd}.\gamma_{Ms}}{0{,}7.k_s.n.\mu.f_{ub}}$$

$$A_s \geq \frac{31280.1{,}4}{0{,}7.0{,}85.1.0{,}4.800}$$

$$A_s \geq 230{,}00\ mm^2$$

2° Verification of members subjected to bearing (diametral pressure):

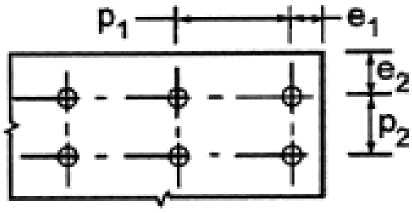
For a single shear plane, the following condition shall be satisfied:

$$F_{vsd} \leq F_{b,Rd}$$

$$F_{b,Rd} = \frac{2{,}5.\alpha.f_u.d.t}{\gamma_{Mb}}\ ; \qquad (\gamma_{Mb}=1.25)$$

For **M20 bolts**, the **hole diameter** is taken from **Table 3.1**.

$d_0 = d + 2 = 20 + 2 = 22$ mm ; $e_1 = 55$ mm, $p_1 = 110$ mm ; $t = 15$ mm.



With :

$$\alpha = \min\left\{\frac{e_1}{3d_0}, \frac{P_1}{3d_0} - \frac{1}{4}, \frac{f_{ub}}{f_u}, 1\right\}$$

$$\alpha = \min\left\{\frac{55}{3.22}, \frac{100}{3.33} - \frac{1}{4}, \frac{800}{360}, 1\right\}$$

$$\alpha = \min\{0.833; 1.417, 1.86, 1\}$$

$$\alpha = 0.833 .$$

$$F_{b,Rd} = \frac{2{,}5.0{,}833.360.20.15}{1.25} = 215000 \text{ N} = 215 \text{ KN (OK)}$$

The connected members resist the bearing (diametral) pressure.

3.6.4. Application 04 (Preloaded bolts)

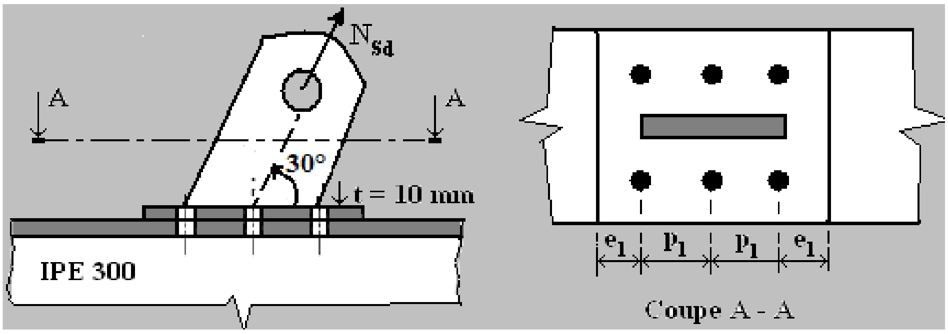
Consider a fixing loop transmitting a tensile design force N_{Sd} inclined at an angle of 30° with respect to the top flange of an IPE main beam.

The preloaded joint of this device is ensured by 6 M20 bolts of grade 8.8.

- The steel grade of the plate is S235.
- The bolted connection is verified at the Ultimate Limit State (ULS).
- Between the two connected parts, the contact surfaces have been treated to class A.
- The holes are standard (normalised).

Required:

Determine the design tensile force N_{Sd} that this loop can withstand. (Verification of the connected plate is not required).



Solution :

1° This connection is of Category C; therefore, for each bolt, the following condition applies:

$$F_{vsd} \leq F_{s,Rd} = \frac{k_s.\mu.(F_{p,Cd} - 0{,}8F_{t,sd})}{\gamma_{Ms}}$$

Where:

$F_{p,Cd}$: design preload force, given by $\mathbf{F_{p,Cd}=0.7 \cdot f_{ub} \cdot A_s}$

Normal force carried by each bolt: $\boldsymbol{F_{sd}} = \frac{\boldsymbol{N_{sd}}}{\boldsymbol{n_B}}$

Coefficient of friction (surface treatment class A): **μ=0.5**

Hole type : standard holes, hence **: k_s=1**

Partial safety factor : $\boldsymbol{\gamma_{Ms} = 1{,}25}$

Number of slip planes : **n=1**

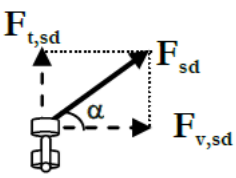
$$F_{vsd} \leq F_{s,Rd} = \frac{k_s.\mu.0{,}7.n.f_{ub}.A_S}{\gamma_{Ms}} - \frac{0{,}8.k_s.\mu.n.F_{t,sd}}{\gamma_{Ms}}$$

$$F_{vsd} + \frac{0{,}8.k_s.\mu.n.F_{t,sd}}{\gamma_{Ms}} \leq \frac{k_s.\mu.0{,}7.n.f_{ub}.A_S}{\gamma_{Ms}}$$

We know that :

Ft,sd = F_{sd} x sin 30°

Fv,sd = F_{sd} x cos 30°



$$F_{sd}.[\cos(30°) + \frac{0{,}8.k_s.\mu.n.F_{sd}\ \mathrm{x}\sin(30°)}{\gamma_{Ms}}] \leq \frac{k_s.\mu.0{,}7.n.f_{ub}.A_S}{\gamma_{Ms}}$$

$$F_{sd}.[0,866+\frac{0,8.1.0,5.1\ \text{x}\sin(30°)}{1,25}]\leq\frac{k_s.0,5.0,7.1.800.245}{1,25}$$

$F_{v,sd}x1,116 \leq 54880 \Rightarrow F_{sd} = 49175,63N$

Hence : $N_{sd} \leq n_b.F_{sd} \Rightarrow N_{sd} \leq 6X49175 \Rightarrow N_{sd} =$ 295053,76 N ;

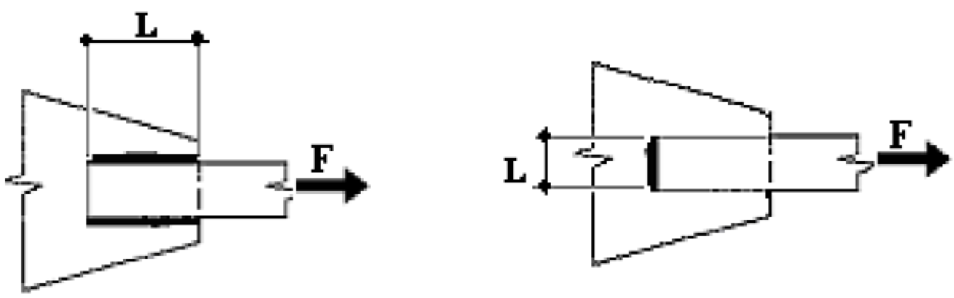
Hence, at the limit : $N_{sd} = 295$ KN

3.6.4. Application 04 (Welded Connections)

Consider a welded connection between two steel plates. The connection is made by a weld seam transmitting an ultimate limit state force of F=80 KN between the two plates. The length of the weld seam is: L=100 mm.

- The steel grade of the plates is S235.
- The filler metal grade is the same as that of the plates (S235).
- The throat thickness of the weld is: a = 5 mm.

Verify the resistance of this welded connection.



Solution :

F = 250 KN

Throat thickness of the weld: a=5 mm

Length of one weld seam: L=100 mm

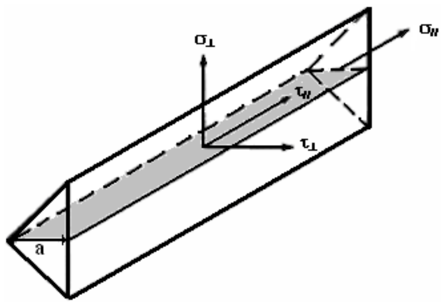
Design conditions:

$a \geq 3\ mm$ and $L > \min(6.a = 6X5 = 30\ mm; 40\ mm)$.

Steel graded S235 $\Rightarrow f_u = 360\ MPa$; $\beta_w = 0{,}8$; $\gamma_{mw} = 1{,}25$

The strength of the fillet weld is sufficient if the following two conditions are met :

$$\begin{cases} \sqrt{\sigma_\perp{}^2 + 3\left(\tau_\perp^2 + \tau_\parallel^2\right)} \leq \frac{f_u}{\beta_w \gamma_{Mw}} \\ \sigma_\perp \leq \frac{f_u}{\gamma_{Mw}} \end{cases} \quad \text{...............................(01)}$$



Case 1: Two cords are subjected to lateral stress : $\sigma_\perp = 0$ and $\tau_\perp = 0$

At the same time, the cords are subjected to stress : $\tau_\parallel = \frac{F}{a.2L}$............(2)

(1)$\Rightarrow \sqrt{3}.\tau_\parallel \leq \frac{f_u}{\beta_w \gamma_{Mw}} \Rightarrow \tau_\parallel \leq \frac{f_u}{\sqrt{3}.\beta_w \gamma_{Mw}}$

(2)$\Rightarrow F \leq \frac{f_u.a.2L}{\sqrt{F}.\beta_w.\gamma_{Mw}}$

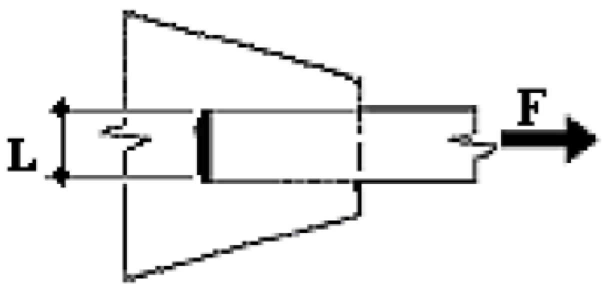
$$\Rightarrow F \leq \frac{360.5.2.100}{\sqrt{F}.\,0{,}8.1{,}25}$$

$\Rightarrow F \leq \mathbf{332553{,}75\ N}$

Hence: $F = 30\ KN < \mathbf{332{,}553\ KN}$

The weld seams are able to resist the applied load.

2nd case : This is a front weld bead $\Rightarrow \tau_\parallel = 0$



Thus, the beads are subject ed to :

$$\sigma_{\perp} = \frac{F}{\sqrt{2}.a.L} \text{ and } \tau_{\perp} = \frac{F}{\sqrt{2}.a.L}$$

$$(1) \Rightarrow \begin{cases} \sqrt{\sigma_{\perp}^2 + 3(\tau_{\perp}^2)} \leq \frac{f_u}{\beta_w \gamma_{Mw}} \\ \sigma_{\perp} \leq \frac{f_u}{\gamma_{Mw}} \end{cases} \Rightarrow \begin{cases} \sqrt{(\frac{F}{\sqrt{2}.a.L})^2 + 3(\frac{F}{\sqrt{2}.a.L})^2} \leq \frac{f_u}{\beta_w \gamma_{Mw}} \\ \frac{F}{\sqrt{2}.a.L} \leq \frac{f_u}{\gamma_{Mw}} \end{cases}$$

$$\Rightarrow \begin{cases} \frac{\sqrt{2}.F}{a.L} \leq \frac{f_u}{\beta_w \gamma_{Mw}} \\ \frac{F}{\sqrt{2}.a.L} \leq \frac{f_u}{\gamma_{Mw}} \end{cases}$$

$$\Rightarrow \begin{cases} F \leq \frac{a.L.f_u}{\sqrt{2}.\beta_w \gamma_{Mw}} \\ F \leq \frac{\sqrt{2}.a.L.f_u}{\gamma_{Mw}} \end{cases}$$

$$\Rightarrow \begin{cases} F \leq \frac{5.100.360}{\sqrt{2}.0,8.1,25} \\ F \leq \frac{\sqrt{2}.5.100.360}{1,25} \end{cases}$$

$$\Rightarrow \begin{cases} F \leq \text{N } 127279,22 \\ F \leq \text{N } 203646,75 \end{cases}$$

F ≤ Min (127279,22 ; 203646,75) N⇒ F ≤127,279 KN

Hence : $F = 30\ KN < 127{,}279$ KN

The butt weld bead resists the applied load.

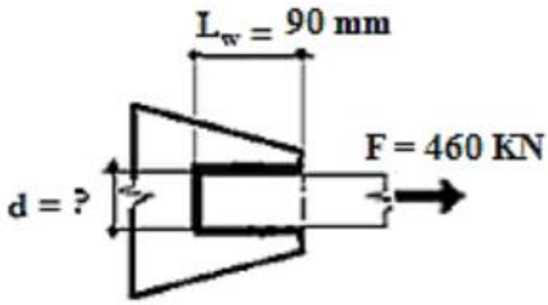
3.6.5. Application 05 (Welded Connections)

Consider a lap joint welded assembly of two plates. It transmits an ultimate limit state force **F = 460 KN** between the two plates.

- The length of each horizontal weld bead: **L_w = 90 mm**
- The throat thickness of all welds: **a = 8 mm**
- The steel grade of the plates and the filler metal is the same: **S235 (Fe360)**

1° Calculate the force **F**$_{\parallel}$ that can be carried by the horizontal weld beads.

2° Calculate the minimum length **d** of the vertical weld bead.



Solution :

1° Force carried by the horizontal weld beads

Steel grade S235 → f_u=360 Mpa, β_w= 0.80, γ_{mw}=1.25

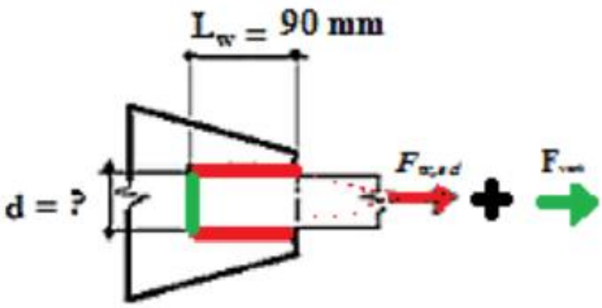
Force carried by the two lateral weld beads, with L_w=90 mm.

The loading applied to these weld beads is in-plane (longitudinal), therefore:

$$\sigma_{\perp} = 0 \text{ and } \tau_{\perp} = 0$$

$$\begin{cases} \sqrt{\sigma_{\perp}^2 + 3(\tau_{\perp}^2 + \tau_{\parallel}^2)} \leq \dfrac{f_u}{\beta_w \gamma_{Mw}} \\ \sigma_{\perp} \leq \dfrac{f_u}{\gamma_{Mw}} \end{cases} \Rightarrow \begin{cases} \sqrt{3(\dfrac{F_{w,sd}}{2.a.L_W})^2} \leq \dfrac{f_u}{\beta_w \gamma_{Mw}} \\ \sigma_{\perp} \leq \dfrac{f_u}{\gamma_{Mw}} \end{cases} \Rightarrow \sqrt{3(\dfrac{F_{w,sd}}{2.8.90})^2} \leq 360$$

$$\Rightarrow F_{w,sd} \leq \frac{360.8.2.90}{\sqrt{3}} \text{ Hence } F_{w,sd} \leq 299298{,}38\ N \Rightarrow F_{w,sd} = 299{,}30\ KN$$



2° Force carried by the vertical weld bead

$F = F_{red} + F_{green} \Rightarrow F_{green} = F - F_{red} = 460 - 299{,}30 = 160{,}70$ KN

The loading applied to this weld bead is frontal, therefore: $\tau_{\parallel} = 0$

The normal stress perpendicular to the weld throat:

$$\sigma_{\perp} = \frac{F_{green}}{\sqrt{2}.a.d} = \frac{160700}{\sqrt{2}.8.d}\ \mathrm{N/mm^2}$$

The shear stress in the plane of the throat, perpendicular to the weld axis

$$\tau_{\perp} = \frac{F_{green}}{\sqrt{2}.a.d} = \frac{160700}{\sqrt{2}.8.d}\ \mathrm{N/mm^2}$$

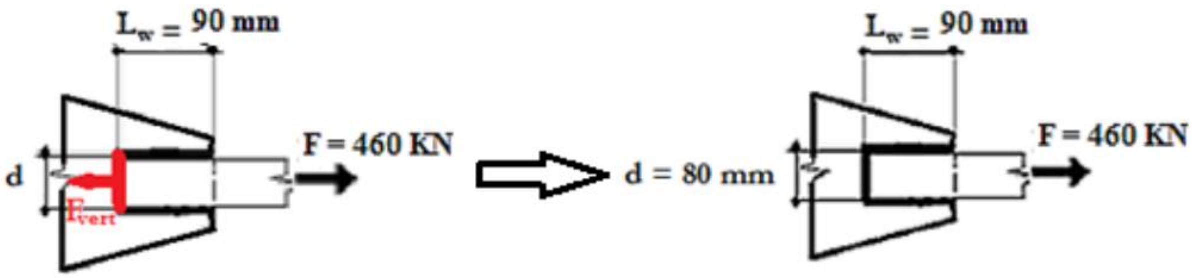
The resistance of the fillet weld is sufficient if the following two conditions are satisfied:

$$\begin{cases} \sqrt{\sigma_{\perp}^2 + 3(\tau_{\perp}^2 + \tau_{\parallel}^2)} \le \dfrac{f_u}{\beta_w \gamma_{Mw}} \\ \sigma_{\perp} \le \dfrac{f_u}{\gamma_{Mw}} \end{cases} \Rightarrow \begin{cases} \sqrt{\left(\dfrac{160700}{\sqrt{2}.8.d}\right)^2 + 3\left(\dfrac{160700}{\sqrt{2}.8.d}\right)^2} \le \dfrac{360}{0{,}80.1{,}25} \\ \dfrac{160700}{\sqrt{2}.8.d} \le \dfrac{360}{1{,}25} \end{cases}$$

$$\Rightarrow \begin{cases} 2.\dfrac{160700}{\sqrt{2}.8.d} \le \dfrac{360}{1} \\ \dfrac{160700}{\sqrt{2}.8.d} \le \dfrac{360}{1{,}25} \end{cases} \qquad \Rightarrow \begin{cases} d \ge \dfrac{2x160700}{360x8x\sqrt{2}} \\ d \ge \dfrac{160700}{288x8x\sqrt{2}} \end{cases}$$

$$\Rightarrow \begin{cases} d \ge 78{,}91\ mm \\ d \ge 49{,}32\ mm \end{cases}$$

$$d \ge \max(78{,}91\ mm, 49{,}32\ mm) \Rightarrow thus: \boldsymbol{d = 80mm}$$



Simple Tension

Chapter4. Calculation of Members Subjected to Simple Tension

4.1 Introduction

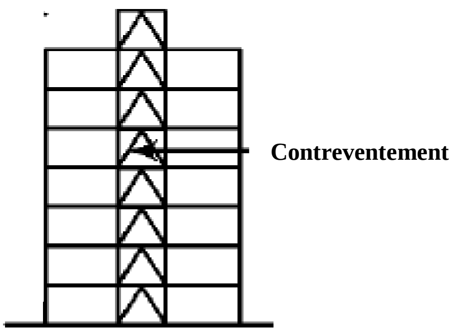
There are several instances in metal construction where elements are subjected to simple tension, either as primary components or secondary ones:

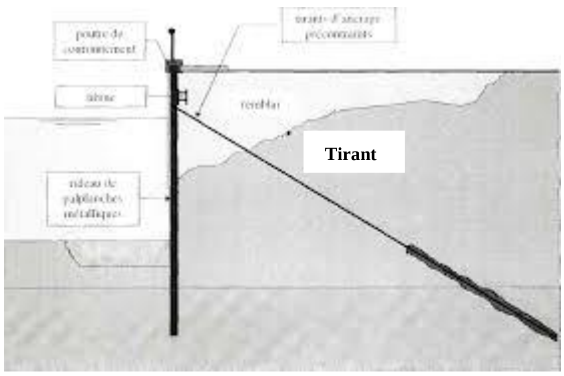
a) Bracing bars and tie rods that ensure the stability of structures and multi-story buildings;

b) Chords that form part of the trusses in a structure (bridges, truss frames, etc.);

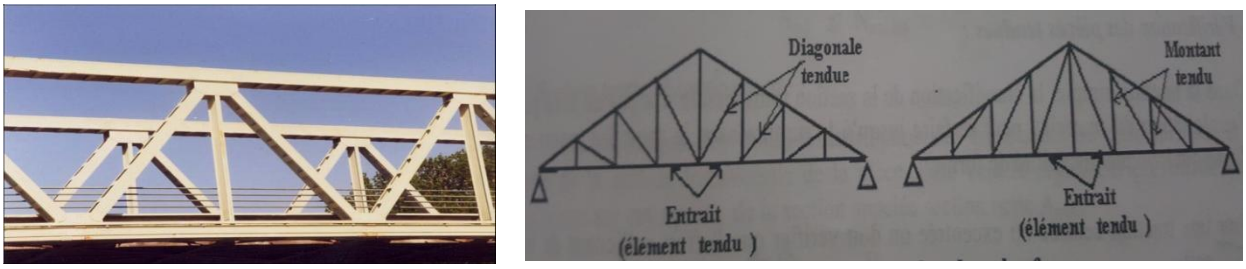
c) Cables;

d) Anchors that secure the structure to the foundation.

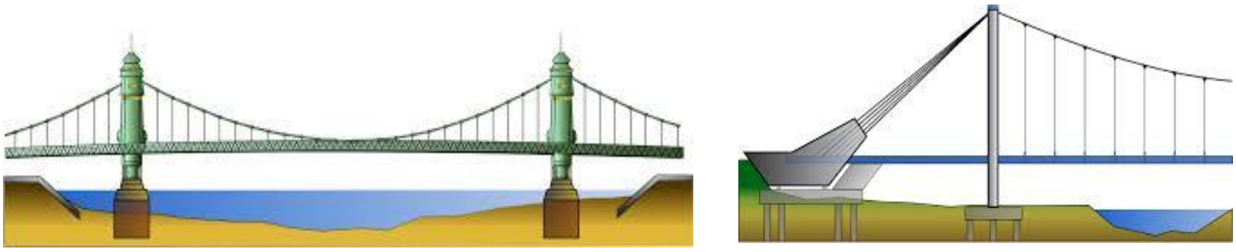




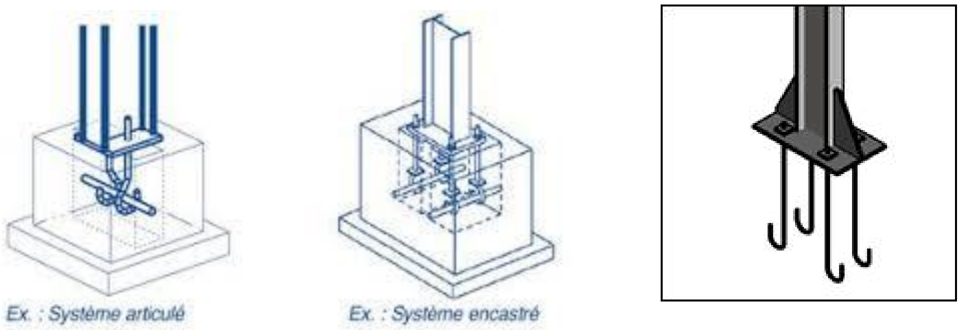
Cas a *:Bracing Member and Tie Rods*



***Cas b**: Truss Members*



***Cas c**: Cables*



***Cas d**: Foundation Anchors*

Figure (4.1) *Examples of Tension Elements*

4.2 Design of Tension Members

Tension members are the simplest structural elements to design, as they are subjected to a uniaxial stress state. Moreover, their behavior is not affected by instability issues, and finally, the shape of the cross-section has no influence on the strength of the member.

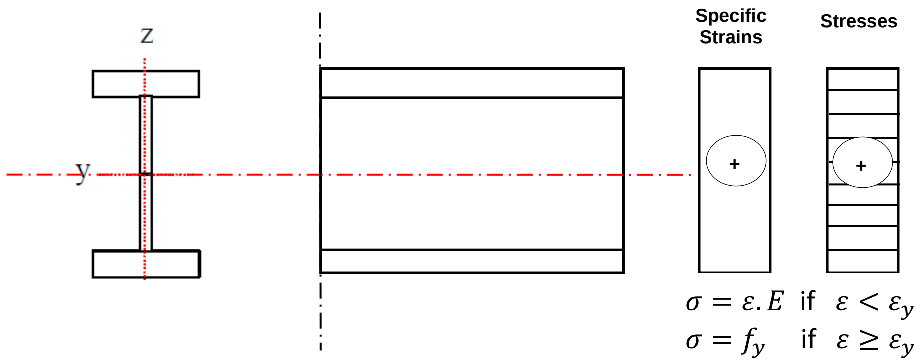


Figure (4.2) *Specific Strains and Stresses in a Section Subjected to Axial Tensile Forc*

ε: Specific deformation;

ε_y: Elastic limit deformation;

f_y: Yield strength.

The ultimate resistance N_R to a normal tensile force can therefore be expressed by the following relation:

$$N_R = N_{pl} = f_y.A$$

Where :

$\boldsymbol{N_{pl}}$: Plastic resistance of the gross section ;

A : Gross cross-sectional area of the profile.

To design the element subjected to a normal tensile force Nd, the following formula is used:

$$N_d \leq \frac{N_{pl}}{\gamma_M} = \frac{f_y.A}{\gamma_M}$$

Thus

$$A \leq N_d \frac{\gamma_M}{f_y}$$

4.3 Tensile Verification :

For an element subjected to axial tension, the tensile force Nsd in each cross-section must remain less than the tensile resistance, expressed as:

$$N_{sd} \leq N_{t,Rd} = min(N_{pl,Rd}, N_{u,Rd})$$

Where:

$\boldsymbol{N_{pl}}$: Plastic resistance of the gross section ;

$$N_{pl,Rd} = A.\frac{f_y}{\gamma_{M0}}$$

$\boldsymbol{N_u}$:Ultimate resistance of the net section at the location of connection holes.

$$N_{u,Rd} = 0{,}9A_{nette}.\frac{f_u}{\gamma_{M2}}$$

A : Gross cross-sectional area of a member;

$\mathbf{A_{nette}}$: Net cross-sectional area of a member;

$\boldsymbol{\gamma_M}$: Partial safety factor ;

$\boldsymbol{f_u}$: Ultimate strength of a member;

$\boldsymbol{t}$: Thickness of the member;

$\boldsymbol{d_{tr}}$: Diameter of the holes.

The factor 0.90 is a reduction coefficient accounting for unavoidable eccentricities, stress concentrations, etc. The design tensile resistance ($N_{pl,Rd}$) is thus taken as the smaller of the values provided by the expressions for $N_{u,Rd}$, and it is compared to the design value of the applied tensile force (N_{sd}).

For connections using preloaded bolts, the design plastic resistance of the net section ($N_{net.Rd}$) is limited to the plastification of the net section, hence:

In cases where ductile behavior is required (e.g., seismic design), it is essential to ensure that the ultimate condition is governed by the plastification of the gross section and not by failure at the net section. Therefore:

$$N_{net,Rd} = A_{net}.\frac{f_y}{\gamma_{M0}}$$

Dans le cas où un comportement ductile est exigé (pour le calcul sismique, par exemple), il est nécessaire de s'assurer que la condition limite est la plastification de la section brute et non la ruine au niveau de la section nette. Donc :

$$N_{u,Rd} \geq N_{pl,Rd}$$

This condition is satisfied if :

$$\frac{A_{net}}{A} \geq \frac{\left[\frac{f_y}{f_u}\right]\left[\frac{\gamma_{m2}}{\gamma_{m0}}\right]}{0.9}$$

4.3.1 Rules for Calculating the Net Section

In the general case where holes are arranged in parallel rows perpendicular to the direction of the tensile force, the net section is equal to the gross section reduced by the area of the holes.

Where:

t: Thickness of the member ;

$\boldsymbol{d_{tr}}$: Diameter of the holes;

$\boldsymbol{n}$: Number of holes in the considered section.

Thus: $\boldsymbol{A = b.t}$

$$A_{net} = A - n.t.d_{tr}$$

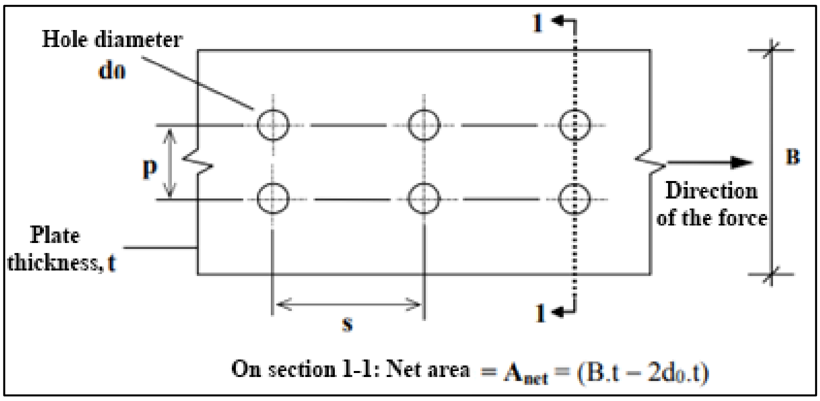


Figure (4.3) *Net section when holes are arranged in parallel*

In cases where the holes are arranged in a staggered pattern, the calculation of the net section requires determining the most critical failure line (i.e., the one with the smallest net section). This involves considering various potential failure lines. For each failure line, the corresponding net section must be calculated, and the smallest value is retained.

For any arbitrary staggered failure line, the net section is determined using the following relation:

$$A_{net} = t\left[b - n.d_{tr} + \sum \frac{s_i{}^2}{4p_i}\right]$$

s_i : Longitudinal spacing between holes ;

p : Transverse spacing between holes.

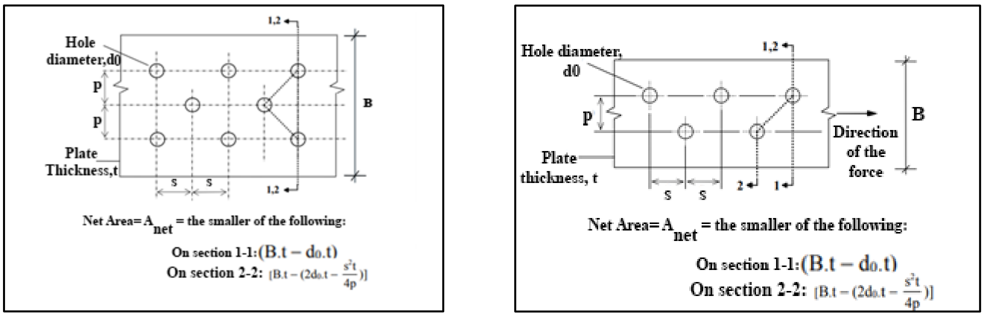


Figure (4.4) *Net section when holes are arranged in a staggered pattern*

4.3.2 Partial Safety Factors

In the calculation of cross-sections, the design resistances are multiplied by a partial safety factor γ_M with the following values :

- *Gross section of class 1,2 or 3* : $\boldsymbol{\gamma_{M0}=1}$ (ou **1.1** for unapproved steels) ;
- *Gross section of class 4 :* $\boldsymbol{\gamma_{M1}=1.1}$ **;**
- *Net section at the location of holes :* $\boldsymbol{\gamma_{M1}=1.25}$.

4.3.3 Tensile Strength of Angles with Bolt Holes:

In general, angles are connected through a single leg (figure 4.5). This results in a complex stress distribution in the connection, as the tensile force in the entire cross-section must be transferred solely through the connected leg. Tests have shown that, in this case, a portion of the section may tear long before the net section fails.

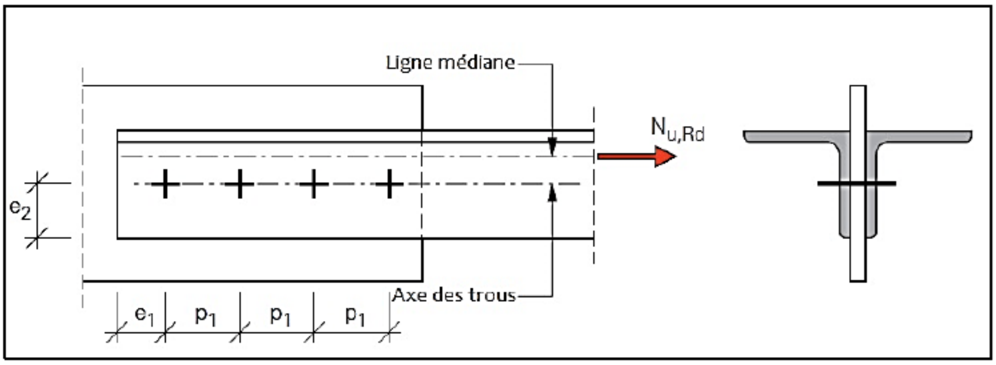


Figure (4.5) *Angles Connected Through a Single Leg.*

The design tensile strength Nu, depends on the number of bolts, the spacing p_1, and the edge distance (end distance) e_2.

For a connection with a single bolt, the following equation applies:

$$N_{u,Rd} = \frac{2(e_2 - 0.5d_0)t.f_u}{\gamma_{m2}}$$

For angle connections with two or more bolts in a leg, a reduction factor β is introduced. This reduction must be applied to the design axial strength of the net section. The value of β depends on the number of bolts and their spacing $P1$. The design strength (Nu,) is calculated as follows:

Case of 2 bolts:

$$N_{u,Rd} = \frac{\beta_2.A_{net}.f_u}{\gamma_{m2}}$$

Case of 3 bolts or more:

$$N_{u,Rd} = \frac{\beta_3.A_{net}.f_u}{\gamma_{m2}}$$

The values of β_2 (two bolts) and β_3 (three bolts or more) as a function of P_1/d_0 are shown in Figure 4.6.

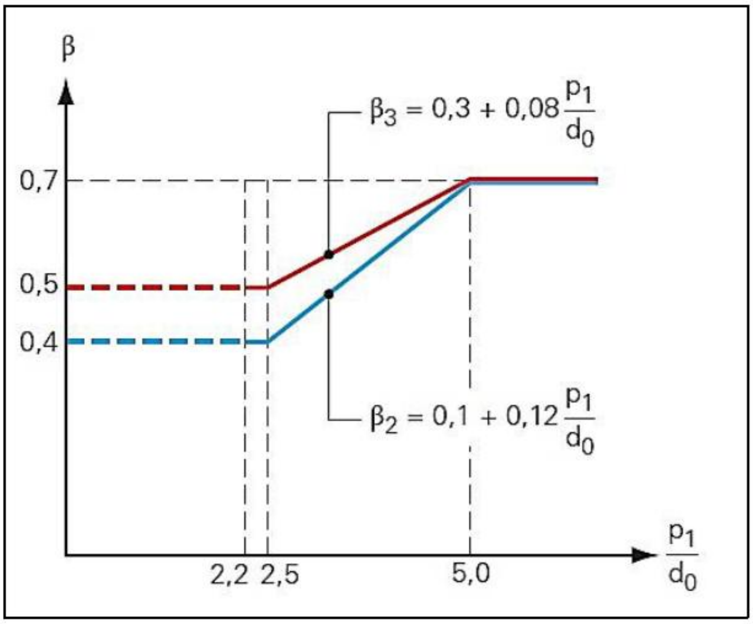


Figure (4.6) *: Reduction Factors β_2 and β_3.*

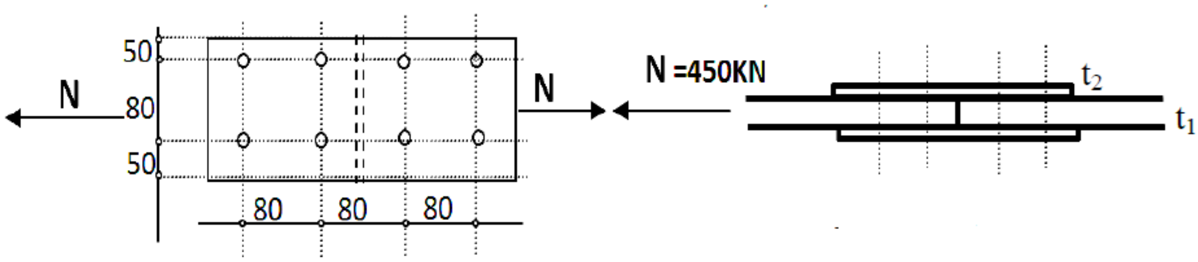
4.4 Applications :

4.4.1 Application 01 :

Consider a continuity splice made of two splice plates with holes of diameter d_0 = 20 mm, transmitting an axial force N_{sd} = 450 KN.

If the steel used is grade S275, γ_{M0}=1 and γ_{M2}=1.25 :

Calculate the required thicknesses t_1 and t_2 of the splice plates, assuming (t_1 =2.t_2) ?



Solution :

Tensile resistance requirement:

$$N_{sd} \le N_{Rd} = minimum\ value\{N_{pl,Rd}, N_{u,Rd}\}$$

Using plastic analysis, we obtain:

$$N_{sd} \le N_{pl,Rd} = A_{brt}\frac{f_y}{\gamma_{m0}}$$

$$A_{brt} \ge N_{sd}\frac{\gamma_{m0}}{f_y}$$

$$A_{brt} \ge 450000\frac{1}{275}$$

$$A_{brt} \ge 1636.363\ mm^2$$

$$bxt_1 \ge 1636.363\ mm^2$$

$$b = 50 + 80 + 50 = 180mm$$

$$t_1 \ge \frac{1636.363}{180} \to t_1 \ge 9.09\ mm$$

From the ultimate (strength) analysis, we have:

$$N_{sd} \le N_{u,Rd} = 0.9A_{net}\frac{f_u}{\gamma_{m2}}$$

$$A_{net} \ge N_{sd}\frac{\gamma_{m2}}{0.9f_u}$$

$$(b - n.d)t_1 \ge N_{sd}\frac{\gamma_{m2}}{0.9f_u}$$

$$(180 - 2X20)t_1 \ge 1453.488mm^2$$

$$\to t_1 \ge 10.382\ mm$$

Finally: t_1 = maximum value (9.09mm ; 10.382mm) = 10.382 mm, i.e., t_1 = 12mm, therefore : t_2= 6mm.

4.1.2 Application 02 :

Determine the design tensile force N_{Sd} that can be carried by the bracing member shown in the figure below. The connection is ensured by bolts with a diameter of 18 mm. The steel grade is S235.

The partial safety factors are $\gamma_{m0} = 1.1$, $\gamma_{m2} = 1.25$ and the ultimate tensile strength is: f_u=360 Mpa.

Solution

The tensile resistance condition is: $N_{sd} \leq N_{Rd}$

with : $N_{Rd} = min\{N_{pl,Rd}, N_{u,Rd}\}$

- **Plastic resistance of the gross section:**

$$N_{pl,Rd} = A_{brt}\frac{f_y}{\gamma_{m0}}$$

Gross section area (total area of the angle): A= 3519 mm^2

Hence :

$$N_{pl,Rd} = 3519.\frac{235}{1,1}$$

$$\boldsymbol{N_{pl,Rd} = 751,786\ \mathrm{KN}}$$

- **Ultimate resistance of the net section :**

$$N_{u,Rd} = 0.9A_{net}\frac{f_u}{\gamma_{m2}}$$

Net section area :

$$A_{net} = \mathrm{A} - (d_{tr}xt)$$

$$A_{net} = 3519 - (18x9)$$

$$\boldsymbol{A_{net} = 3339\ mm^2}$$

Hence :

$$N_{u,Rd} = 0,9.3339.\frac{360}{1,25}$$

$$N_{u,Rd} = \boldsymbol{865,469\ \mathrm{KN}}$$

So, Design tensile resistance :

$$N_{Rd} = min\{751{,}786\ \mathrm{KN}, 865{,}469\} = \boldsymbol{751,786\ \mathrm{KN}}$$

Hence, the design tensile force :

$$N_{Sd} \leq N_{Rd}$$

$$N_{Sd} \leq \mathbf{751,786\ KN}$$

Design of flexural members

Chapter 5. Design of flexural members

5.1 Introduction

To study the mechanical behaviour of a steel frame under applied loads using the finite element method, the analysis is based on linear beam theory regardless of the type of loading.

In each cross-section and under the applied load, internal forces appear, known as the bending moment M, the shear forces V that balance the load, and the support reactions.

In a steel structure, flexural members may consist of hot-rolled profiles (IPN, IPE, H-sections, etc.), trusses, and so on, depending on their role.

Generally, IPE sections are used as the main floor beams in a building. IPN sections are used as secondary beams and joists. H-sections may be used to carry large and substantial loads (large-span beams, crane runways, etc.).

For flexural members (beams) to remain stable and fulfil their load-carrying function, the following verifications are required:

- Strength under factored loads;
- Deflection under unfactored loads;
- Stability against instability phenomena.

5.2 Classification of Cross-Sections

Cross-sections are classified into four classes according to Eurocode 3. This classification is carried out based on various criteria:

- Plate slenderness;
- Design strength;
- Plastic rotation capacity;
- Risk of local buckling, etc.

Determining the class of a section provides information about its behaviour and its resistance, and therefore makes it possible to choose the appropriate design method.

Table (5.1) *Design Method According to Cross-Section Class*

Class	Calculation Method
1	Plastic (allowing the formation of a plastic hinge)
2	Plastic (no hinge formation)
3	Elastic, based on the full section
4	Elastic, based on the effective section

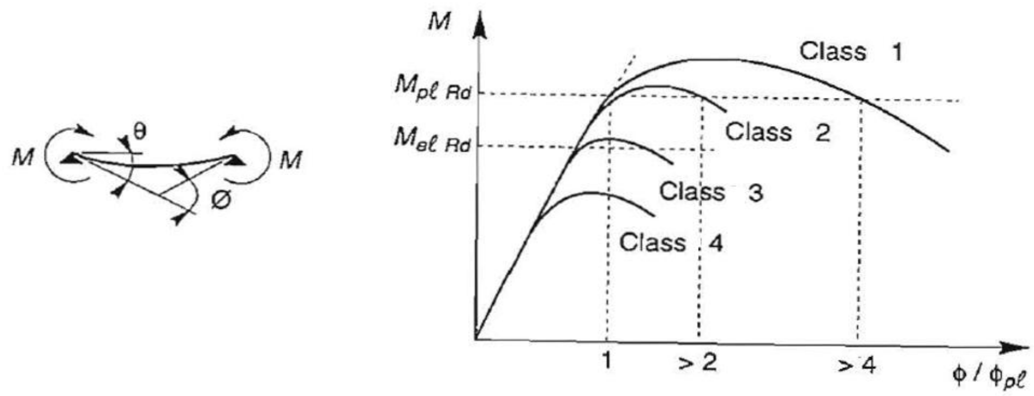


Figure (5.1) *Behaviour Curves of Cross-Section Classes*

5.3 Failure Mechanism

When a steel beam is subjected to simple bending, the behaviour developed in the initial stages after the application of loads is elastic, and the maximum stress in the extreme fibers is:

$$\sigma_f = \frac{M_{el}}{W_{el}} \leq f_e$$

Where : **M_{el}** : elastic moment ($M_{el} = W_{el}.f_e$) ;

W_{el} : elastic section modulus ($W_{el} = \frac{I_y}{v}$)

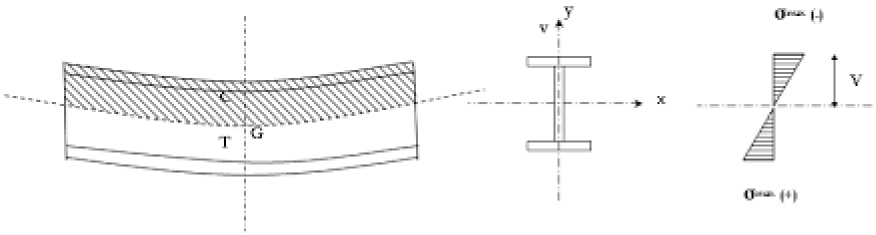


Figure (5.2) *Failure Mechanism of a Beam*

After some time, and if the applied loads are increased, the beam can develop plastic behaviour (formation of plastic hinges).

$$M_{pl} = W_{pl}.f_y$$

Where : **W_{pl}** : plastic section modulus ($W_{pl} = 2.S$).

S : first moment of area of the half-section.

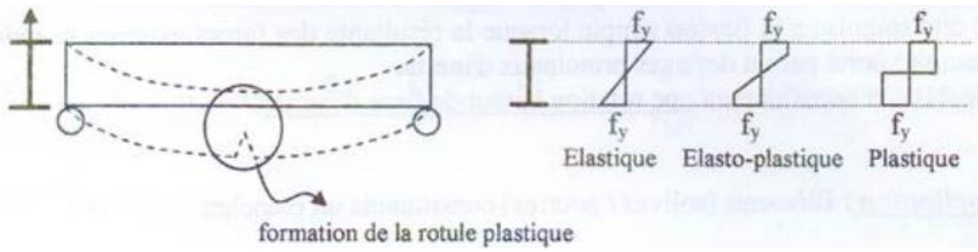


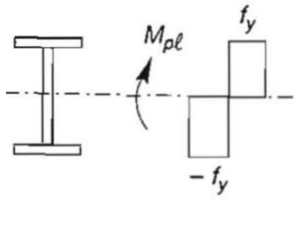
Figure (5.3) *Behaviour of a Bending Beam*

5.4 Resistance of a Cross-Section to a Bending Moment

5.4.1 Section without Connection Holes

a) Plastic Bending

The design value of the bending moment MsdM_{sd}Msd in each cross-section classified as Class 1 or 2 must satisfy the following condition:



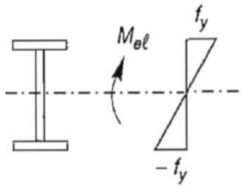
$$M_{sd} \leq M_{c,RD} = M_{pl,Rd} = W_{pl}\frac{f_y}{\gamma_{M0}}$$

Where : **$M_{c,Rd}$** : Design bending resistance;

$M_{pl,Rd}$: Plastic resisting moment;

b) Elastic Bending

The design value of the bending moment MsdM_{sd}Msd in each cross-section classified as Class 3 must satisfy the following condition:

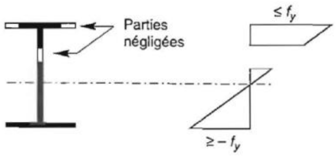


$$M_{sd} \leq M_{c,RD} = M_{el,Rd} = W_{el}\frac{f_y}{\gamma_{M0}}$$

Where : $\mathbf{M_{c,Rd}}$: elastic resisting moment;

$\mathbf{W_{el}}$: elastic section modulus

c) Bending of Class 4 Cross-Sections



$$M_{sd} \leq M_0 = W_{eff}\frac{f_y}{\gamma_{M1}}$$

Where : $\mathbf{M_0}$: local buckling resisting moment ;

$\mathbf{W_{eff}}$: section modulus accounting for local buckling ($W_{eff} = \frac{I_{eff}}{x}$).

5.4.2 Section with Connection Holes

a) Compressed Flange

For compressed flanges, the holes should not be taken into account (the presence of bolts fills the holes).

b) Tension Flange

For tension flanges, the holes should not be taken into account if the following condition is satisfied:

$$0.9\frac{A_{t\,net}}{A_t} \geq \frac{f_y}{f_u}.\frac{\gamma_{M2}}{\gamma_{M0}}$$

Where : $\mathbf{A_{t\,net}}$: area of the tension flange without holes;

$\mathbf{A_t}$: area of the tension flange.

If the above condition is not satisfied, then

$$A_{t\,réduite} = 0.9A_{t\,net}\frac{f_u}{f_y}.\frac{\gamma_{M0}}{\gamma_{M2}}$$

5.5 Shear Resistance of a Cross-Section

The design shear force $\mathbf{V_{sd}}$ in each cross-section must remain lower than the design plastic shear resistance $\mathbf{V_{pl,Rd}}$, namely:

$$V_{sd} \leq V_{pl,Rd} = A_v \left(\frac{f_y}{\sqrt{3}}\right) / \gamma_{M0}$$

Where : $\mathbf{A_V}$: shear area ;

$\frac{f_y}{\sqrt{3}}$: elastic shear stress τ_u for pure shear.

The shear area can be determined as follows (for a force parallel to the web).

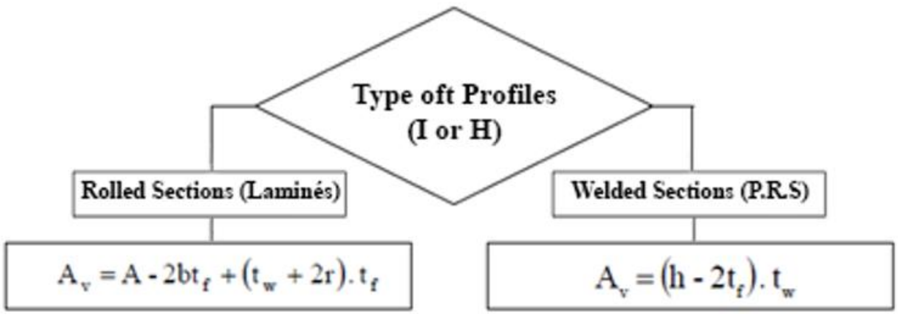


Figure (5.4) *Shear Area for Rolled and Welded Sections*

5.6 Effect of Shear Force on the Resistance Moment

The plastic resistance moment of a cross-section is reduced due to the presence of shear.

If the shear force is low, this reduction is negligible (and compensated by the strain hardening of the material).

If the shear force exceeds half of the plastic shear resistance, its interaction with the plastic resistance behavior must be considered, as follows:

If :

$$\frac{V_{sd}}{V_{pl}} \leq 0.5 \rightarrow \quad M \leq M_R$$

If :

$$\frac{V_{sd}}{V_{pl}} > 0.5 \rightarrow \quad M \leq M_v$$

Where : **M_R** : Plastic resistance moment ;

M_v : Reduced plastic resistance moment due to shear force, determined using a reduced yield strength $f_{réd}$ for the shear area only, as follows:

$$f_{réd} = (1 - \rho).f_y$$

Where :
$$\rho = \left(\frac{2V}{V_{pl}} - 1\right)^2$$

For cross-sections with equal flanges and subjected to bending about the strong axis of inertia, the following expression is derived:

$$M_v = \frac{\left(W_{pl}.f_y - W_v.f_y + W_v.f_{réd}\right)}{\gamma_{M0}}$$

Hence:

$$M_v = \left(W_{pl}. - W_v.\rho\right)\frac{f_y}{\gamma_{M0}}$$

With : **W_v** : Plastic section modulus of the shear area :

$$W_v = \frac{A_v^2}{4t_w}$$

Which is expressed as:

$$M_v = \left(W_{pl} - \rho A_v^2/_{4t_w}\right)\frac{f_y}{\gamma_{M0}}$$

5.7 Service ability Limit State Verification

In addition to the strength requirements that the cross-sections must satisfy under various loadings, they must also meet the serviceability limit state requirements. Deflections and vibrations of the beams must be limited in order to avoid adverse effects on the appearance and functional use of the structure.

At the serviceability limit state, it must be ensured that the total deflection satisfies:

$$\boldsymbol{\delta \leq \delta_1}$$

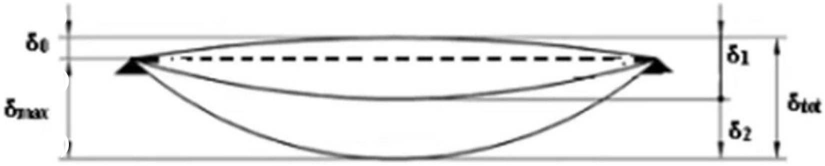


Figure (5.5) *Vertical Deflections of a Simply Supported Beam*

$\boldsymbol{\delta_0}$: Camber in the unloaded element (caused before erection).

$\boldsymbol{\delta_1}$: Deflection under the permanent load.

$\boldsymbol{\delta_2}$: Deflection under the imposed (live) load.

$\boldsymbol{\delta_{tot}}$: The sum of all displacements of the neutral axis: $\delta_{tot} = \delta_1 + \delta_2$

$\boldsymbol{\delta_l}$: The limiting value of deflection (Tab 5.2)

Table (5.2) *Recommended limit values for vertical deflections*

Conditions	$\boldsymbol{\delta_1}$ **Limit for**	
	δ_{max}	$\boldsymbol{\delta_2}$
Roofs in general	L/200	L/250
Roofs frequently accessed by maintenance personnel	L/250	L/300
Floors in general	L/250	L/300
Floors and roofs supporting partitions in plaster or other fragile or stiff materials	L/250	L/350
Floors supporting columns (unless the deflection is included in the overall limit state analysis)	L/400	L/500
Cases where δ_{max} may adversely affect the building appearance	L/250	-
Crane beam: overhead crane for usage group 1-2	L/500	-
Crane beam: overhead crane for usage group 3-4	L/750	-
Crane beam: overhead crane for usage group 5-6	L/1000	-

Not *: The effective area A_{eff} is the reduced area that accounts for local buckling in the thin elements of a class 4 section. It is calculated by replacing the gross widths with effective widths beffb_\text{eff}beff obtained using the formulas of Eurocode 3 (clause 5.3.5).*

5.8 Applications

5.8.1 Application 01

Consider a simply supported joist beam resting on main beams. The beam has a span of L = 4.00 m and is subjected to a uniformly distributed load, consisting of:

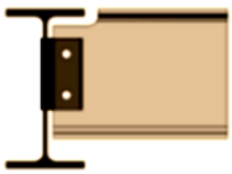
- Permanent load: G = 20 KN/m (self-weight of the section neglected),
- Live load: Q = 28 KN/m.

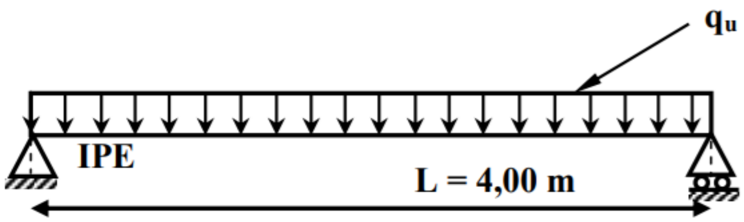
The steel grade used is S235.

1° Determine the maximum design bending moment $M_{y,SdM}$ and shear force $V_{z,SdV}$ acting on the beam at the ultimate limit state (ULS).

2° Design the beam using an IPE section at the ultimate limit state.

3° Check the shear resistance of this beam.





Solution

1° Calculation of the Maximum Internal Forces Acting on the Beam

→ Design Uniformly Distributed Load

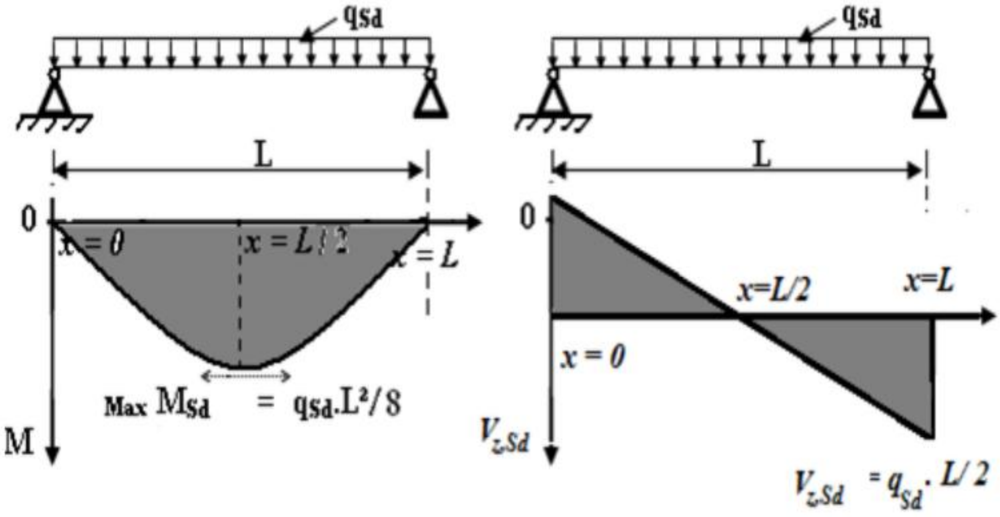
$$q_{sd}=1.35G+1.5Q \rightarrow q_{sd}=69 \text{ KN/ml}$$

→ Maximum Bending Moment

$$M_{sd}^{max}=\frac{q_{sd}L^2}{8} \rightarrow M_{sd}^{max}=138\ KN.m$$

→ Maximum Shear Force

$$V_{sd} = \frac{q_{sd}.L}{2} \rightarrow V_{sd} = 138 \text{ KN}$$



2° Design of the Beam Using an IPE Section

$$M_{sd} \leq M_{pl,Rd}$$

$$M_{sd} \leq W_{ply} \frac{f_y}{\gamma_{m0}}$$

$$\boldsymbol{W_{pl,y}} \geq M_{sd} \frac{\gamma_{m0}}{f_y}$$

$$\boldsymbol{W_{pl,y}} \geq 587.23\ cm^3$$

Let an IPE 300 section be used

$$\text{IPE 300} \begin{cases} W_{pl,y} = 628{,}4\ cm^3 \\ A_{vz} = 25{,}7\ cm^2 \\ t_w = 7.1\ mm \end{cases}$$

3° Verification of the shear resistance of this beam

$$V_{sd} \leq V_{pl,Rd}$$

→ Plastic shear resistance force

$$V_{pl,Rd} = \frac{A_v}{\sqrt{3}} \frac{f_y}{\gamma_{m0}}$$

$$V_{pl,Rd} = 348{,}69\ KN$$

Hunse : $V_{sd} < V_{pl,Rd}$ (OK)

Therefore, the shear resistance is satisfied

4° Verification of the Interaction Effect of the Shear Force on the Bending Resistance

The ratio: $\frac{V_{sd}}{V_{pl,Rd}} = 0.396 < 0.5$

For this beam, there is no interaction between the shear force and the maximum bending moment for which the section was designed.

5.8.2 Application 02

Consider a cantilever beam made of a rolled HEB section with a span of L = 2.00 m. This cantilever is subjected to a constant linear permanent load: G=20 kN/m and a constant linear live load: Q=36 kN/m

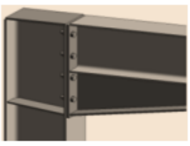
The steel grade used is S355 and γ_{m0}= 1,0. (The self-weight of the section is neglected.)

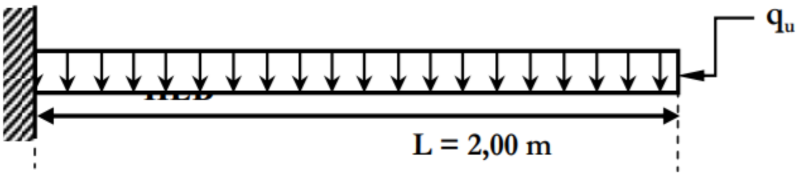
1° Determine the maximum design bending moment $M_{y,Sd}$ and shear force $V_{z,Sd}$ acting on the beam at the Ultimate Limit State (ULS).

2° Design the beam using a HEB section at the Ultimate Limit State.

3° Verify the shear resistance of this beam.

4° Check the interaction effect of the shear force V_{Sd}





Solution

1° Calculation of the Maximum Internal Forces Acting on the Beam

→ Design Uniformly Distributed Load

$$q_{sd}=1.35G+1.5Q \rightarrow q_{sd}=81 \text{ KN/ml}$$

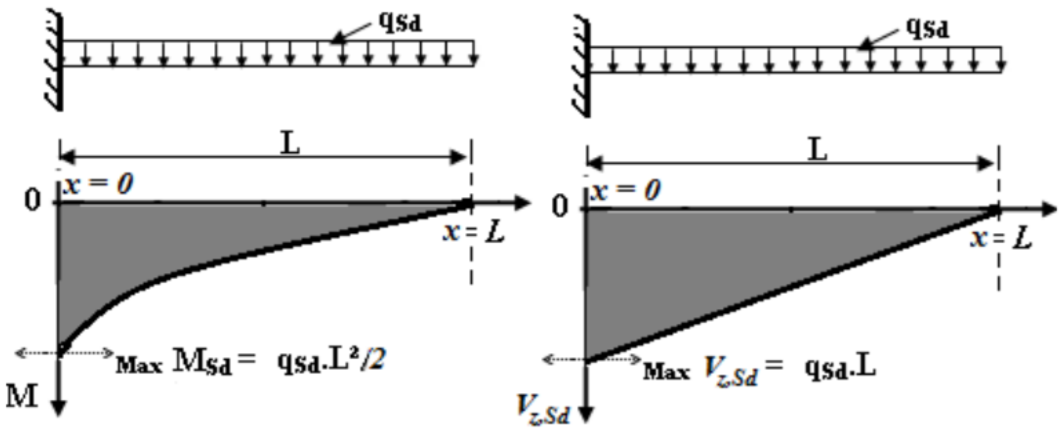
For a uniformly loaded cantilever, the maximum bending moment and shear force occur at the fixed support. They are given by:

→ Maximum Bending Moment

$$M_{sd}^{max}=\frac{q_{sd}L^2}{2} \rightarrow M_{sd}^{max}=162\ KN.m$$

→ Maximum Shear Force

$$V_{sd} = q_{sd}L \rightarrow V_{sd} = 162 \text{ KN}$$



2° Design of the Beam Using an IPE Section

$$M_{sd} \leq M_{Rd} = M_{pl,Rd}$$

$W_{pl,y}$: All HEB sections are Class 1 in bending.

$$M_{sd} \leq W_{ply}\frac{f_y}{\gamma_{m0}}$$

$$\boldsymbol{W_{pl,y}} \geq M_{sd}\frac{\gamma_{m0}}{f_y}$$

$$\boldsymbol{W_{pl,y}} \geq 162000 \frac{1}{355}$$

$$\boldsymbol{W_{pl,y}} \geq 456.33\ cm^3$$

Let an HEB 180 section be used

$$\text{HEB 180} \begin{cases} W_{pl,y} = 481{,}4\ cm^3 \\ A_{vz} = 20{,}2\ cm^2 \\ t_w = 8.5\ mm \end{cases}$$

The plastic bending moment of the selected section is:

$$M_{plRd} = W_{ply} \frac{f_y}{\gamma_{m0}} = 481{,}4 \frac{355}{1} = 170{,}89 kN.M$$

Hence, the verification is carried out as follows (neglecting the self-weight):

$$M_{sd} = 162 \text{ kN} < M_{pl,Rd} = 170{,}89 kN.M \text{ (OK)}$$

The HEB 180 section meets the Ultimate Limit State (ULS) design strength for this beam.

The section's utilization factor is:

$$\frac{M_{sd}}{M_{pl,Rd}} = 94.79\%$$

3° Verification of the shear resistance of this beam

$$V_{sd} \leq V_{pl,Rd}$$

→ Plastic shear resistance force

$$V_{pl,Rd} = \frac{A_v}{\sqrt{3}} \frac{f_y}{\gamma_{m0}}$$

$$V_{pl,Rd} = \frac{2020}{\sqrt{3}} \frac{355}{1}$$

$$V_{pl,Rd} = 414{,}01\ KN$$

Hunse : $V_{sd} = 162 \text{ kN} < V_{pl,Rd} = 414{,}01\ KN$ (OK)

Therefore, the shear resistance is satisfied.

4° Verification of the Interaction Effect of the Shear Force on the Bending Resistance

The ratio: $\frac{V_{sd}}{V_{pl,Rd}} = 0.391 < 0.5$

The interaction between the maximum shear force and the maximum bending moment at the fixed support (x = 0) has no influence on the plastic bending resistance of the section.

5.8.3 Application 03

Consider a cantilever beam made of a rolled HEA A section with a span of L= 0.75 m. This cantilever is subjected to:

- A concentrated permanent load: P_G=40 kN
- A concentrated live load: P_Q=100 kN

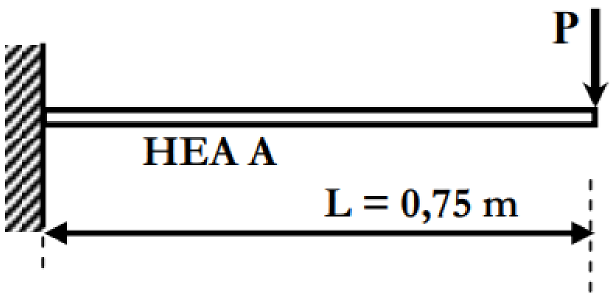
The steel grade used is S275. (The self-weight of the section is neglected.)

1° Determine the maximum design bending moment $M_{y,Sd}$ and shear force $V_{z,Sd}$ acting on the beam at the Ultimate Limit State (ULS).

2° Design the beam using an HEA A section at the Ultimate Limit State.

3° Verify the shear resistance of this beam.

4° Check the interaction effect of the shear force V_{Sd} on the bending resistance ?



Solution

1° Calculation of the Maximum Internal Forces Acting on the Beam

→ Design Uniformly Distributed Load

$$P_{sd}=1.35P_G+1.5P_Q \rightarrow P_{sd}=204\ KN$$

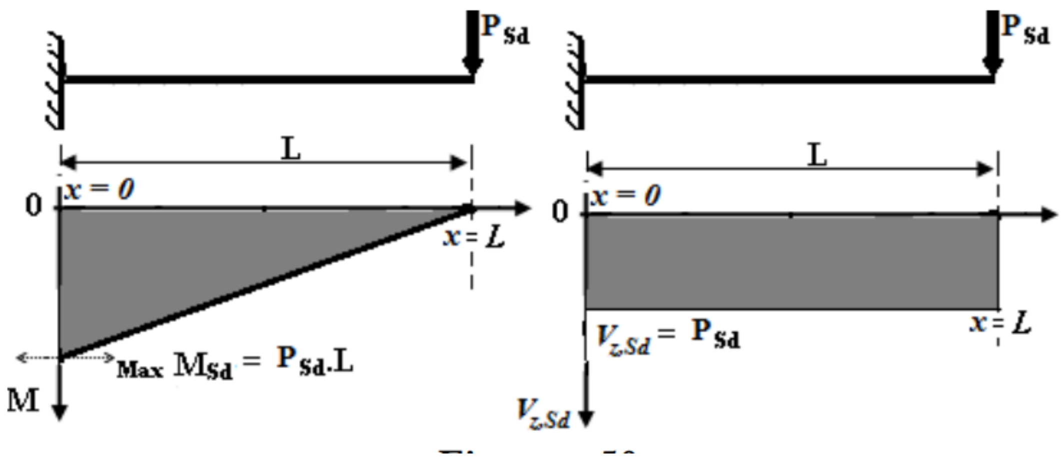
For a cantilever subjected to bending, the maximum bending moment coincides with the maximum shear force at the fixed support. In this case, we have:

→ Maximum Bending Moment

$$M_{sd}^{max}=P_{sd}.L \rightarrow M_{sd}^{max}=153\ KN.m$$

→ Maximum Shear Force

$$V_{sd}^{max}=P_{sd} \rightarrow V_{sd}^{max}=204\ kN$$



2° Design of the Beam Using an IPE Section

$$M_{sd} \leq M_{c,Rd} = M_{?,Rd}$$

$$M_{sd} \leq W_{ply} \frac{f_y}{\gamma_{m0}}$$

$$W_{?,y} \geq M_{sd} \frac{\gamma_{m0}}{f_y}$$

$\boldsymbol{W_{?,y}}$**:** HEA A sections exhibit different section classes in bending depending on their dimensions. In this case, the bending section modulus must be selected with regard to the section class of the profile.

$$W_{?,y} \geq 15300 \frac{1}{275}$$

$$W_{?,y} \geq 556{,}36\ cm^3$$

From the table of rolled HEA A sections, it is found that the (HEA A 240) section provides a plastic bending section modulus**:**

$W_{pl,y}$=570.6 cm^3>556.36 cm^3

However, this section is Class 3**,** which means that it can be exploited only elastically**.** Its elastic bending section modulus is insufficient since:

$W_{el,y}$=521 cm^3<556.36 cm^3 .

Therefore, we move to the next section **(**HEA A 260), which is also Class 3. For this section :

$W_{el,y}$=654.1 cm^3>556.36 cm^3

Thus, we select an HEA A 260 section ($W_{el,y}$=654.1 cm3)

The elastic bending resistance moment of the selected section is:

$$M_{c,Rd} = M_{el,Rd}$$

$$M_{c,Rd} = W_{ely} \frac{f_y}{\gamma_{m0}}$$

$$M_{c,Rd} = 654{,}1 \frac{275}{1}$$

$$M_{c,Rd} = M_{el,Rd} = 179{,}87 \mathrm{KN.m}$$

Hence, the verification is carried out as follows (neglecting the self-weight):

$$M_{y,sd} = 153\ \mathrm{kN.m} \leq M_{el,Rd} = 179{,}87 \mathrm{KN.m}$$

The HEA A 260 section satisfies the Ultimate Limit State (ULS) requirements and is suitable for this beam.

3° Verification of the shear resistance of this beam

$$V_{z,sd} \leq V_{pl,Rd}$$

→ Plastic shear resistance force

$$V_{pl,Rd} = \frac{A_v}{\sqrt{3}} \frac{f_y}{\gamma_{m0}}$$

$$V_{pl,Rd} = \frac{2470}{\sqrt{3}}\frac{275}{1}$$

$$V_{pl,Rd} = 392{,}16\,KN$$

Hunse : $V_{z,sd} = 204\text{ kN} < V_{pl,Rd} = 392{,}16\,KN$ (OK)

4° Verification of the influence of the shear force on the beam

The ratio: $\frac{V_{sd}}{V_{pl,Rd}} = 0.520 > 0.5$

There is an influence of the shear force $V_{z,Sd}$ on the plastic bending resistance $M_{pl,Rd}$,and the latter must be reduced.

Therefore, the reduced plastic bending moment resistance is calculated as follows:

$$M_v = \left(W_{pl} - \frac{\rho A_v^2}{4t_w}\right)\frac{f_y}{\gamma_{M0}}$$

Where :

$$\rho = \left(\frac{2V_{sd}}{V_{pl,Rd}} - 1\right)^2 = \left(\frac{2x204}{392{,}16} - 1\right)^2 = 0{,}0016$$

$W_{pl} = 714{,}5\text{ cm}^3$; $t_w = 6{,}5\,mm$

$$M_v = \left(714{,}5 - \frac{0{,}0016x24{,}7^2}{4x6{,}5}\right)\frac{275}{1}196382{,}294\text{ N.m} \approx 196{,}38\text{KN.m}$$

Hence :

$$M_{c,Rd} = M_{el,Rd} = 179{,}87\text{KN.m} < M_v = 196{,}38\text{KN.m}$$

The interaction between the shear force and the maximum bending moment does not pose a risk to the elastic bending resistance of the section, since reducing the plastic resistance (which is not applicable in this case, as the section is Class 3) does not make it lower than the design resistance $M_{el,Rd}$.

5.8.3 Application 03

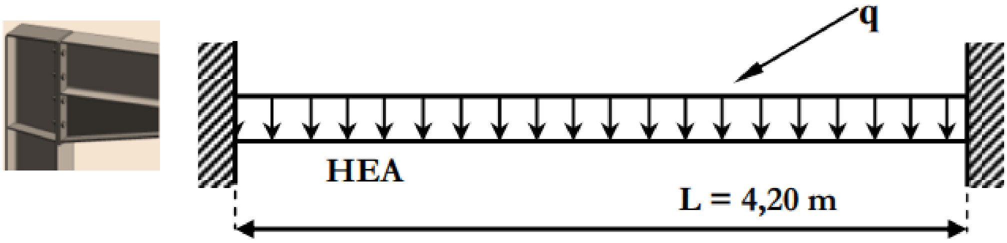
Consider a fixed–fixed beam with a span of L = 4.20 m. The beam is uniformly subjected to a permanent load G=54 kN/m and a live load: Q=100 kN/m. The steel grade used is S235. (The self-weight of the section is neglected.)

1° Determine the maximum design bending moment $M_{y,Sd}$ and shear force $V_{z,Sd}$ acting on the beam at the Ultimate Limit State (ULS).

2° Design the beam using an HEA section at the Ultimate Limit State.

3° Verify the shear resistance of this beam.

4° Check the interaction effect of the shear force V_{Sd} on the bending resistance of the selected section ?



Solution

1° Calculation of the Maximum Internal Forces Acting on the Beam

→ Design Uniformly Distributed Load

$$q_u = 1.35G + 1.5Q \rightarrow q_u = 222,9 \ \text{KN/m}$$

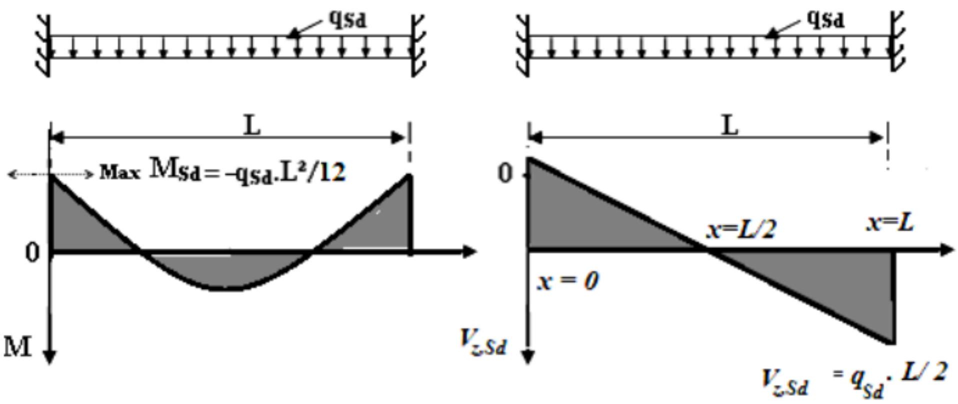
→ Maximum Bending Moment

For a fixed–fixed beam subjected to a uniformly distributed load, the maximum external bending moment, located at the fixed supports, is given by:

$$M_{sd}^{max} = \frac{q_u L^2}{12} = \frac{222900 X 4,2^2}{12} \rightarrow M_{sd}^{max} = 327663 \text{ N.m} = 327,663 \text{KN.m}$$

→ Maximum Shear Force

$$V_{sd}^{max} = \frac{q_u . L}{2} = \frac{222900 X 4,2}{2} \rightarrow V_{sd}^{max} = 468090 \text{ N} = 468,09 \text{ KN}$$



2° Design of the Beam Using an IPE Section

From the resistance of the beam section, we have:

$$M_{sd} \leq M_{c,Rd} = M_{?,Rd}$$

$$M_{sd} \leq W_{ply} \frac{f_y}{\gamma_{m0}}$$

W $_{?,y}$: In bending, HEA sections may have different cross-section classes depending on their geometric proportions.

$$W_{?,y} \geq M_{sd} \frac{\gamma_{m0}}{f_y}$$

$$W_{?,y} \geq 327663 \frac{1}{235}$$

$$W_{?,y} \geq 1394{,}31 \text{ cm3}$$

Selection of the steel section : From the table of rolled HEA sections, the following section is selected: HEA 320 ($W_{pl,y}$ = 1628,1 cm^3 > 1394,31 cm^3).

This section is Class 1 in bending.

Therefore, the plastic moment resistance of the cross-section is:

$$M_{c,Rd} = M_{pl,Rd} = W_{ply} \frac{f_y}{\gamma_{m0}}$$

$$M_{pl,Rd} = 1628{,}1 \frac{235}{1}$$

$$M_{pl,Rd} = 382603{,}5 \text{ N. m} = 382{,}60 \text{ KN. m}$$

Hence, the verification is carried out as follows (neglecting the self-weight):

$$M_{y,sd} = 327{,}663 \text{ KN. m} \leq M_{pl,Rd} = 382{,}60 \text{ KN. m (Ok)}$$

The HEA 320 section satisfies the Ultimate Limit State (ULS) resistance requirements for this beam.

3° Verification of the shear resistance of this beam

$$V_{z,sd} \leq V_{pl,Rd}$$

→ Plastic shear resistance force

$$V_{pl,Rd} = \frac{A_v}{\sqrt{3}} \frac{f_y}{\gamma_{m0}}$$

A_v: shear area (A_{vz}=41,1 cm² = 4110 mm²)

$$V_{pl,Rd} = \frac{4110}{\sqrt{3}}\frac{235}{1}$$

$$V_{pl,Rd} = 557633{,}757 \text{ N} = 557{,}63 \text{ KN}$$

Hunse :

$$V_{z,sd} = 468{,}09 \text{ KN} < V_{pl,Rd} = 557{,}63 \text{ KN (OK)}$$

Therefore, the shear resistance is satisfied.

4° Verification of the influence of the shear force on the beam

The ratio: $\frac{V_{sd}}{V_{pl,Rd}} = 0.839 > 0.5$

Therefore, there is an influence of $V_{z,Sd}$ on $M_{pl,Rd}$, and the latter must be reduced.

For this purpose, the reduced plastic bending moment is calculated as follows:

$$M_v = \left(W_{pl} - \frac{\rho A_{yz}^2}{4t_w}\right)\frac{f_y}{\gamma_{M0}}$$

Where :

$$\rho = \left(\frac{2V_{sd}}{V_{pl,Rd}} - 1\right)^2 = \left(\frac{2x468{,}09}{557{,}63} - 1\right)^2 = 0{,}46$$

$W_{pl} = 1628{,}1 \text{ cm}^3$; $t_w = 9\, mm$

$$M_v = \left(1628{,}1 - \frac{0{,}46x41{,}1^2}{4x0{,}9}\right)\frac{275}{1} = 331788{,}87 \text{ N.m} \approx 331{,}79 \text{ KN.m}$$

Hence :

$$M_{c,Rd} = M_{pl,Rd} = 327{,}663 \text{KN.m} < M_v = 331{,}79 \text{KN.m (Ok).}$$

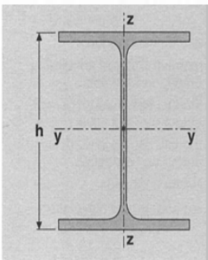
If the reduction results in a bending resistance lower than the applied moment, the section should be increased to an HEA 340 and then subjected again to all previous verifications.

Annex

Section Properties

Table A.1 ***Section Properties of European Sections H (HEA)***

PROFIL	h mm	A cm^2	S m^2/m	I_y cm^4	W_y cm^3	$W_{pl,y}$ cm^3	A_{vz} cm^2	I_z cm^4	W_z cm^3	$W_{pl,z}$ cm^3	I_t cm^4	$I_w \times 10^3$ cm^6
HEA 100	96	21,24	0,561	349,2	72,76	83,01	7,56	133,8	26,76	41,14	5,24	2,58
HEA 120	114	25,34	0,677	606,2	106,3	119,5	8,46	230,9	38,48	58,85	5,99	6,47
HEA 140	133	31,42	0,794	1 033	155,4	173,5	10,12	389,3	55,62	84,85	8,13	15,06
HEA 160	152	38,77	0,906	1 673	220,1	245,1	13,21	615,6	76,95	117,6	12,19	31,41
HEA 180	171	45,25	1,024	2 510	293,6	324,9	14,47	924,6	102,7	156,5	14,8	60,21
HEA 200	190	53,83	1,136	3 692	388,6	429,5	18,08	1 336	133,6	203,8	20,98	108
HEA 220	210	64,34	1,255	5 410	515,2	568,5	20,67	1955	177,7	270,6	28,46	193,3
HEA 240	230	76,84	1,369	7 763	675,1	744,6	25,18	2769	230,7	351,7	41,55	328,5
HEA 260	250	86,82	1,484	10 450	836,4	919,8	28,76	3 668	282,1	430,2	52,37	516,4
HEA 280	270	97,26	1,603	13 670	1 013	1 112	31,74	4 763	340,2	518,1	62,1	785,4
HEA 300	290	112,5	1,717	18 260	1 260	1 383	37,28	6 310	420,6	641,2	85,17	1 200
HEA 320	310	124,4	1,756	22 930	1 479	1 628	41,13	6 985	465,7	709,7	108	1 512
HEA 340	330	133,5	1,795	27 690	1 678	1 850	44,95	7 436	495,7	755,9	127,2	1 824
HEA 360	350	142,8	1,834	33 090	1 891	2 088	48,96	7 887	525,8	802,3	148,8	2 177
HEA 400	390	159	1,912	45 070	2 311	2 562	57,33	8 564	570,9	872,9	189	2 942
HEA 450	440	178	2,011	63 720	2 896	3 216	65,78	9 465	631	965,5	243,8	4 148
HEA 500	490	197,5	2,11	86 970	3 550	3 949	74,72	10 370	691,1	1 059	309,3	5 643
HEA 550	540	211,8	2,209	111 900	4 146	4 622	83,72	10 820	721,3	1 107	351,5	7 189
HEA 600	590	226,5	2,308	141 200	4 787	5 350	93,21	11 270	751,4	1 156	397,8	8 978
HEA 650	640	241,6	2,407	175 200	5 474	6 136	103,2	11 720	781,6	1 205	448,3	11 030
HEA 700	690	260,5	2,505	215 300	6 241	7 032	117	12 180	811,9	1 257	513,9	13 350
HEA 800	790	285,8	2,698	303 400	7 682	8 699	138,8	12 640	842,6	1 312	596,9	18 290
HEA 900	890	320,5	2,896	422 100	9 485	10 810	163,3	13 550	903,2	1 414	736,8	24 960
HEA 1000	990	346,8	3,095	553 800	11 190	12 820	184,6	14 000	933,6	1 470	822,4	32 070



h : section depth

A : cross-sectional area

S : external surface area per linear meter

Iy : second moment of area about the major axis

Wpl,y : plastic section modulus about the major axis

It : torsional constant

Av,z : shear area

Iz : second moment of area about the minor axis

Wz : elastic section modulus about the minor axis

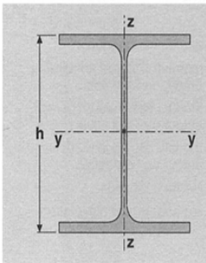
Wy : elastic section modulus about the major axis

Wpl,z : plastic section modulus about the minor axis

Iw : warping constant

Table A.2 ***Section Properties of European Sections I (IPE)***

PROFIL	h mm	A cm^2	S m^2/m	I_y cm^4	W_y cm^3	$W_{pl,y}$ cm^3	A_{vz} cm^2	I_z cm^4	W_z cm^3	$W_{pl,z}$ cm^3	I_t cm^4	$I_w \times 10^{-3}$ cm^6
IPE 80	80	7,64	0,328	80,14	20,03	23,22	3,58	8,49	3,69	5,82	0,70	0,12
IPE 100	100	10,32	0,400	171	34,20	39,41	5,08	15,92	5,79	9,15	1,20	0,35
IPE 120	120	13,21	0,475	317,8	52,96	60,73	6,31	27,67	8,65	13,58	1,74	0,89
IPE 140	140	16,43	0,551	541,2	77,32	88,34	7,64	44,92	12,31	19,25	2,45	1,98
IPE 160	160	20,09	0,623	869,3	108,7	123,9	9,66	68,31	16,66	26,10	3,60	3,96
IPE 180	180	23,95	0,698	1 317	146,3	166,4	11,25	100,9	22,16	34,60	4,79	7,43
IPE 200	200	28,48	0,768	1 943	194,3	220,6	14,00	142,4	28,47	44,61	6,98	12,99
IPE 220	220	33,37	0,848	2 772	252,0	285,4	15,88	204,9	37,25	58,11	9,07	22,67
IPE 240	240	39,12	0,922	3 892	324,3	366,6	19,14	283,6	47,27	73,92	12,88	37,39
IPE 270	270	45,94	1,041	5 790	428,9	484,0	22,14	419,9	62,20	96,95	15,94	70,58
IPE 300	300	53,81	1,160	8 356	557,1	628,4	25,68	603,8	80,50	125,2	20,12	125,9
IPE 330	330	62,61	1,254	11 770	713,1	804,3	30,81	788,1	98,52	153,7	28,15	199,1
IPE 360	360	72,73	1,353	16 270	903,6	1 019	35,14	1 043	122,8	191,1	37,32	313,6
IPE 400	400	84,46	1,467	23 130	1 156	1 307	42,69	1 318	146,4	229,0	51,08	490
IPE 450	450	98,82	1,605	33 740	1 500	1 702	50,85	1 676	176,4	276,4	66,87	791
IPE 500	500	115,5	1,744	48 200	1 928	2 194	59,87	2 142	214,2	335,9	89,29	1 249
IPE 550	550	134,4	1,877	67 120	2 441	2 787	72,34	2 668	254,1	400,5	123,2	1 884
IPE 600	600	156,0	2,015	92 080	3 069	3 512	83,78	3 387	307,9	485,6	165,4	2 846
IPE 750 × 137	753	174,6	2,506	159 900	4 246	4 865	92,90	5 166	392,8	614,1	137,1	6 980
IPE 750 × 147	753	187,5	2,510	166 100	4 411	5 110	105,4	5 289	399,2	630,8	161,5	7 141
IPE 750 × 173	762	221,3	2,534	205 800	5 402	6 218	116,4	6 873	514,9	809,9	273,6	9 391
IPE 750 × 196	770	250,8	2,552	240 300	6 241	7 174	127,3	8 175	610,1	958,8	408,9	11 290



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S : external surface area per linear meter

Iy : second moment of area about the major axis

Wpl,y : plastic section modulus about the major axis

It : torsional constant

$A_{v,z}$: shear area

Iz : second moment of area about the minor axis

Wz : elastic section modulus about the minor axis

Wy : elastic section modulus about the major axis

Wpl,z : plastic section modulus about the minor axis

Iw : warping constant

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