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Title

Assessing the Effectiveness of Structural System Selection using Multicriteria Decision Making Approach. A case study of a multipurpose hangar.

Defended on ……../………/……………..

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بسم الله الرحمن الرحيم

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To the jury... We have the privilege of judging

this work, and we hope that you will find here the expression of our

respectful thanks.

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DEDICATION

لم تستطع كل الحروف أن تجد الكلمات المناسبة

كل الكلمات تعجز عن التعبير عن الامتنان والحب والاحترام والتقدير

أهدي هذه المذكرة:

إلى أجدادنا

نشكركم على دعواتكم طوال سنوات الدراسة هذه، وندعو الله أن يمنحكم الصحة والعافية والعمر المديد بيننا.

إلى والدينا الأعزاء

نعتذر عن لحظات الغياب الكثيرة. نحيي هذا العمل على الحب الذي أظهرتموه لنا دائماً، وعلى كل التشجيع والمساعدة التي قدمتموها لنا خلال دراستنا.

لا يمكن لأي كلمات أو إهداء أن يعبّر عن احترامنا وتقديرنا وحبنا لتضحياتكم من أجلنا

نأمل أن يكون هذا العمل تقديراً لامتناننا العميق.

نأمل أن نكون قد حققنا آمالكم

حفظكم الله ورزقكم الله الصحة والسعادة والعمر المديد.

إلى إخواننا وأخواتنا الأعزاء

تعجز اللغة عن التعبير عن احترامنا وتقديرنا للدعم والتشجيع والمحبة التي تقدمونها لنا طوال الوقت. نشكركم ونتمنى لكم السعادة والنجاح والازدهار.

إلى جميع أفراد عائلتي

أعمامي، واخوالي، وخالاتي، وعماتي، وأبناء عمومتي، أهديكم هذا العمل عربوناً لمودتي العميقة وتعلقي الثابت بكم، أدعو الله العلي القدير أن يمدكم بالصحة والسعادة.

Abstract

In this research, our aim is to identify the specific system for instantaneous multidisciplinary structural design from a set of available alternatives, by applying the work to the creation of multiple shapes. This is achieved to optimise between the geometric requirements the different quotations. We studied a Steel structure hangar located in the industrial zone (Naftal) in Ghardaia and we studied reinforced concrete to find out which is better in all aspects. This hangar is usually used for multiple purposes such as storage, maintenance, which requires a careful structural study to ensure sustainability, efficiency, and meeting functional requirements the main part of the study focuses on applying MCDM methodology to select the best structural system for the hangar. Data was collected from multiple sources including interviews with construction experts, cost analysis, and previous case studies. After applying the decision-making tools, the structural system that achieves the best balance of criteria was identified.

Keywords: Reinforced Concrete, Steel Structures, Multi-criteria decision-Making Methods.

ملخص

الهدف من هذه الدراسة هو اختيار أفضل نظام تصميم إنشائي لحظيرة متعدد الأغراض من بين عدة بدائل باستخدام أساليب اتخاذ القرار متعددة المعايير. حيت كانت هده الحظيرة مدروسة على شكل هيكل معدني والمتواجدة في المنطقة الصناعية (نفطال) ولاية غرداية وقمنا بدراستها بالخرسانة المسلحة وهدا بصدد معرفة ايهما احسن من كل الجوانب وهده الحظيرة عادةً تستخدم لأغراض متعددة مثل التخزين، الصيانة، مما يتطلب دراسة هيكلية دقيقة لضمان الاستدامة، الكفاءة، وتلبية المتطلبات الوظيفية ويركز الجزء الرئيسي من الدراسة لاختيار أفضل نظام إنشائي للحظيرة.وتم جمع البيانات من مصادر متعددة بما في ذلك مقابلات مع خبراء البناء، تحليل التكلفة، ودراسات الحالات السابقة.بعد تطبيق أدوات اتخاذ القرار، تم تحديد النظام الإنشائي الذي يحقق أفضل توازن بين المعاييروعادة ما يتطلب اختيار النظام الإنشائي النظر في عدة عوامل مثل التكلفة، قابلية التنفيذ، العمر،السلامة، والاستدامة.

الكلمات المفتاحية:الخرسانة المسلحة،الهياكل المعدنية ، أساليب اتخاذ القرار متعددة المعايير.

Résumé

L'objectif de cette étude est de sélectionner le meilleur système de conception structurelle pour un hangar polyvalent parmi plusieurs alternatives en utilisant des méthodes de prise de décision multicritères. Nous avons étudié un hangar à structure métallique situé dans la zone industrielle (Naphtal) de Ghardaïa et nous avons étudié le béton armé pour déterminer lequel est le meilleur à tous les égards. Ce hangar est généralement utilisé à des fins multiples telles que le stockage, la maintenance, ce qui nécessite une étude structurelle minutieuse pour assurer la durabilité, l'efficacité et la satisfaction des exigences fonctionnelles. La partie principale de l'étude se concentre sur l'application de la méthodologie MCDM pour sélectionner le meilleur système structurel pour le hangar. Les données ont été collectées à partir de sources multiples, notamment des entretiens avec des experts en construction, des analyses de coûts et des études de cas antérieures. La sélection d'un système structurel nécessite généralement la prise en compte de plusieurs facteurs tels que le coût, la durabilité, la maniabilité, la sécurité et la viabilité.

Mots clés : Béton armé, Structures en acier, Méthodes de décision multicritères

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LIST OF NOTATIONS:

f_{cj} d-day compressive strength of the concrete.

f_{tj} :d-day tensile strength of concrete.

Eij: longitudinal deformation modulus of concrete.

Evj: delayed modulus of concrete deformation.

G: Modulus of transverse deformation.

V: Fish coefficient.

fbu: conventional ultimate compressive strength.

ξbc: unit deformation of the concrete.

σ_{bc}:: compressive stress of concrete.

γ: is a safety coefficient.

θ: is a coefficient as a function of the duration of application of the action considered.

$\sigma_{\overline{bc}}$: Permissible concrete compressive stress.

$\tau_{\overline{u}}$:Ultinate shear stresses.

f_{su}: ultimate characteristic strength of steel.

$\sigma_{\overline{s}}$: permissible steel stress.

f_{e} : yield strength of the steels used

η : cracking coefficient

Nu : is the ultimate normal force

Br : is the reduced concrete section

λ : slenderness of the column element.

φ: the ratio of the final deformation due to creep to the initial instantaneous deformation under. This ratio is generally taken to be equal to 2

μ_{bu}: reduced time.

A_{s} : section of tensioned reinforcement.

S_{t}: escapement of reinforcement.

A_{r}:cross-section of distribution reinforcement.

M_{0}:bending moment in the span.

$\mathbf{M_t}$:The maximum moment in the span considered.

$\mathbf{M_w}$**:** The absolute value of the moment on the left support in the span in question.

$\mathbf{M_e}$**:** The absolute value of the moment on the right-hand support in the span in question.

$\mathbf{M_{ut}}$**:** moment capable of the compression table.

$\mathbf{A_{min}}$:minimum section of reinforcement.

$\emptyset_t$**:** diameter of transversd reinforcement.

$\mathbf{I_0}$**:** moment of inertia of the total cross-section made homogeneous.

$\mathbf{T_U}$**:** The ultimate torsional moment.

Ω: The area of the contour drawn at mid-wall thickness.

N: number of levels above the ground.

G: centre of gravity.

R: centre of sttifress.

V: shear force.

δk: Absolute horizontal displacements.

Δk: Relative displacements.

Q: quality factor.

η: damping correction factor.

CT: coefficient, of the system.

$\mathbf{A_t}$:minimum reinforcement cross-section.

τ_{se} :the driving adhesion stress.

General introduction:

In several cases, the selection of an optimized structural system, as well as appropriate structural elements, presents a recurring challenge for civil engineers, especially structural designers. They must consider a range of factors, including economic considerations, environmental requirements, and technical demands, within the complex task of design and decision-making to meet the client's needs.

Many studies have addressed this complex task by providing comparisons between different types of structures such as low-frequency structures, steel structures and even timber structures, often comparing the structural system with respect to resistance to environmental factors, cost, stiffness and sustainability.

Furthermore, researchers are using various methods, including multicriteria decision-making approaches, to reduce the complexity of the decision-making process during design and to make it more structured, allowing them to select the best design alternative based on multiple interacting criteria and expert perspectives.

This thesis investigates the efficiency of selecting the structural system for a multipurpose hangar constructed in 2022 using a steel structure. As a first step, we obtained access to the design and calculation documents from the design firm. We then redesigned the same project using a reinforced concrete structural system.

This thesis investigates the efficiency of choice of the structural system for a multipurpose Hangar constructed in 2022 using a steel structure system, to do so we redesigned the hangar using RC structure proposition after having obtained all the necessary documents used in the first design by the design firm as well as the documents of the implemented steel structure in order to compare and evaluate both structures with regard to the sustainability imperative and clients demands. The following research questions have been posed:

What criteria should be considered when selecting the most sustainable structural system?

What tools or methods can be used to provide a structured decision-making process?

This research aims to assess the effectiveness of choice of the different actors implicated in the design and construction of this project in a structured and scientifically based way the method proposed can also support decision-makers in selecting the most appropriate material for other specific construction projects, considering the trade-offs between the different criteria. The final outcome of the comparison will offer practical insights into which material steel or reinforced concrete can deliver optimal results for this project, while also adhering to specific project goals such as project cost, environmental impact, and technical requirements.

By applying these MCDM methods, decision-makers can comprehensively evaluate the advantages and disadvantages of steel and reinforced concrete structures. This systematic approach ensures that the selected material aligns with the project's overall objectives, balancing cost, performance, sustainability, and other relevant factors.

Structures of thesis:

- **Introduction:**

Part 1 : Theoretical framework

- ♦ **CHAPTER I:** An Overview of Structural elements in Concrete and Steel Structures
- ♦ **CHAPTER II:** Multicriteria decision making methods.
- ♦ **CHAPTER III:** Comparative Studie of Concrete Structures and Steel Structures.

Part 2 : practical part

- ♦ **CHAPTER IV :**The case study design, calculations and analysis.
- ♦ **CHAPTER V:** Application of AHP

- **General conclusion:**

CHAPTER I:

An overview of structural elements in concrete and steel structures

I.1. Introduction:

Reinforced concrete is delineated as a composite material wherein steel is incorporated in a manner that facilitates the collaborative action of both materials in withstanding external forces. The reinforcement steel, whether in the form of rods, bars, or mesh, serves to absorb tensile, shear, and occasionally compressive stresses within a concrete framework. Traditional concrete lacks the capacity to endure tensile and shear stresses induced by various factors like wind, seismic activities, vibrations, among others, rendering it unsuitable for the majority of structural undertakings. In the realm of reinforced concrete, the amalgamation of steel's tensile strength with concrete's compressive strength synergistically enables the s tructural element to endure these stresses across substantial distances. The advent of reinforced concrete during the 19th century marked a pivotal juncture in the construction sector, propelling concrete to the forefront as one of the most prevalent construction materials globally **[2]**. Present-day structures exhibit intricate designs characterized by diverse shapes and numerous curvatures, alongside elongated spans. Consequently, the prevalence of steel structures has emerged, offering several benefits to the construction process: dependability, rapid construction pace, robust steel strength, seismic resilience due to steel ductility, and significantly broader architectural opportunities compared to concrete **[3].**

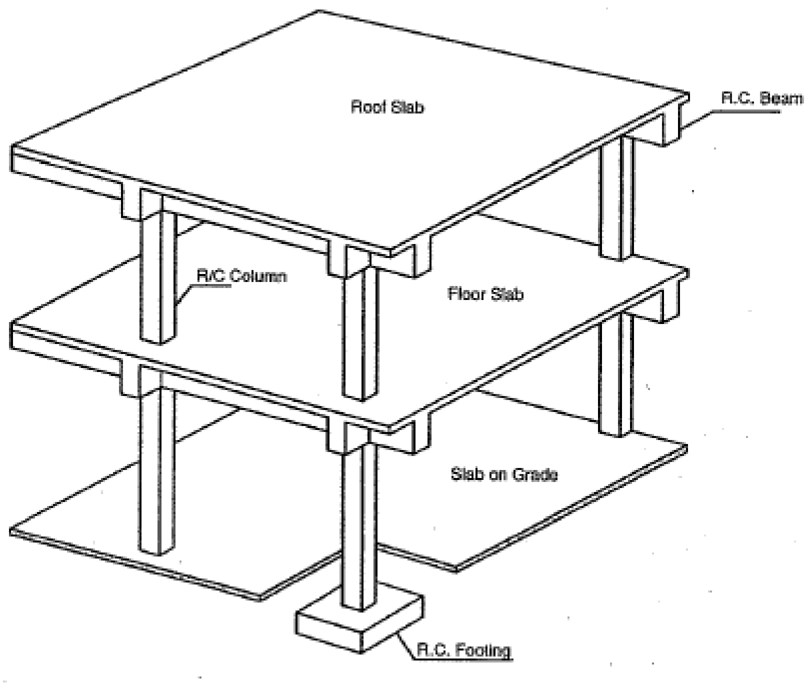


Fig I. 1: Reinforced Concrete Structure **[2].**

I.2. Reinforced concrete structure elements:

I.2.1 Types of slabs:

concrete slab floors have evolved in various manifestations since their inception. Certain iterations exhibited a striking resemblance to their wooden predecessors or wooden surfaces bolstered by steel or iron beams. Conversely, other variations were evidently novel creations, devoid of any identifiable lineage, tailored to accommodate the characteristics of the materials at hand - steel bars and plastic concrete **[4].**

1. Flat Slab : A flat slab, devoid of beams or girders, is a reinforced concrete slab that caters to extensive floor areas necessitating adaptability and straightforward construction.

2. Waffle Slab : Characterized by a grid-like arrangement of ribs and beams generating square or rectangular voids, a waffle slab delivers a commendable strength-to-weight ratio and is ideal for expansive spans.

3. Two-way Slab : Supported on all four sides and adept at bearing loads in both orientations, a two-way slab finds frequent application in structures featuring irregular column distributions.

4. One-way Slab : Relying on support from two opposing sides and designed to bear loads in a singular direction, a one-way slab is commonly chosen for modest spans and un complicated frameworks.

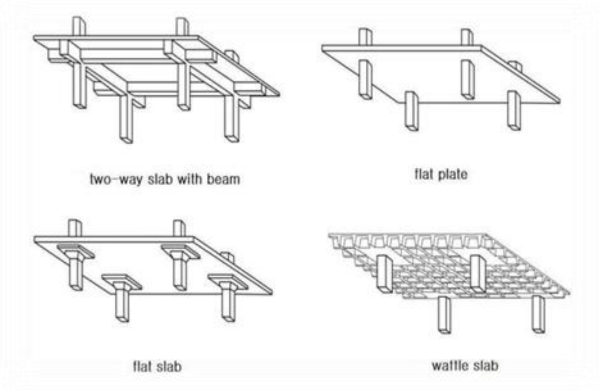


Fig I. 2: Types of tile **[4].**

These examples underscore a fraction of the diversity among concrete slabs utilized in structural edification. The selection of a specific slab variant hinges on factors like span, loadprerequisites, architectural configuration, and construction methodology.

<u>I.2.2 Types of concrete beams:</u>

Reinforced concrete beams are elements of a structure designed to bear the transverse load, typically situated on supports located at its extremities. A girder represents a category of beam that provides support for one or multiple smaller beams **[5].**

Beams are classified as

- Simple Beam
- Continuous Beam
- Semi-Continuous Beam
- cantilever beam
  - T- beam

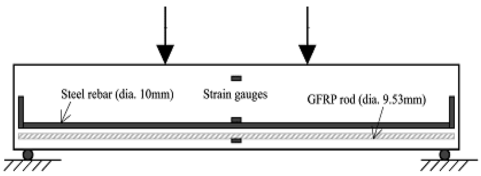
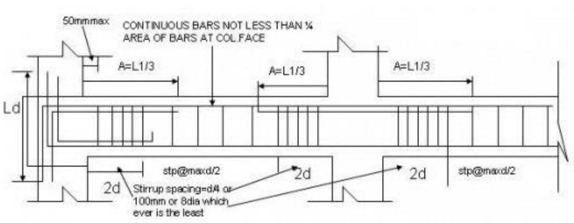


Fig I. 3: Supported Casual **[5].**



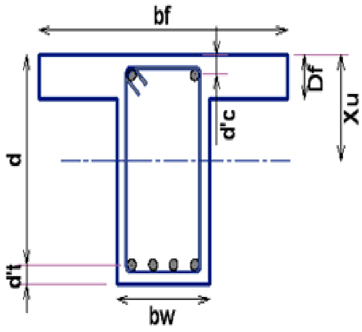


Fig I. 4: Continuous beam with reinforcement details **[5].**

Fig I. 5:RCC T-beam **[5].**

I.2.3 columns:

Vertical load-bearing elements known as columns transmit loads either directly from the slab in the case of a floor-slab or from the beams that provide support to the slab. These columns typically have a square, rectangular, or circular cross-section. The columns can be either cast in place on-site or prefabricated off-site **[6].**

I.2.4. Role of columns:

Support concentrated vertical loads (compression forces).

Contribute to transverse stability through the column-beam system to combat horizontal forces.

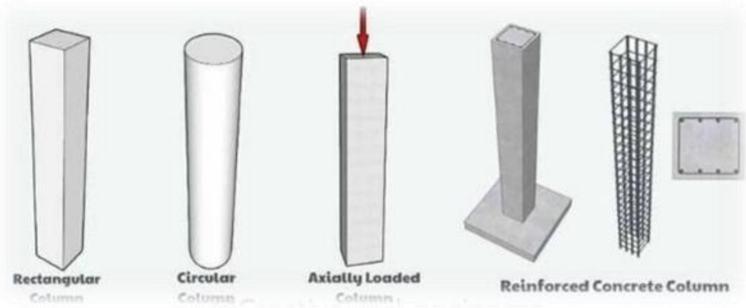


Fig I. 6: Shape Types **[6].**

I.2.5. Foundations:

Foundations are the essential structural elements in any engineering construction, responsible for transferring the loads of the building or structure to the ground. They are designed to evenly distribute the weights onto the soil to avoid uneven settlement or structural failure. Foundations are crucial for ensuring the stability and safety of any building. Foundations can be classified into two main types **[7] :**

a. Shallow Foundations: Used when the surface soil is strong enough to support the structure. These include strip footings, isolated footings, and combined footings.

b. Deep Foundations: Used when the surface soil is weak and cannot support the structural loads. These include piles, caissons, and deep piers.

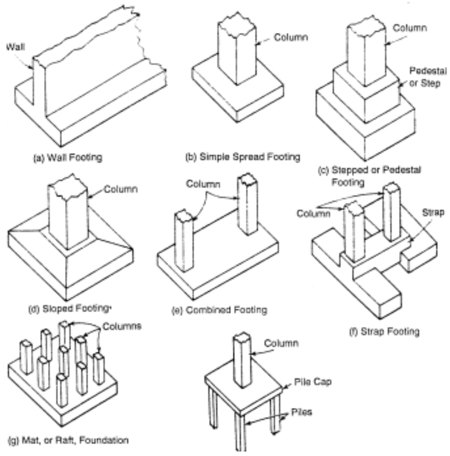


Fig I. 7:types of Foundations **[7].**

The purpose of foundations is to transfer loads from the superstructure to the underlying strong soil or rock layers, ensuring an even and effective load distribution, and minimizing the risks of settlement or collapse **[7].**

I.3. Mechanical characteristics of materials:

The concrete and steel used in the construction of this work will be chosen in accordance with the technical rules for design and calculation of reinforced concrete works, as well as the Algerian seismic regulation.

I.3.1. Cement:

A cementitious substance is defined as possessing the necessary adhesive and cohesive characteristics to unite inert aggregates into a solid mass that exhibits sufficient strength and durability. This category of materials, which holds significant technological value, encompasses not only cements in their true form but also includes limes, asphalts, tars, particularly in the context of road construction, and various other substances.

Hydraulic cements are exclusively utilized in the production of structural concrete. The presence of water is essential for facilitating the chemical process known as hydration, during which the cement powder undergoes setting and hardening to form a unified solid structure. Among the array of hydraulic cements that have been formulated, portland cement stands out as the most prevalent variant, having secured its first patent in England in 1824 **[8]**.

I.3.2 Aggregates:

Aggregates are granular materials commonly used in construction, including sand, gravel, crushed stone, and recycled concrete. They are essential components in the production of concrete, asphalt, and other construction materials, serving to provide bulk, strength, and stability to the final product. Aggregates are classified based on their size, shape, and origin, and they play a critical role in the engineering properties of the composite materials they form **[8].**

Aggregates can be classified based on their size into two main categories: fine aggregates and coarse aggregates. Here's a detailed look at each classification:

a. Fine Aggregates:

Sand: Particles between 0.075 mm to 4.75 mm.

Silt: Particles between 0.002 mm to 0.075 mm.

Clay: Particles smaller than 0.002 mm.

b. Coarse Aggregates:

Gravel: Particles between 4.75 mm to 50 mm.

Cobbles: Particles between 50 mm to 150 mm.

Boulders: Particles larger than 150 mm.

I . 3.3 Concrete:

Concrete is a composite material composed of fine and coarse aggregates bonded together with a fluid cement (cement paste) that hardens over time. It is one of the most widely used construction materials in the world due to its versatility, durability, and strength. Concrete is typically mixed on-site using specific proportions of cement, water, aggregates (such as sand, gravel, or crushed stone), and sometimes additives or admixtures to enhance its properties. Once poured and cured, concrete forms a solid, rigid structure that can be used for various applications, including foundations, slabs, walls, columns, beams, and more **[8].**

- 350 kg of CPA 325 cement.
- 400 kg of sand DS < 5 mm.
- 800 kg of 3/8 and 15/25 Gravel.
- 175.l of fixing water.

I .3.4 The stretch of concrete:

❖ **Compressive stretch:**

The characteristic compressive strength of fcd concrete at days of age is determined from tests on standardized specimens of 16 cm in diameter and 32 cm in height. We most often use the value at 28 days of maturity: f c28. For calculations in the implementation phase, we will adopt the values at d days, defined from f c28 by **[8]**:

- For resistances fc28 ≤ 40MPa:

:

$$\begin{cases} f_{cd} = \dfrac{d}{4.76+0.83d} f_{c28} & \text{if } d < 60 \text{ days} \\ f_{cd} = 1.1 \times f_{c28} & \text{if } d > 60 \text{ days} \end{cases}$$

- For resistances fc28 > 40MPa:

$$\begin{cases} f_{cd} = \dfrac{d}{1.40+0.95d} f_{c28} & \text{if } d < 28 \text{ days} \\ f_{cd} = f_{c28} & \text{if } d > 28 \text{ days} \end{cases}$$

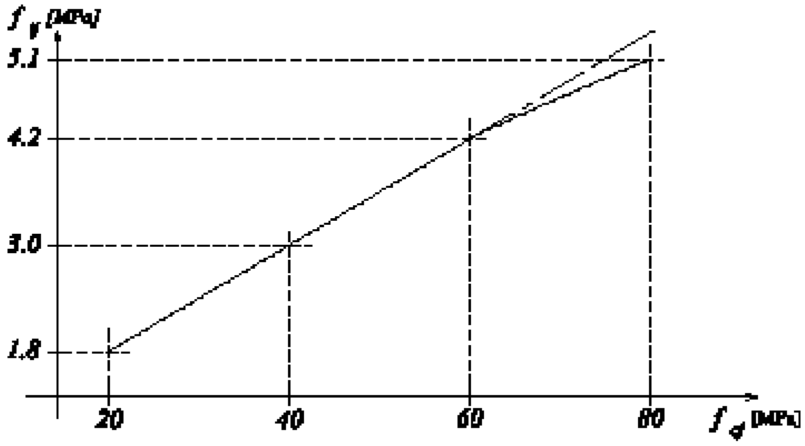
-Compressive strength at 28 days f_{c28} = 25 MPa.

❖ **Tension:**

The characteristic tensile strength of concrete at d-days, denoted ftd, is conventionally defined by the relationships:

ftd = 0.6 + 0.06fcd if fc28 ≤ 60Mpa.

ftd = 0.275(fcd) 2/3 if fc28 ≤ 60Mpa.



Fig(I.7) : Evolution of the resistance of concrete Tensile to f_{td} as a function of that to compression $\mathbf{f_{cd}}$ **[1].**

Tensile strength ft28 = 2.1 MPa

I.3.5.Longitudinal deformation :

✓ Longitudinal Deformation Modulus:

Under normal constraints with an application duration of less than 24 hours, we admit a measurement defect, that at age “d” days, the instantaneous longitudinal deformation modulus of the concrete E_{id} is equal to:

$E_{id} = 11000 \times f_{cd}^{1/3}$ MPa

For f_{c28} = 25 MPa on a E_{id} = 32164.195 MPa.

✓ Deferred Longitudinal Deformation Module:

In the context of long-term application, the variation in the modulus of longitudinal deformation allows for the calculation of the ultimate deformation of the concrete.

$E_{vd} = 3700 \times f_{cd}^{1/3}$

For f_{c28} = 25 MPa on a E_{vd} = 10818.865 MPa .

I.3.6. Poisson coefficient:

υ = 0.0 in the case of Ultimate Limit States (cracked section)
υ = 0.2 in the case of Serviceability Limit States (non cracked)

I.3.7. Limit stresses:

Ultimatum stress of concrete :

For Ultimate Limit States calculations, the parabola-rectangle law on the stress-deformation diagram models the real behavior of the concrete.
The deformations of the concrete are:

- $\varepsilon_{bc1} = 2°/_{00}$
- $\varepsilon_{bc2} = 3.5°/_{00}$ Si $\rightarrow f_{cd} \leq 40$Mpa
- Min(4.5;0.025f_{cd}) $°/_{00}$ Si $\rightarrow f_{cd} \leq 40$Mpa
- The design compressive strength value of concrete f_{bu} is given by:

$$f_{bu} = \frac{0.85 \times f_{cj}}{\theta \times \gamma b}$$

With:

γ_b : Safety factor part l (1.5 for basic combinations and 1.15 for accidental combinations)
µ: a coefficient, which takes into account the duration of application of the loads:

- µ = 1 if the duration is greater than 24 hours
- µ = 0.9 si la durée est comprise entre 1h et 24h.
- µ = 0.85 in other cases.

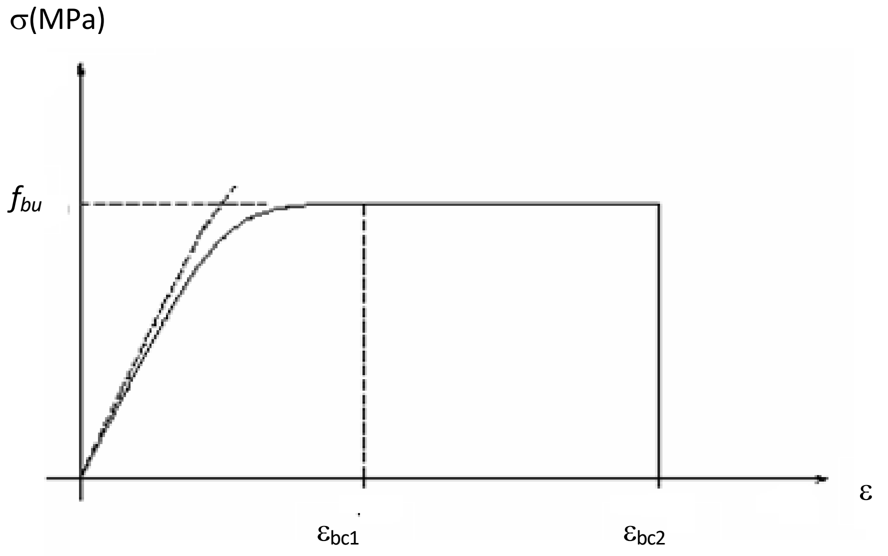


Fig I. 8: Compressive Stress-deformation diagram of concrete at the Ultimate Limit States- **[1].**

Serviceability Lomita States:

The deformations are relatively small, and it is therefore assumed that the concrete remains in the elastic domain. We adopt Hooke's law of elasticity to write the behavior of concrete in this state, with long-term loads. (Eb = Evj and ν= 0.2). The mechanical resistance of tensioned concrete is neglected. In addition, we generally adopt a fixed value for the Young's modulus of concrete equal to 1/15 of that of steel.

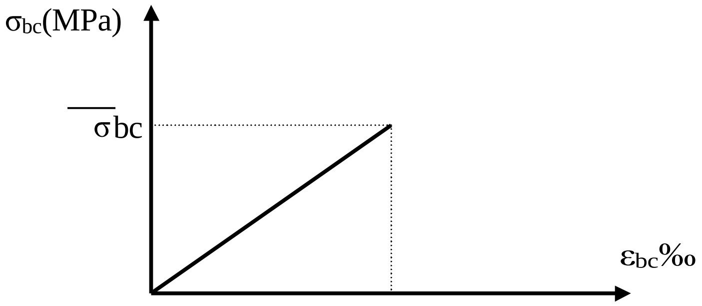


Fig I. 9: Stress deformation diagram of concrete calculation the Serviceability Limit States **[1].**

The compressive service limit stress of the concrete is limited by:

$$\sigma_{bc} \leq \overline{\sigma_{bc}} \qquad \text{with : } \overline{\sigma_{bc}} = 0.6\, f_{c28}$$

I.4. Steel structures:

Steel construction relates to the fields of mechanical and civil engineering, as well as the field of construction in general, where the focus is on Steel construction, especially those made from steel. It is characterised by corrosion-resistant features such as polystyrene and its adaptability, as well as its resistance to environmental factors such as shock when catalysed. It can be moulded and designed into a variety of shapes from different companies. However, steel also has some disadvantages such as its high cost and the need for additional corrosion **[3].**

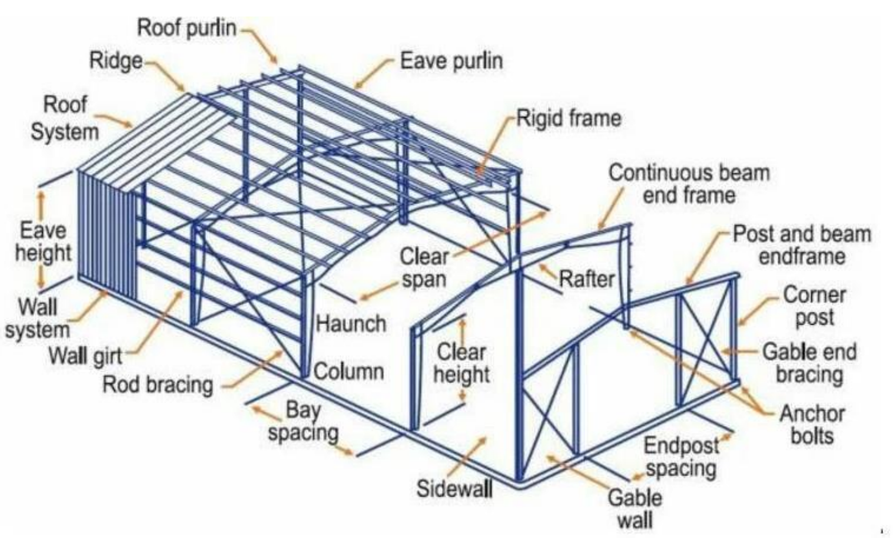


Fig I. 10: Structural steel elements **[9].**

I.4.1. The beams:

a. The I-beam:

There are two types of I-beams:

– IPN: normal I-beams. The flanges are of varying thickness, which difficulties for the fasteners

– IPE: European I-beams. The flanges have parallel edges,the ends are sharp-edged (only the re-entrant angles are rounded) The IPE are a little more expensive, but more practical and are in common use **[10].**

–

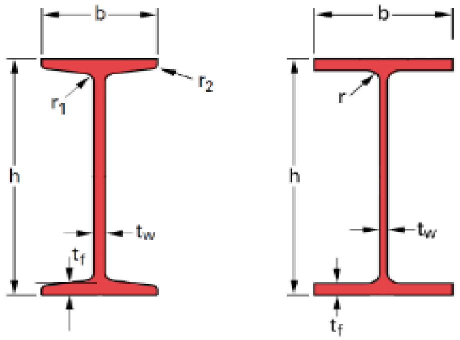


Fig I. 11 :IPNsection geometry,IPE section geometry **[11].**

b. U-beams:

There are also two types of : UPN, UAP and UPE. In the same UPE have parallel edged flanges and are tending to supplant UPN, which are UPN, which are less easy to install. Heights range from 80 to 400 mm **[10].**

c. HE beams:

They are divided into three series: HEA, HEB and HEM, depending on the relative thickness of their core and wings.

thickness of their web and flanges. Their cross-section is approximately square (the width of the web is approximately equal to the height of the profile up to a height of 300 mm). The flanges always have parallel edges. Heights vary from 100 to 1100 mm.

The lightest HEA profiles, which are the lightest, offer the best performance to weight ratio and are therefore the most widely used. The progression of the three series is both technically and architecturally interesting for pro-active components **[10].**

d. Hollow sections:

these are generally used to make the following components:

- Posts: round, square or rectangular;

- Lattice girders: round or square type;

- Diagonals for stability bearings: round type.

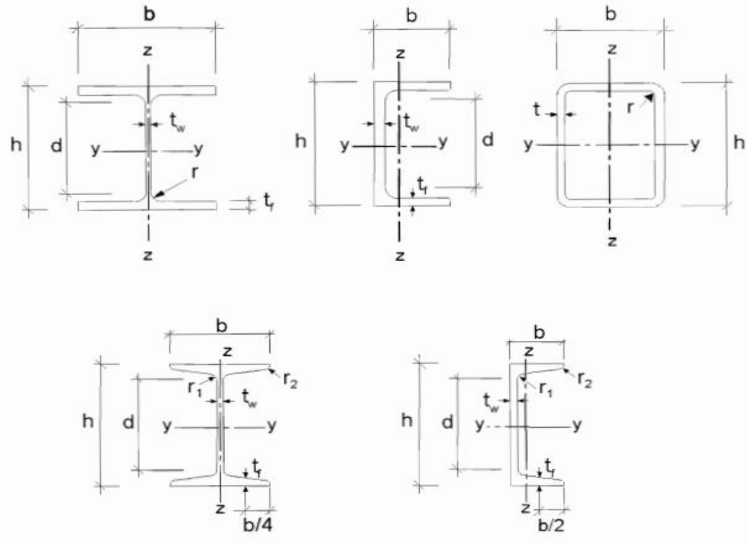


Fig I. 12: Dimensions and axes of sections **[12].**

I.4.2. Materials used:

- I.4.2.1 Steel:

Steel is a material made up essentially of iron and a little carbon, which are extracted from natural raw materials from underground (iron and coal mines).
extracted from natural raw materials found underground (iron and coal mines).
Carbon only plays a very small part in the composition (generally less than
less than 1%).
In addition to iron and carbon, steel may contain other elements that are associated with it, either :
- Involuntarily, such as phosphorus and sulphur, which are impurities that alter the properties of steel alter the properties of steel.
- Voluntarily, such as silicon, manganese, nickel, chromium, etc., which have the property of improving the properties of steel improve the mechanical properties of steels **[13]**.

- I.4.2.2. Steel properties:

✓ Resistance:

The current steel grades and their limit strengths are given in the Euro code 03

Steel grade (EN10025)	**Thickness t in mm**			
	t < 40 mm		40 mm < t < 100 mm	
	fy (N/mm2)	fu(N/mm2)	fy (N/mm2)	fu (N/mm2)
Fe 360	235	360	215	340
Fe 430	275	430	255	410
Fe 510	355	510	355	490

Table I. 1:Nominal values of fy and fu (CCMA97) **[12].**

✓ b. Ductility:

The structural steel chosen must satisfy the following conditions **12] :**

– The report $f_u/f_y > 1.2$

– The ultimate deformation must be greater than 20 times the elastic deformation $(\varepsilon u \geq 20\varepsilon y)$

– At break, the ultimate relative elongation εu must be greater than or equal to 15%. %.

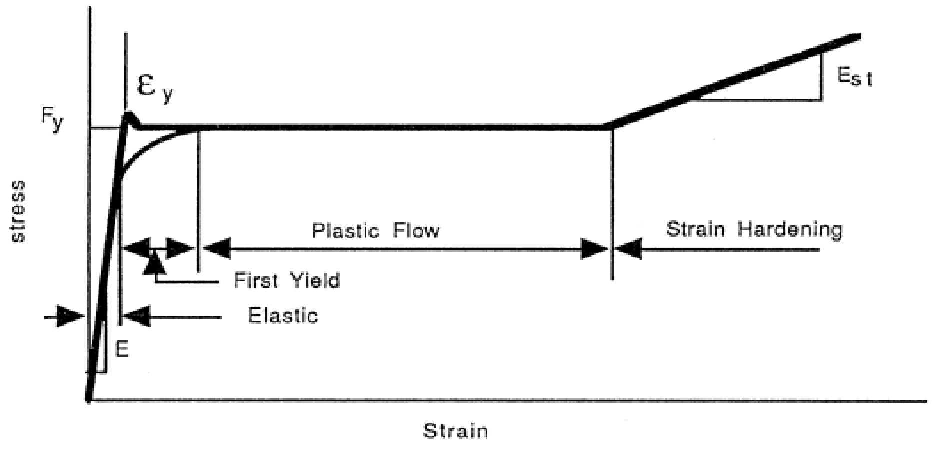


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Fig I. 13:steel deformation diagram **[14].**

✓ c. Mechanical properties:

– Longitudinal modulus of elasticity: E = 210000 MPa.

– Transverse modulus of elasticity: G = E / 2(1+μ).

- Poisson's ratio: μ = 0.3.
- Coefficient of thermal expansion: α = 12x 10-6 per C°.
- Density: ρ = 7850 kg /m.
- Tensile strength: fu = 360 MPa.
- Yield strength: fy = 235 MPa.

I.4.3. Concrete:

For foundations, Concrete is a mixture of sand, cement, gravel and water, the composition of which is adjusted according to the required strength. it is a cost-effective material that offers excellent resistance **[15].**

In our structure, the concrete used in the foundations has the following characteristics:

- Normal density: ρ = 2500 Kg /m3

- Compressive strength: fc_{28}= 25 MPa.

- Tensile strength: $ft_{28} = 0.06 \times fc_{28} + 0.6 = 2.1$ Mpa

I.4.4. Connections:

- 4.4.1. Bolting:

Bolting is the most widely used assembly method in steel construction, because it is easy to use and can be adjusted on site.

Table I. 2: Nominal values of resistance **[7].**

Class	**4.6**	**4.8**	**5.6**	**5.8**	**6.6**	**6.8**	**8.8**	**10.9**
fyb(MPa)	240	302	300	400	360	480	640	900
fub(MPa)	400	400	500	500	600	600	800	1000

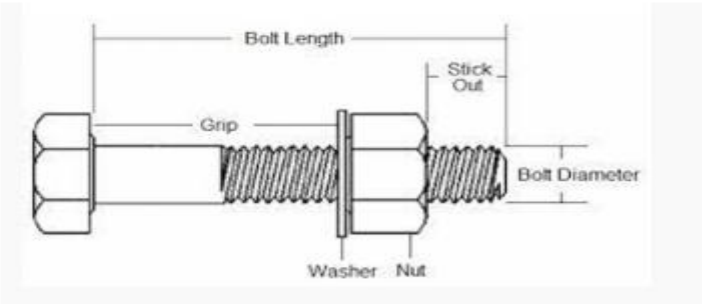


Fig I. 14: Structural screw

I.4.4.2 Welding:

Welding is an operation that involves joining two parts of the same material with a weld bead consisting of a filler metal, which acts as a binder between the 2 parts to be assembled **[3].**

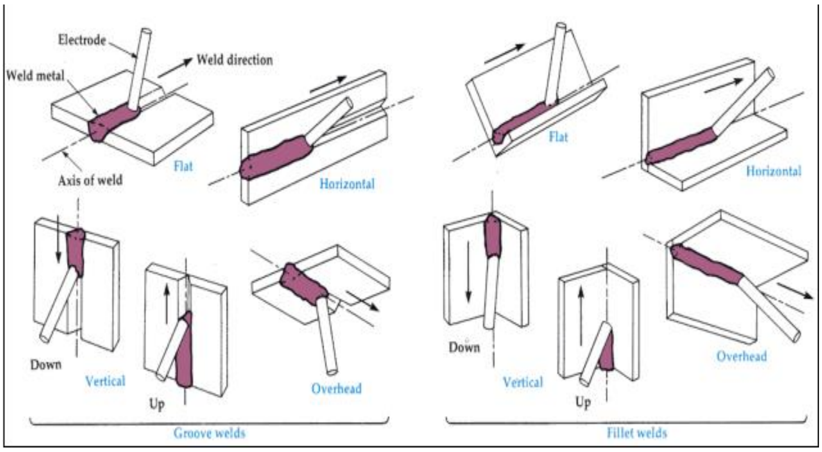


Fig I. 15 : groove welds and fillet

I.4.5. Cases of Haddad:

Limit states are states beyond which the structure no longer meets the performance requirements for which it was designed. Limit states are classified as **[16] :**

- I.4.5.1. Ultimate limit state (ULS):

It corresponds to the mechanical limit beyond which the structure is ruined distinguished as follows **[16] :**

- The ultimate limit state of static equilibrium, which concerns the stability of the structure.
- The ultimate limit state of resistance, which concerns the non-failure of the structure.
- The ultimate limit state of stability of form, which concerns slender parts ubjected, among other things, to axial compression.

Load case: 1.35G+1.5Q snow, wind

- I.4.5.2. Serviceability limit state (SLS):

It corresponds to criteria which, if not met, will prevent the element from being used under satisfactory conditions or will compromise its sustainability **[16] :**

- The serviceability limit state with respect to concrete compression.
- The serviceability limit state for crack opening.
- The serviceability limit state for deformation.
- Load case: G+ Q snow, wind

CHAPTER II:

MULTICRITERIA DECISION MAKING METHODS

Introduction:

Multicriteria Decision Making methods are a set of techniques and approaches used to evaluate and prioritize multiple conflicting criteria in decision-making processes. These methods are widely applied in various fields such as engineering, economics, management, and environmental science to aid in making informed and balanced decisions. Below are some of the most commonly used MCDM methods **[17] :**

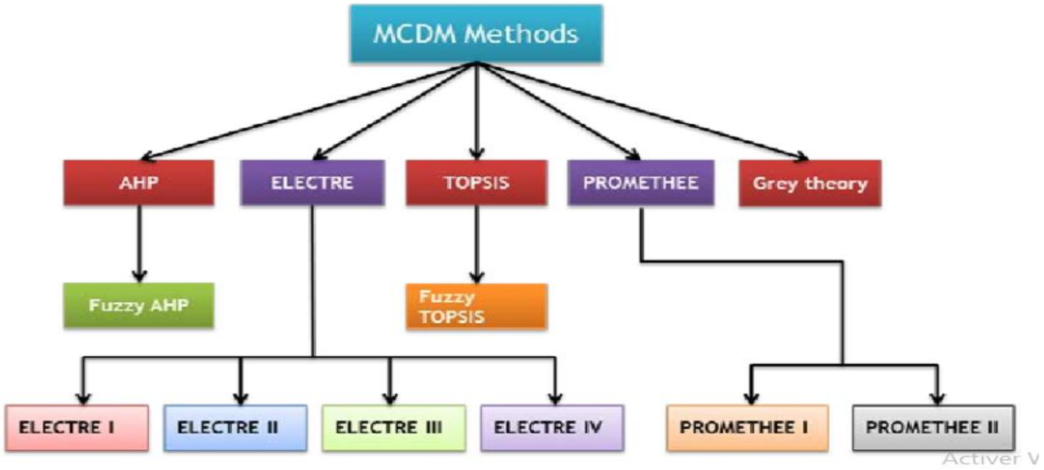


Fig II. 1: Diagram multi-criteria decision-making methods (MCDM**) [18].**

II.1. Analytic Hierarchy Process (AHP):

Description: AHP is a structured technique that involves decomposing a complex decision problem into a hierarchy of more easily comprehended sub-problems, each of which can be analyzed independently. The method uses pairwise comparisons to establish priorities among the criteria and alternatives **[18].**

Key Steps in the AHP Process

a) Define the Problem and Determine the Goal:

Clearly state the decision problem and the ultimate goal to be achieved.

b) Structure the Hierarchy:

Break down the decision problem into a hierarchy of more easily comprehended sub-problems, each of which can be analyzed independently.

c) The hierarchy typically consists of three levels:

the goal at the top, criteria in the middle, and the decision alternatives at the bottom.

d) Construct Pairwise Comparison Matrices:

Compare the elements at each level of the hierarchy pairwise in terms of their impact on an element above them **[18].**

Use a scale of relative importance, usually 1 to 9, where 1 represents equal importance and 9 represents extreme importance.

e) Calculate the Priority Vectors:

Derive the priority vectors (eigenvalues) from the pairwise comparison matrices.

These vectors represent the relative weights of the elements at each level of the hierarchy.

f) Check for Consistency:

Calculate the Consistency Ratio (CR) to ensure that the pairwise comparisons are consistent.

A CR less than 0.1 is generally considered acceptable. If the CR is higher, the pairwise comparisons need to be revised **[18].**

g) Aggregate the Weights:

Synthesize these priorities to determine an overall ranking of the decision alternatives.

Multiply the local priority of each alternative by the priority of the corresponding criterion and sum these products to get the global priority **[19].**

h) Make the Decision:

Select the alternative with the highest global priority as the best decision.

1. Advantages of AHP:

a. Structured Approach: Provides a clear and systematic method for decision-making.

b.Flexibility: Can be applied to a wide range of problems.

c. Consistency Check: Ensures that the decision-making process is logical and coherent.

d. Quantitative and Qualitative Analysis: Combines both types of data effectively **[20].**

2. Disadvantages of AHP:

- ✓ **Complexity:** Can become cumbersome for very large and complex decision problems.
- ✓ **Subjectivity:** Relies on the subjective judgments of decision-makers, which can introduce biases.
- ✓ **Consistency Requirement:** Ensuring consistency can be challenging and time-consuming **[20].**

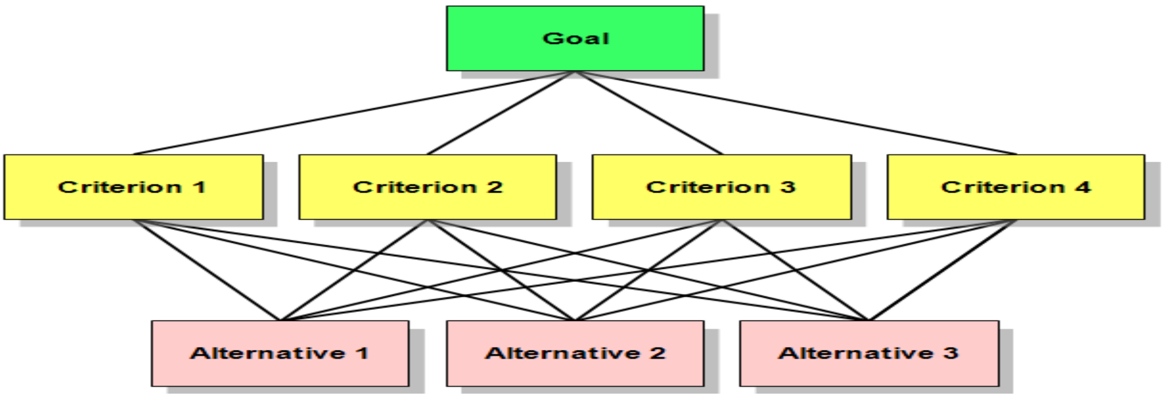


Fig II. 2: Diagram Analytic Hierarchy Process (AHP) **[21].**

II.2. Technique for Order Preference by Similarity to Ideal Solution (TOPSIS) :

The Technique for Order Preference by Similarity to Ideal Solution (TOPSIS) is a multi-criteria decision-making method used to identify the optimal solution from a set of alternatives based on their proximity to an ideal solution. Developed by Hwang and Yoon in 1981, this technique is widely used in fields such as business management, engineering, and scientific research.

Key Steps in the TOPSIS Process **[22].**

a) **Define the Problem, Alternatives, and Criteria:**

Clearly define the decision problem and identify the set of available alternatives and the criteria that will be used to evaluate these alternatives **[22].**

- Construct the Decision Matrix:

Create a decision matrix that contains the evaluation of each alternative with respect to each criterion **[22].**

- Normalize the Decision Matrix:

Normalize the values in the decision matrix to convert them to a common scale.

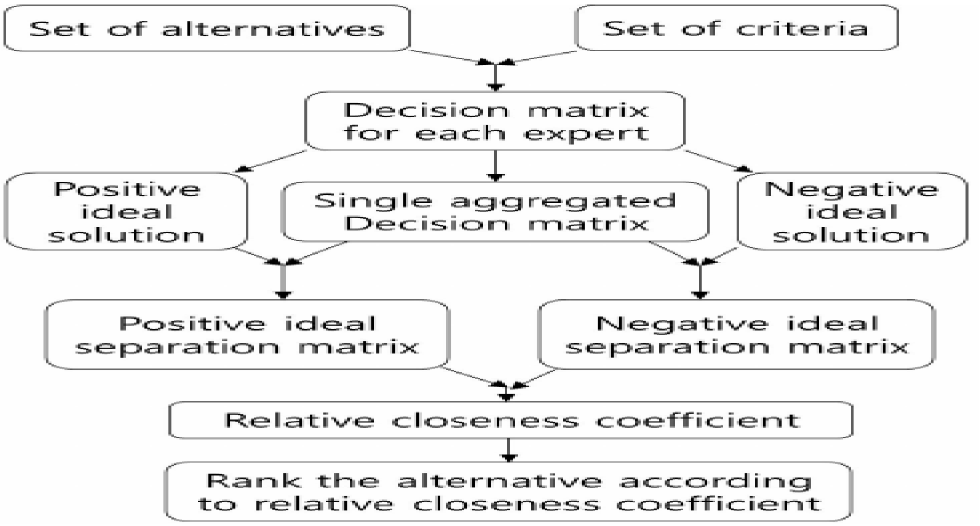


Fig II. 3: Diagram Technique for Order Preference by Similarity to Ideal Solution (TOPSIS) **[23].**

II.3. Elimination and Choice Expressing Reality (ELECTRE):

Description: ELECTRE is a family of MCDM methods that use pairwise comparisons to outrank alternatives based on a set of criteria, considering both concordance and discordance indices **[24].**

Steps:

1) **Construct the Decision Matrix:**

List the alternatives and their performance scores for each criterion.

2) Normalize the Decision Matrix:

Convert the performance scores to a common scale, typically by dividing each score by the maximum score for that criterion **[24].**

3) Calculate the Weighted Normalized Decision Matrix:

Multiply the normalized scores by the weights of the corresponding criteria.

4) Determine Concordance and Discordance Sets:

For each pair of alternatives, determine the concordance set (criteria where one alternative is at least as good as the other) and the discordance set (criteria where one alternative is significantly worse than the other) **[24].**

- ✓ **Calculate Concordance and Discordance Indices:**
- ✓ **Compatibility Index (C):** Sum the weights of the criteria in the concordance set.
- ✓ **Discordance Index (D):** Determine the maximum difference in performance scores for the criteria in the discordance set

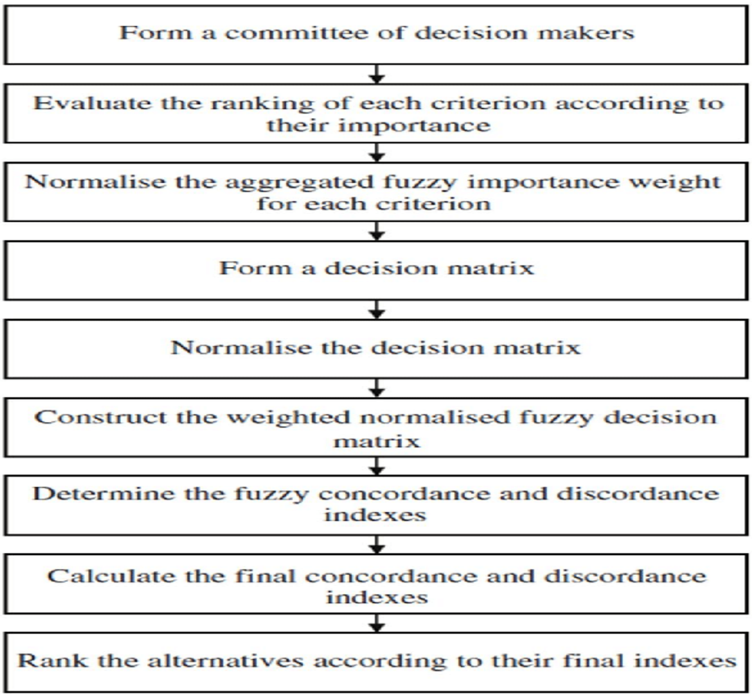


Fig II. 4: Elimination and Choice Expressing Reality (ELECTRE) **[24].**

II.4. Simple Multi-Attribute Rating Technique (SMART) :

Description: SMART is a simplified version of multi-attribute utility theory that involves assigning weights to criteria and scoring each alternative against these criteria **[25].**

Steps:

- ❖ **Identify Alternatives and Criteria:**

List all the alternatives that need to be evaluated.

Identify the criteria that will be used to evaluate the alternatives **[25]**.

- **Normalize the Scores (if necessary):**

Normalize the scores to ensure they are comparable, especially if different scales are used for different criteria.

Normalized score $=\frac{Raw\ score}{Sum\ of\ raw\ scores}$

- **Calculate the Weighted Scores:**

Multiply the normalized score for each criterion by the weight of that criterion to get the weighted score. Weighted score=Normalized score×Weight

- **Rank the Alternatives:**

Rank the alternatives based on their overall scores. The alternative with the highest score is considered the best option **[25].**

II.5. Preference Ranking Organization METHod for Enrichment Evaluations (PROMETHEE):

Description: PROMETHEE is a method based on outranking that helps in ranking and selecting among a set of alternatives by comparing them pairwise with respect to each criterion **[26].**

Steps:

Define Preference Functions:

Choose a preference function for each criterion. Common types include linear, Gaussian, and step functions. The choice depends on the nature of the criteria and the decision-maker's preferences **[26].**

a. **Calculate Preference Degrees:**

For each pair of alternatives and each criterion, calculate the preference degree using the chosen preference function **[26].**

b. **Compute Preference Indices:**

Aggregate the preference degrees across all criteria, weighted by the importance of each criterion, to obtain the preference index for each pair of alternatives **[26]**.

c. **Determine Outranking Flows:**

Calculate the positive, negative, and net flows for each alternative.

d. **Rank the Alternatives:**

Rank the alternatives based on their net flow values. The higher the net flow, the better the alternative **[26].**

II.6. Multi-Attribute Utility Theory (MAUT):

The Multi-Attribute Utility Theory (MAUT) is a decision-making framework that helps individuals or organizations evaluate and make decisions when faced with multiple criteria or factors. This theory is based on the idea that decision-makers have preferences for various factors and that these preferences can be measured and used to determine the best course of action **[27].**

In MAUT, decision-makers identify a set of factors or criteria that are relevant to the decision at hand. These factors can include things like cost, quality, time, risks, and other important considerations. Decision-makers then assign weights to each factor based on their relative importance. They then evaluate the performance or value of each option with respect to each factor **[27].**

MAUT involves calculating the utility function for each factor, which rep

resents the decision-maker's preferences and contributes to determining trade-offs between different levels of that factor. The utility function transforms the performance or value of the alternative into a numerical value that reflects the decision-maker's satisfaction or preference for that level of performance **[27].**

By combining the weighted factors and their utility functions, MAUT generates a total utility value for each alternative. Decision-makers can then compare these total utility values to determine the alternative that provides the highest overall satisfaction or utility.

MAUT is a valuable tool in complex decision-making situations where multiple criteria must be considered at the same time. It helps decision-makers structure their thinking, clarify their preferences, and systematically evaluate and compare different options to make informed decisions **[27].**

II .7. MCDM methods in civil engineering :

Decision-making is applied in different areas of human activities. In the case of existence of at least two possible options, a person (i.e., a decision-maker) has to make a decision and to select the one which is best suited for his demands. Complex problems in science, engineering, technology, or management are characterized by multiple criteria. Usually they are hardly measurable, conflicting or interacting with each other. Decision-making (DM) problems based on multiple criteria are objects of MCDM.

MCDM is a discipline concerned with the theory and methodology for handling problems common in everyday life. They arise in such areas as business, engineering, social organization, and so forth . MCDM has grown as a part of operation research pertaining to the design of computational and mathematical tools for supporting the subjective evaluation of performance criteria by decision-m **[28].**

In order to find the optimum solution to the structural system selection problem, evaluation criteria were determined according to the literature review. In the literature, it is seen that there are plenty of different criteria selected for this decision problem. Yildirim (2003) **[29].** preferred the evaluation criteria for the structural system selection as: the resistance to external conditions, earthquake safety, fire safety, wind resistance, construction energy, material production energy, reuse of the material, number of floors, openings to be passed, usable interior volume, external appearance, interchangeability, disassembly, construction cost, operating costs, labour and construction machine requirement, construction period and service life. Kuzman and Grošelj (2012) **[29].** determined the evaluation criteria as: construction time, construction cost, depreciation costs, design and embedded energy. Balali et al. (2014) **[29].** chose the evaluation criteria for the selection of a suitable structural system for a housing project as: cost, dead load, feasibility, number of coefficients, conservation of energy and lifecycle time. Dagilgan (2019) **[29].** chose the evaluation criteria for the selection of a suitable structural system for crossing wide gaps as: place of use, passable span, suitability for prefabrication, acoustic effect, suitability for installation, natural illumination possibility, system section according to the opening (h/l), joint details, fire resistance, resistance to environmental influences, energy, workmanship and construction equipment requirements, manufacturing and assembly process, lifecycle and disassembly and recycling facilities **[29].**

II.7.1.Some areas of use in civil engineering:

This passage provides an overview of the application of Multi-Criteria Decision-Making (MCDM) approaches in the construction industry. MCDM methods have been widely employed for selecting alternatives related to construction plans, materials, and methods. The text highlights various studies and applications **[29]**:

- **Retrofitting Existing Buildings**: utilized MCDM methods for retrofitting projects **[29].**
- **Bridge Construction**: a fuzzy Analytic Hierarchy Process (AHP) model to evaluate bridge construction methods **[29].**
- **Seismic Retrofitting**: compared different MCDM methods, specifically using TOPSIS and VIKOR, for seismic retrofitting **[29].**
- **Highway Bridge Design**: developed a fuzzy-TOPSIS model for selecting highway bridge superstructure designs **[29].**
- **Sustainability Assessment**: applied the MIVES method to assess the sustainable performance of industrial buildings **[29].**

II.7.2. In the Emerging Fields of Civil Engineering, Multi-Criteria Decision Making (MCDM) :

II.7.2.1. Developing Sustainable and Energy Efficient Building:

Sustainability is a natural subject of MCDM, because it automatically includes three subsets of criteria, involving economics, environmental, and social aspects. When solving problems of sustainable building, the fourth subset of criteria, involving engineering-technological dimensions, is also necessary. One of the innovative themes in sustainable construction is related to using materials of low embodied energy and energy efficient applications **[28].**

II.7.2.2. Possibilities to Apply MCDM Methods within BIM Process:

A development of a large number of the alternatives a_i and characterization of each of them by the criteria c_j can encumber the design process. In essence, a solution of a MCDM problem will require to prepare m different designs of building related to the LOD reached in the design process. Thus, one can say that there is a “mega-uncertainty” related to the number of LODs at which an application of MCDM will be most efficient **[28].**

II.7.3.Method selection :

The large number of existing MCDM methods confuses potential decision makers, resulting in inappropriate pairing of methods and problems. The authors were not suggesting that one MCDM method was better than another, but that one MCDM method could deliver a more robust outcome than another for a specific problem. To recommend a single method for a decisional problem, risk and uncertainty factors needed to be considered. Both performance measures and criteria weights were studied, and sensitivity analysis applied to performance measures and criteria weights to give a recommendation **[30].**

A decision-maker having to rank the alternatives a_i in the presence of uncertainties may face the following problems:

(1) The problem of choice among different representations of uncertainty related to criteria values c_{ij} and weights w_j.

(2)Specification of the weights w_j in the case where they are uncertain quantities.

CHAPTER III:

COMPARATIVE STUDIE OF CONCRETE STRUCTURES AND STEEL STRUCTURES

III.1. Introduction:

In the field of civil engineering and construction, the choice of structural materials plays a crucial role in determining the performance, durability, and overall success of a project. Among the most commonly used materials are concrete and steel, each offering unique advantages and disadvantages. Concrete structures, known for their compressive strength and durability, are widely used in buildings, bridges, and infrastructure. On the other hand, steel structures are recognized for their tensile strength, flexibility, and speed of construction **[31].**

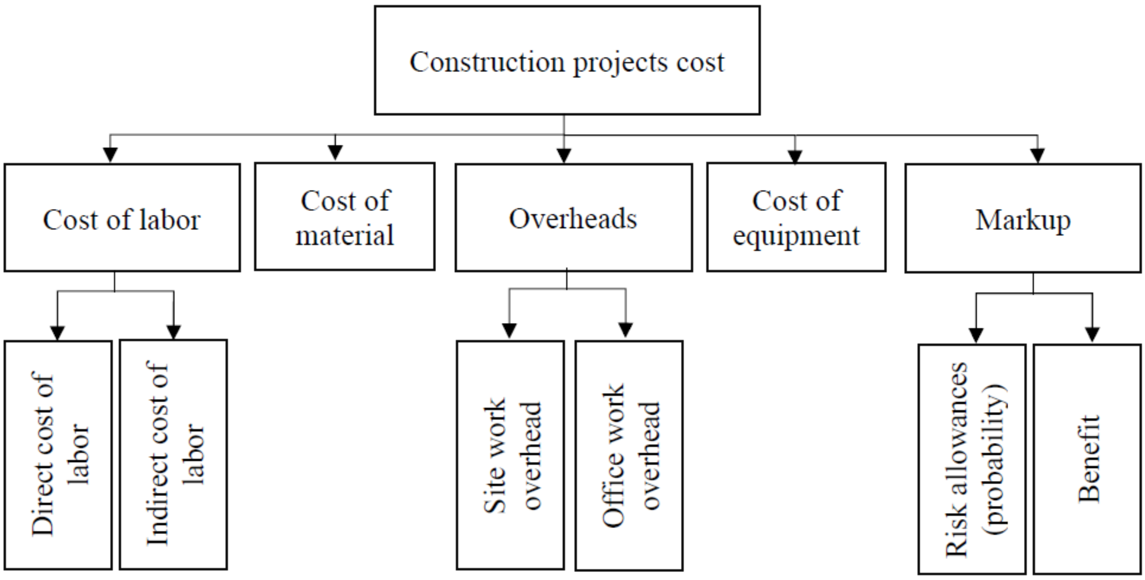
This comparison aims to explore the key differences between concrete and steel structures, considering factors such as load-bearing capacity, construction time, cost-effectiveness, maintenance requirements, and environmental impact. By understanding these distinctions, engineers and architects can make informed decisions that align with the specific needs of their projects, ensuring safety, efficiency, and sustainability in the built environment **[31].**

III.2. Construction Cost Estimation:

The financial assessment of steel and reinforced concrete frame structures is conducted through a specific methodology, taking into account the prevailing market rates for labor, materials, and other pertinent resources, as well as the potential challenges faced by certain firms engaged in the fabrication of steel structures. Consequently, the preliminary estimated expenditure is of significant importance in any construction endeavor, enabling architects and stakeholders to evaluate the feasibility of the project and accurately manage financial resources. articulated that cost estimators formulate their estimates by considering these stages:

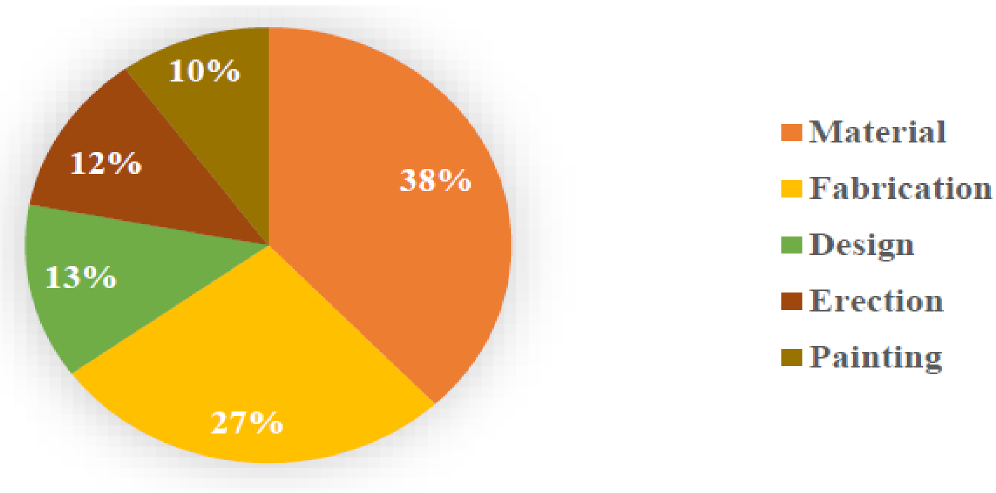
- Specify quantitative information required in project details for measuring.
- Evaluate the collected information.
- Create schedules.

A construction project is fundamentally comprised of two distinct phases: the structural framework phase and the finishing phase, each of which encompasses critical elements that substantially impact the comprehensive expenses associated with the construction endeavor. The vital factors that considerably influence construction expenditures encompass the height of the stories, the characteristics of the soil beneath the foundation, the spacing between columns, the type of slab utilized, and the specific context of the construction site. Accurate cost estimation is essential for enhancing the efficiency of cost-saving measures within construction operations **[31].**



FigIII. 1: Analysis of projects construction costs **[31].**

The financial assessment associated with steel construction frameworks is markedly distinct from the projected cost calculations pertaining to reinforced concrete (RC) construction frameworks, as specific designs are necessitated. The connections represent a critical element within the architectural design and implementation of steel frame structures. It is estimated that the expenditure related to connections constitutes approximately 50% of the total cost of steel structural frame **[31].**



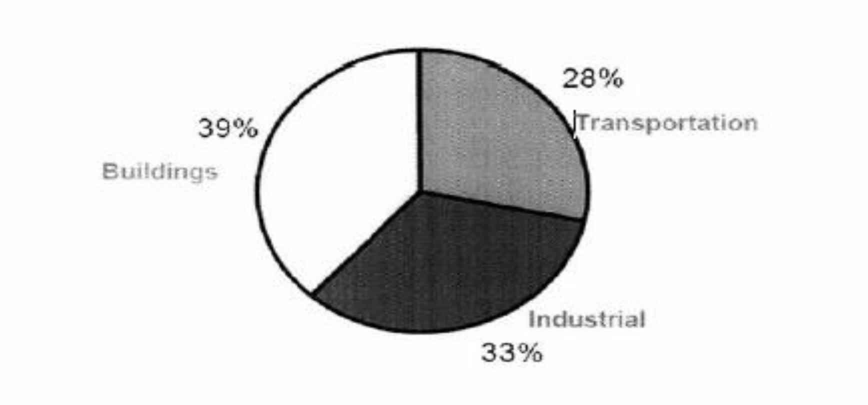
FigIII. 2:Percentage of resources and activities cost in the construction of steel structures **[31].**

III.3. Environmental Impacts Defined:

The three environmental concerns that serve as the focus for this study are energy consumption, harmful air emissions and their impact on global warming,and depletion of the limited supply of natural resources **[31].**

3.1.Energy Consumption:

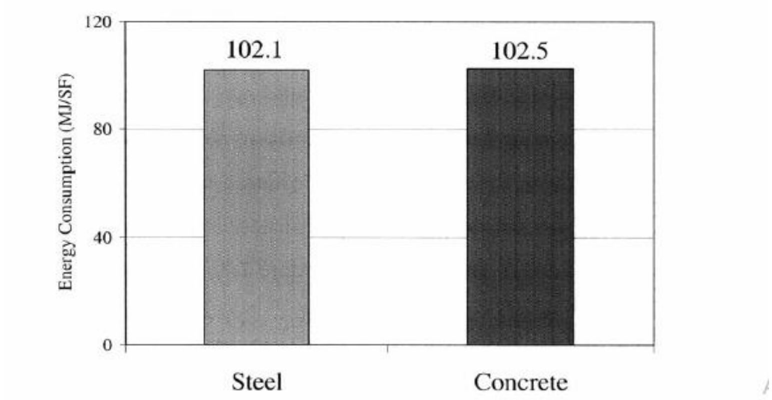
Rising energy prices have a drastic effect on the built environment. Buildings are a large consumer of energy in the United States, accounting for **39%** of total energy consumption.



FigIII. 3: United States Energy Consumption **[32].**

- Energy Consumption Comparison:

In the context of overall energy consumption, steel and concrete (as fundamental construction materials) exhibit commensurate energy prerequisites during the pre-utilization phase, taking into account the variability associated with the input data. The comprehensive energy consumption delineated in this investigation corresponds directly to the embodied energy of the particular building category specified by the functional unit. The aggregate embodied energy for both materials exceeds 10 Tera-Joules (TJ) per complete structure, exhibiting a negligible variance of less than 1%. The juxtaposition of the two categories of building materials **[32].**



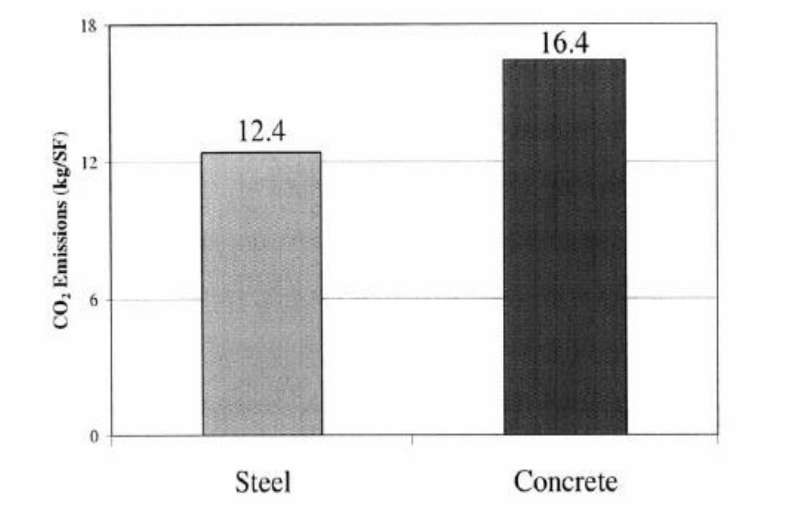
FigIII. 4: Energy Consumption Comparison **[32].**

- Air emissions:

Harmful air emissions have become a major issue in today's world due to the effects of global warming and discernable climate changes over the past decade. According to the Natural Resources Defense Council (NRDC), global warming is the most complex environmental issue of our time. (2006). The Kyoto Protocol of 1997 placed CO2 emission, in particular, at the forefront of environmental policy issues. The facts are that the building industry is the largest contributor to the total upstream CO2 emissions **[32].**

- Emission Comparison:

The raw data results indicate that concrete has a 25% greater impact on CO 2 emissions than steel but both are on the same order of magnitude (x10 6 kg of CO 2 for the functional unit defined) **[32].**

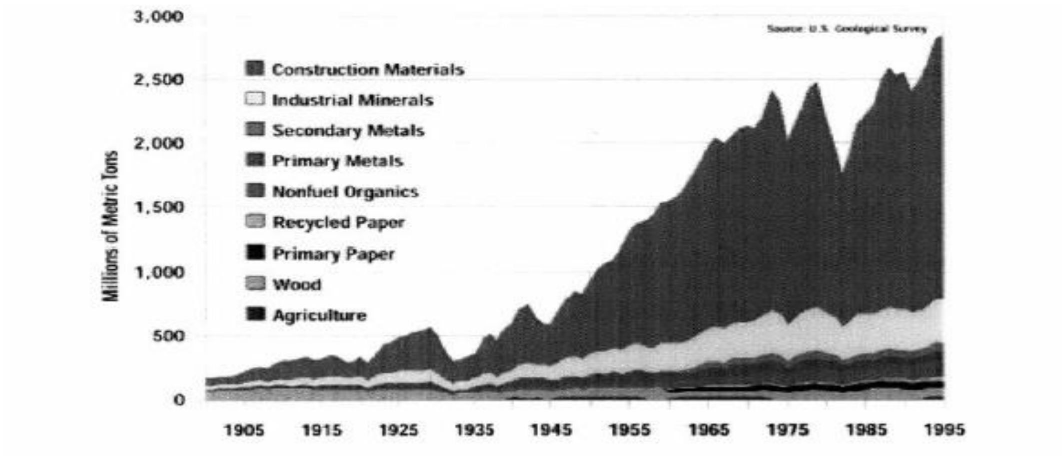


FigIII. 5:CO 2 Emission Comparison **[32].**

Carbon dioxide (CO2) is emitted in nearly all unit operations, either as a direct consequence of a chemical reaction occurring during the production phases or through the combustion of fossil fuels utilized for kiln heating or to supply the electrical energy necessary for the operation of the production facility. The procedure of calcination is essential to the synthesis of Portland cement, as it generates CO2 as a byproduct of the chemical transformation. Consequently, it is reasonable to anticipate that concrete exerts a more significant influence on CO2 emissions compared to steel **[32].**

- Resource Depletion:

The construction industry accounts for a vast majority of the raw materials consumed in the United States, as shown by Fig This enormous consumption rate, nearly two billion metric tons per year, poses a major environmental challenge because of the limited supply of natural resources on hand. The extraction and use of natural resources has significant potential impact on the environment.**[32].**



FigIII. 6: Raw Material Consumption in the United States by Sector, 1900-199**5** **[32].**

3.2.Resource Depletion Comparison:

When comparing concrete structures to steel structures in terms of raw material depletion, it can be said that both types have different impacts on the environment and resource consumption **[32].**

Concrete Structures: They require large quantities of cement, sand, and gravel, which need to be extracted from quarries, potentially leading to resource depletion and environmental degradation **[32].**

Steel Structures: They rely on iron as the primary raw material, which requires mining and smelting processes, consuming energy and impacting the environment in a different way **[32].**

Overall, it can be considered that steel structures deplete raw materials more in terms of the energy used in production, while concrete structures deplete natural resources more in terms of quantity. However, the final assessment depends on many factors such as design, usage, and production methods **[32].**

III.4. Durability:

Durability is one of the fundamental characteristics that determine the quality and performance of materials and structures in architectural and civil engineering. It refers to the ability of a material or structure to withstand environmental conditions and temporal changes without suffering damage or failure. Durability plays a vital role in determining the lifespan and efficiency of constructions, which directly affects maintenance and operational costs **[32].**

The durability of materials is influenced by several factors, including the type of material itself, surrounding environmental conditions, and the construction methods used. For example, concrete is known for its ability to resist weather conditions, while steel possesses high strength but requires protection against corrosion.

In this context, understanding durability becomes crucial when selecting materials and designing structures to ensure their sustainability and safety over the long term **[32].**

A. comparative analysis of the durability :

When undertaking a comparative analysis of the durability of concrete and steel structures, numerous factors warrant consideration:

- ✓ **Concrete Structures :**

1. Longevity: Concrete is widely recognized for its extended lifespan, frequently surpassing 50 years when subjected to appropriate maintenance practices **[33].**

2. Weather Resistance: Concrete possesses the capacity to endure severe meteorological conditions, including precipitation, snowfall, and ultraviolet radiation, without experiencing substantial degradation **[33].**

3. Corrosion Resistance: Although concrete itself is not subject to corrosion, it may be vulnerable to fissuring and spalling, particularly when exposed to freeze-thaw cycles or de-icing agents **[33].**

4. Maintenance: In general, concrete structures necessitate less frequent maintenance compared to their steel counterparts; however, the emergence of cracks over time may necessitate remedial interventions **[33].**

- ✓ **Steel Structures:**

1. Strength-to-Weight Ratio: Steel exhibits a superior strength-to-weight ratio, facilitating the construction of lighter infrastructures that can support substantial loads **[33].**

2. Corrosion: Steel is inherently susceptible to corrosion when subjected to moisture and atmospheric oxygen unless adequately treated (e.g., through galvanization or coating with paint) **[33].**

3. Fatigue Resistance: Steel may demonstrate increased susceptibility to fatigue over time, particularly in structures that are subject to dynamic loading conditions (such as bridges) **[33].**

4. Maintenance: Steel structures demand systematic inspections and maintenance to avert corrosion and maintain structural integrity **[33].**

- Concrete is generally regarded as more durable in terms of resistance to weathering and longevity, necessitating less frequent maintenance interventions **[33].**

- Steel provides superior strength but requires meticulous treatment to mitigate corrosion risks and regular maintenance to uphold its durability **[33].**

Ultimately, the determination of whether to utilize concrete or steel will be contingent upon the specific application, prevailing environmental conditions, and the anticipated lifespan of the structure **[33].**

CHAPTER IV :

THE CASE STUDY DESIGN, CALCULATIONS AND ANALYSIS

IV.1.presentation of the project and calculation hypotheses:

IV.1.1.Presentation of the project :

The work studied is multipurpose hangar aaffiliated to the Naftal complex, located in the state of Ghuardia

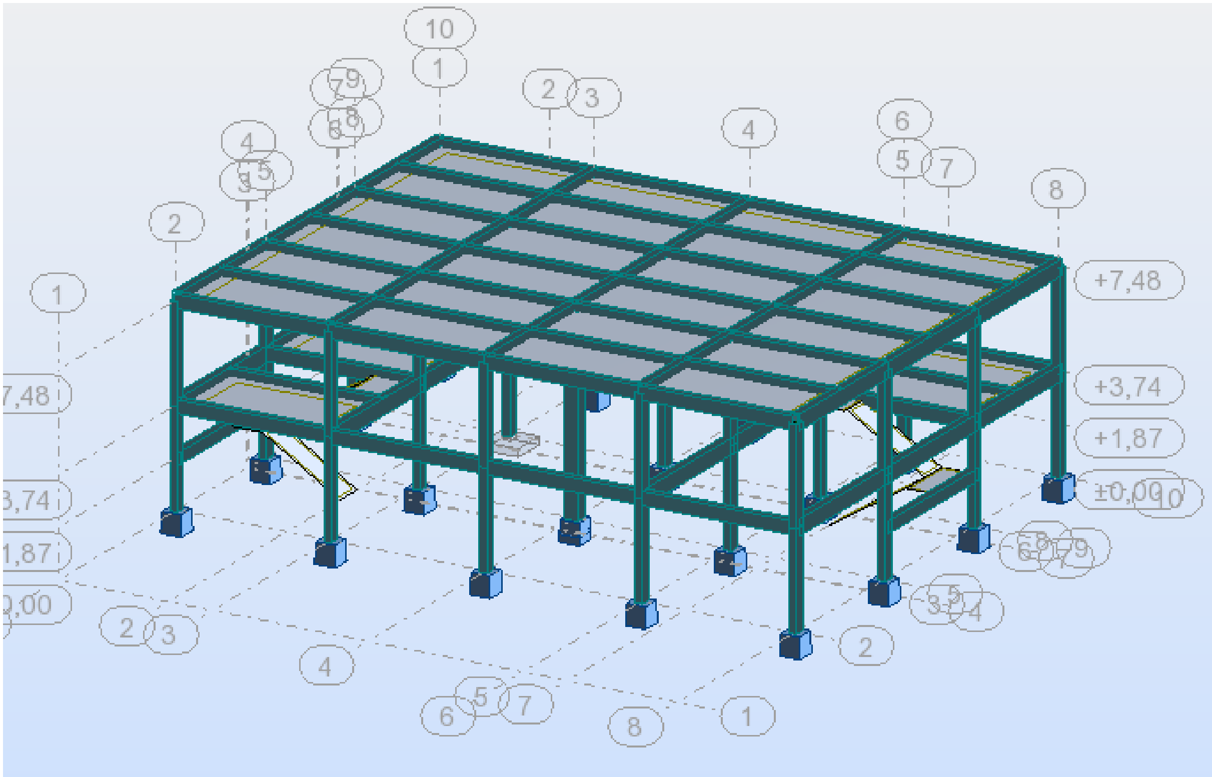
a:Details of the structure :

- Width in plan **18.00m**
- Lent in plan **24.00m**
- Number of levees 02 **: G +1**
- Height of the ground floor **3.74m**
- Leigh of the flour 3.74m
- Total height of the building (without parapet) **7.48m**

B.Site data :

- multipurpose hangar is located in the Wiley of Guardia therefore in a zone of negligible seismicity (Zone 0) according to the classification of RPA99/Version 2003.
- Multipurpose hangar is for industrial use and therefore belongs to use group 1B.

- The site is considered a rocky site (S1) according to the soil study.



FigIV. 1: Modelisation of the structure **(no need).**

IV.2: LOAD DESCENT AND PRE-SIZING OF ELEMENTS STRUCTURAL AND SECONDARY

IV.2.1. INTRODUCTION:

The purpose of pre-sizing is the preliminary calculation of the various resistant elements while respecting the requirements of RPA99/Version 2003 and CBA93.

IV.2.2. Regulatory loads:

The regulatory loads taken into account are:

- Permanent loads representing dead weight.
- Operating loads or live loads.

IV.2.2.1.Dead loads:

This involves taking into account the actual weight of the elements used to build the building. Once again, in order to standardize and facilitate calculation procedures, the legislator provides lists of volumetric weights based on the materials used. These lists are available in the Technical Regulatory Document (D.T.R B.C. 2.2) for permanent loads and operating loads.

IV.2.2.2.Live loads:

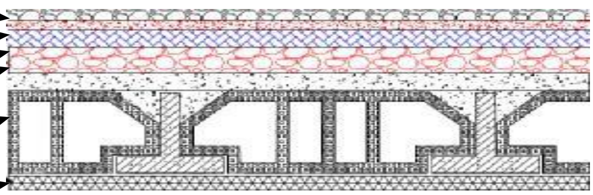
Every building falls into a regulatory category and must be able to withstand the loads and stresses corresponding to "normal" use. It is easy to understand that the floor of a residential building, for example, is generally less loaded than the floor of a library.

To facilitate the consideration of these loads without having to recalculate them systematically, the legislator has chosen to define regulatory loads. These are presented in the Regulatory Technical Document (D.T.R B.C. 2.2) for permanent loads and operating loads.

IV.2.3.Lowering of loads:

IV.2.3.1. Inaccessible Terrace flour:

- Bastard mortar (5 cm)
- Gravel layer 8/15 (2cm)
- Sand thermal insulation (10cm
- Cement mortar screed (2 cm)
- -Cavity floor: (16+4) cm
- Gypsum plaster (1,5 cm)



FigIV. 2: Diagram of a terrace flour

- Bastard mortar (5 cm) :2200 × 0,05 ……….:…………1.10 KN/m²
- Gravel layer 8/15 (2cm) :…………………………………0.30 KN/m²

- Sand thermal insulation (10cm): 1600 × 0.10……………1.60 .KN /m²

- Cement mortar screed (2 cm) :2200 × 0.02……………......0.44 KN /m²

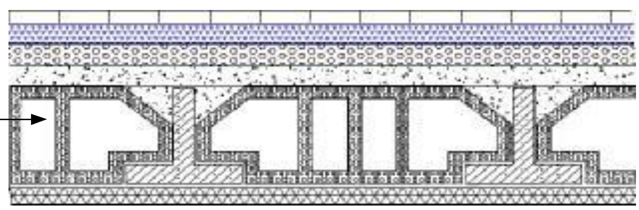
-Cavity floor: (16+4) cm…………………………………......2.85 KN /m²

- Gypsum plaster (1,5 cm) :1400 × 0.015………………………0.21 KN/ m²

G = 6.5kN/m². **Q = 1.00kN/m²**

IV.2.3.2. Grounds flour:

- Tiles (0.02m)
- Mortar scare (0.02m)
- Sand bead (0.03m)
- hollow body slab(0.20m)
- Plaster coating(0.02m)



FigIV. 3: Ground floor **[34].**

Tiles ……………………………………………… 0.02 × 22.00 =0. 44kN/m²
Mortar screed ……….…………………….... ……. 0.02 ×20.00 =0. 40kN/m²
Sand bed…………………………………..……… 0.03 ×18.00 =0. 54kN/m²
Hollow body slab……………………………..………………. 2.85kN/m²
Plaster coating…………..…………………..…0.02 ×10.00 =0. 20 kN/m²
Lightpartitions…………………………………………………...1.00kN/m²
G = 5.43 kN/m². **Q = 2.50 kN/m**

IV.2.3.3. Exterior masonry walls:

The masonry used is brick (double partition) with 30% openings:

Exterior coating .... 0.02 × 20.00 =0.40kN/m²

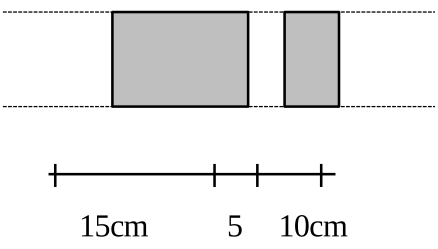
Hollow bricks ....0.15 × 14.00 = 2.10Kn/m²

Hollow bricks ....0.10 × 14.00 = 1.40kN/m²

Interior coating ....0.015 × 12.00 = 0.18kN/m²

Σ = 4.08 kN/m²

With 30% opening:…………..408 ×0.007 = 2.856kN/m² **G = 2.856kN/m²**

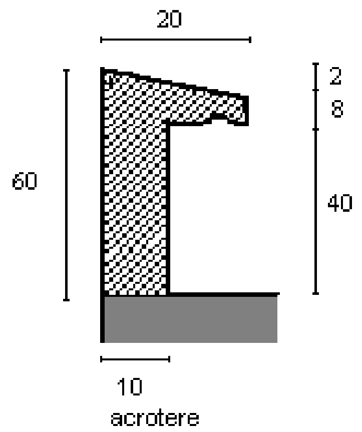


FigIV. 4: Double partition wall diagram

IV.2.4. Parapets:

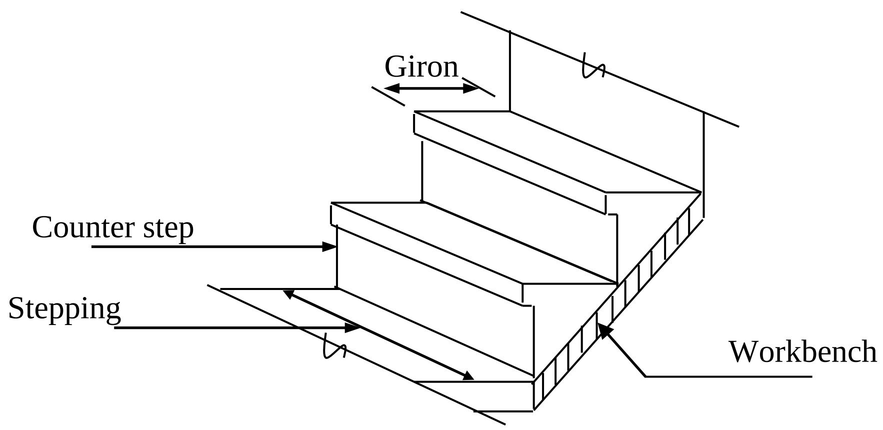
The surface :S = $\frac{0.02\times0.1}{2}$+ (0.1×0.08) + (0.1×0.6) = 0.0025 + 0.005 + 0.06 = 0.0690 m2

Weight : **G = 0.0690×25 = 1.72 KN/ml** **Q = 0.90kN/ml**



FigIV. 5: Diagram of the parapet **[1]**.

IV.2.5. Stairs:



FigIV. 6: Diagram of a staircase **[1]**.

IV.2.5.1. Landing :

Tile .... 0.02 ×22.00 = 0.44KN/m²

Mortar screed.. 0.02 ×20.00 =0. 40KN/m²

Sand bed .... 0.02 ×18.00 = 0.36KN/m²

Solid slab .... 0.15 ×25.00 = 3.75KN/m²

Cement coating... 0.02 ×20.00 = 0.40KN/m²

G = 5.35KN / m². **Q = 2.50 KN / m²**

IV.2.5.2. Flight:

Tile .... 0.02 ×22.00 =0. 44KN/m²

Mortar screed.. 0.02 ×20.00 = 0.40KN/m²

Weight of steps…………………………………0.17 ×22.00 /2 = 1.87KN/m²

flight…………...………….....................0.15 × 25.00 / cos33.45 = 4.49KN/m²

Plastercoating……..0.02×10.00=0.20KN/m²

Bodyguard... 0. 15kN/m

G = 7.52 KN / m². **Q = 2.50 KN / m²**

IV.3. PRE-SIZING OF STRUCTURAL AND SECONDARY ELEMENTS:

IV.3.1. Pre-sizing of floors:

To determine the thickness of the hollow body floor, we use the deflection condition (BAEL 91)

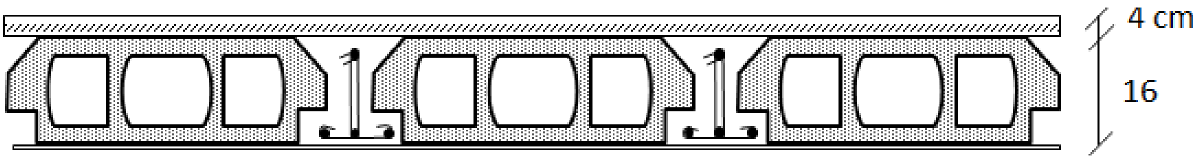
IV.3.2. Introduction :

The purpose of pre-sizing is the preliminary calculation of the different elements resistant. Complying with the requirements of RPA99/Version 2003, CBA93 and BAEL 91.
The results obtained are not final, they can be increased after verification in the sizing phase **[35].**

IV.3.3. Pre-sizing of secondary elements :

IV.3.3. 1.The floors :

The floors, whatever their nature, they transmit to the supporting elements (sails, walls, column and beams) permanent loads and operating surcharges. They serve also to the transmission of horizontal forces. In our case we opt for floors with hollow bodies which are made up of: slabs, beams and a compression slab inreinforced concre te (**FigIV.7**).



FigIV. 7: shape of a hollow body floor **[1].**

The thickness of the floor is determined from the deflection condition according to

(CBA93 Art B.6.8.4.2.4)

e ≥ min (L x max, L y max) / 22.5

e ≥ min (6.00m, 3.00m) / 22.5

$e \geq \frac{300}{22.5} \Rightarrow e \geq 13.33$

We adopt a floor of a thickness:

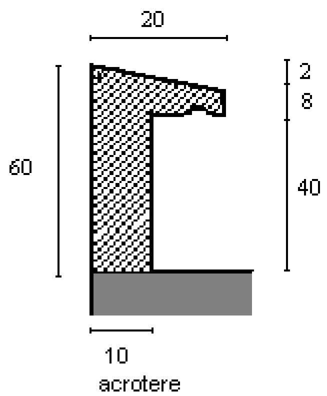
$h_t = 20\text{ cm} : \begin{cases} 16\ cm: \textit{the thickness of hollow body} \\ 4cm: \quad \textit{compression slab} \end{cases}$

IV.3.3. 2.The parapet :

The parapet is likened to a vertical console embedded at its base in the floor terrace, its role is to ensure total security at the terrace level and to protect the gravel from the wind. It is stressed in compound flexion under the action of its weight own “G” and the horizontal action due to the handrail.

The surface :$S = \frac{0.02 \times 0.1}{2} + (0.1 \times 0.08) + (0.1 \times 0.6) = 0.0025 + 0.005 + 0.06 = 0.0690$ m2

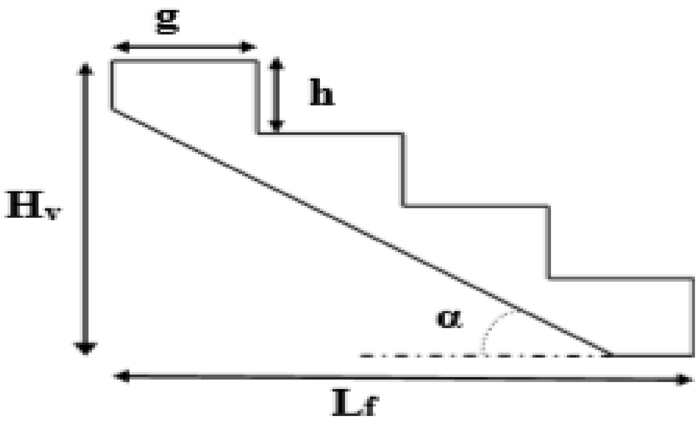
Weight : G = 0.0690×25 = 1.72 KN/ml.



FigIV. 8: shape of a parapet **[1].**

<u>IV.3.4.The stairs :</u>

Stairs are elements made up of a succession of steps allowing the Passage from one level to another, they are made of reinforced concrete, steel or wood. In our case they are made of concrete poured on site.



FigIV. 9: Dimensions of the staircase **[34].**

IV.3.4.1.Technical characteristic :

IV.3.4.2.For current floor :

Floor height: =H = 3.74 m

Giron:g=30cm

Height of the step from the BLONDEL formula:

We have: 59<2h+g<66 => 14.5<h<18h:

h :varied from 14 cm to 20 cm.

g: varied from 22 cm to 33 cm

For: h=17 cm

IV.3.4.3. Number of counter steps:

n=$\frac{H}{h} = \frac{374}{17} = 22$

There are two flights, we will have 18 counter steps, So: there are 9 counter steps in each stolen.

IV.3.4.4.Number of steps in each flight :

n=n-1=11-1=10 steps

IV.3.4.5. Landing:

Length of landing L=1.20m

The flight height there are two flights therefore: L=H/2=3.74/2=1.87m

IV.3.4.5. Bench :

Tg α=H′/L′

$H'=n\times h=11\times 17=1.87m$

$L'=(n-1)\times g \Longrightarrow L'=(11-1)\times 30 \Longrightarrow L'=3.00m$

$tg\alpha=1.87/3.00=0.623$ $\alpha=31.93^{\circ}$

The length of the flight is: $L=1.87/\sin\alpha$ $L=3.53m$

The thickness of the bench is: $L/30 \leq e \leq L/20 \Longrightarrow$ $11.76 \leq e \leq 17.65 \Longrightarrow e=15cm$

IV.3.5.Pre-sizing of main elements :

IV.3.5.1.Beams :

The beams are horizontal elements of reinforced concrete cast on site supporting the loads and overloads. Their pre-sizing is carried out using formulas given by the BAEL91 and verifies the dimensions given by RPA Version 2003.

We distinguish the main beams which constitute supports for the beams and the secondary beams which ensure the chaining.

According to BAEL91 : $\begin{cases} \frac{L}{15} < h < \frac{L}{10} \\ 0.3h < b < 0.8h \end{cases}$ **[36].**

IV.3.5.1.1.Verifications:

According to RPA99 (version 2003) [article 7.5.1 P64] : $\begin{cases} b \geq 20 \text{ cm} \ldots\ldots\ldots\ldots \text{checked} \\ h \geq 30 \text{ cm} \ldots\ldots\ldots\ldots \text{checked} \\ \frac{h}{b} \leq 4 \ldots\ldots\ldots\ldots\ldots\ldots \text{checked} \end{cases}$

IV.3.5.1.2.Main beam :

According to BAEL 91 we have : L = 6.00 m

Height : $\frac{600}{15} \leq h \leq \frac{600}{10}$

$40cm \leq h \leq 60cm$ we take h =60 cm

Width: $0.3\times 60 \leq b \leq 0.8\times 60$

$18cm \leq b \leq 48cm$ we take b =30 cm

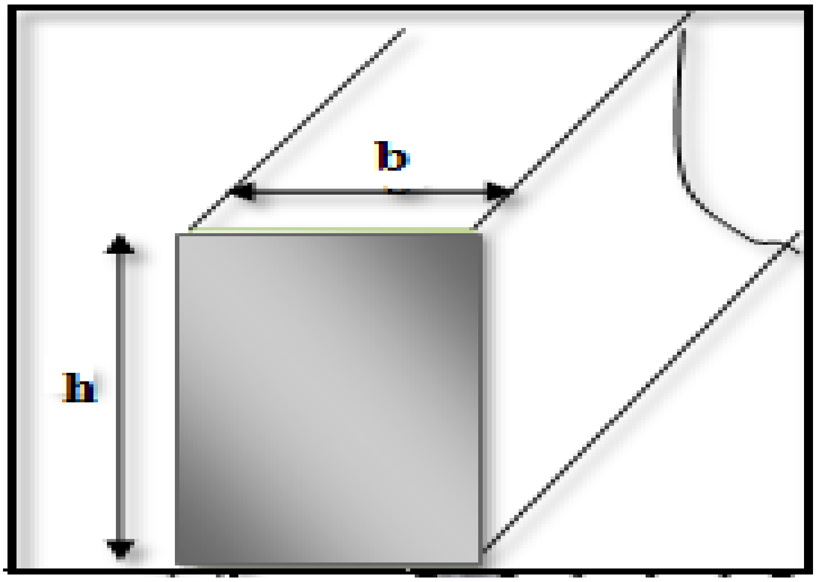
Checks in accordance with RPA99 version 2003 [article7.5.1 P64] :

$h=60cm \Longrightarrow h \geq 30$ cm……………………...checked

$b=30$ cm $\Longrightarrow b \geq 20$ cm ……………………..checked

$\frac{h}{b} = \frac{60}{30} = 2 \Longrightarrow \frac{h}{b} \leq 4$ ……………………... checked

We opt for beams of section (b, h) = (30, 60)



FigIV. 10: Section diagram of a Main Beam **[34]**.

IV.3.5.1.3.Secondary beams :

According to BAEL 91 we have : L =6.00 m

Height : $\frac{600}{15} \leq h \leq \frac{600}{10}$

40cm ≤ h ≤ 60 cm $\Longrightarrow$ we take h =50cm

Width: 0.3×50 ≤ b≤0.8×50 $\Longrightarrow$ 15cm ≤ b ≤ 40cm $\Longrightarrow$ we take b =30 cm

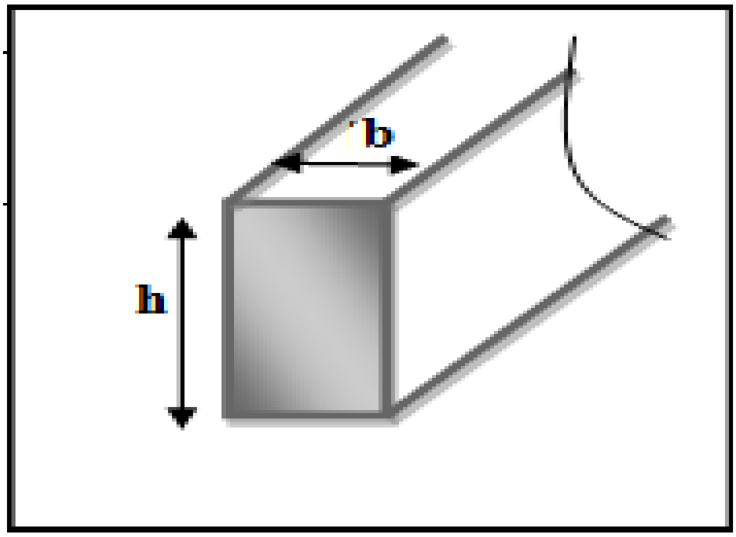
Checks in accordance with RPA99 version 2003 [article7.5.1 P64]:

h=60cm $\Longrightarrow$ h ≥ 30 cm……………………...checked

b =30 cm $\Longrightarrow$ b ≥ 20 cm ……………………..checked

$\frac{h}{b} = \frac{50}{30}$ =1.66 $\Longrightarrow \frac{h}{b} \leq 4$ ………………………checked

We opt for beams of section (b, h) = (30, 50).



FigIV. 11: diagram of the section of a Secondary Beam

IV.3.5.2.column :

IV.3.5.2.1.Pre-dimensioned column :

Columns are structural elements whose role is to carry loads Vertical and horizontal and transferred to the foundations.

According to the BAEL 91 rules, (article B.8.4.1), the ultimate normal force Nu acting

in the column

Must verify that:

$$Nu \leq \alpha \left[\frac{Br \times fc28}{0.9 \times \gamma b} + A \frac{fe}{\gamma s} \right]$$ **[36].**

As: The minimum steel section.

Fe: elastic limit of the steel used fe=400MPa

Br : The diminished cross-sectional area of the column is derived by subtracting one centimeter from its actual cross-sectional area.

Thick throughout its entire circumference, such as:

Br = (a-2) (b-2)…………cm2

F_{c28} : compressive strength of concrete F_{c28} = 25Mpa

γ_b =1.5

γs =1.15

The calculation is based primarily on the section of the most stressed column (central)

IV.3.5.2.2. Calculation method :

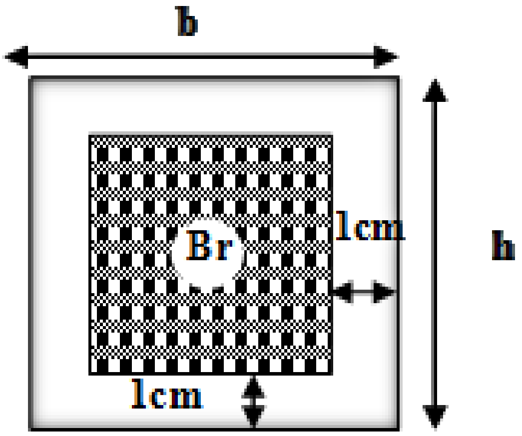
Their pre-sizing must respect the following three conditions:

Resistance condition, stability condition and condition imposed by RPA99

IV.3.5.2.3.Resistance condition:

According to BAEL91: $\beta r \geq \frac{k\beta Nu}{[\theta(\frac{\sigma bc}{0.9})+0.85(\frac{A}{\beta r})\sigma s]}$

Br: reduced section obtained by removing 2 cm of concrete thickness over the entire periphery
from the column :



FigIV. 12: Determination of the Br section **[34].**

Such as : $\begin{cases} \theta = 1 \\ \mathrm{K} = 1 \end{cases}$

$$\beta = \begin{cases} 1 + 0.2\left(\frac{\lambda}{35}\right)^2 & \lambda \leq 50 \\ \frac{0.8\lambda^2}{150} & 50 \leq \lambda \leq 70 \end{cases}$$

So that all the reinforcements participate in the resistance we will take (λ = 35)

→ β=1,2

According to RPA99 V2003 the minimum percentage of reinforcement is 0.7%

$\frac{\mathrm{A}}{\mathrm{B_r}} = 0.7\% = 0.007$

$$\begin{cases} \sigma_{bc} = \frac{0.85.f_{c28}}{\theta.\gamma_b} = \frac{0.85\times 25}{1\times 1.5} = 14.2MPA \\ \sigma_s = \frac{f_e}{\gamma_s} = \frac{400}{1.15} = 348MPA \end{cases}$$

Nu=1.35Ng+1.5Nq

The calculation of N_u from the load descent.

IV.3.5.2.4.Stability condition:

To avoid buckling it must be $\lambda \leq 35$

$\lambda = \frac{Lf}{i}$

$i = b/\sqrt{12} \rightarrow \lambda = \sqrt{12} \times \frac{Lf}{b}$ $\qquad L_f = 0.7L_0$

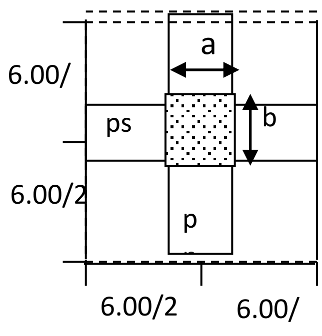
=> $\lambda = 2.618 \times 2{,}14 / 0{,}30 = 18.67 < 35$………… checked

condition imposed by RPA99 (version 2003) [article 7.4.1 P60]:

$$\begin{cases} \min(h_1, b_1) \geq 25cm \\ \min(h_1, b_1) \geq \frac{h_e}{20} \\ \frac{1}{4} \leq \frac{b_1}{h_1} \leq 4 \end{cases} \qquad \begin{cases} \min(30,40) \geq 25cm \ldots\ldots\ldots\ldots\ldots\ldots\text{checked} \\ \min(30,40) \geq \frac{374}{20} = 18.7cm \ldots\ldots \text{ checked} \\ \frac{1}{4} \leq \frac{30}{40} = 0.75 \leq 4 \ldots\ldots\ldots\ldots\ldots\ldots\text{checked} \end{cases}$$

[37].

The most stressed column is the central column :



FigIV. 13: Central **c**

IV.3.5.3.Calculation of column loads and overloads

Table IV. 1: Loads and overloads on columns

Levels	Element	G(KN)
1-1	Terrace floor Main beam Secondary beam	36×6.5=234 (0.30×0.60)×6×25=27 (0.30×0.50)×6×25=22.5
Total		G=283.5 KN
ground floor	ground floor Main beam Secondary beam Column	18×5.43=97.74 (0.30×0.60)×6×25=27 (0.30×0.50)×3×25=11.25 (0.30×0.40)×3.74×25=11.22 G=147.24KN
Total		**G=430.74 KN**

IV.3.5.3.1.Reduction loads :

Q_{Terrace} =1×36=36 KN

Q_1=36+3.5×18=99KN

Q_{total}=99KN

N_U=1,35×430.74 +1,5×99= 729.99 KN

$$\beta r \geq \frac{1.2 \times 729.29 \ \times 10^3}{\left[\left(\frac{14.2}{0.9}\right) + 0.85\left(\frac{0.7}{100}\right)348\right]} 10^{-4} = 4.907\ cm^2$$

Br ≥ 4.907cm^2

Br = (a × b)= 4.907 cm^2

Br = $(a-2)^2 \rightarrow$ a =$\sqrt{\text{Br}} + 2$

IV.3.5.3.2.Vérification spécifique:

$$v = \frac{N_U}{B.F_{C28}} \leq 0.3$$

V=708.81 /(0.30×0.40×25) ≤ 0.3…………………checked

Table IV. 2: Pre-dimensioning of columns.

Levels	G(KN)	$G_{cumulé}$	Q(KN)	$Q_{cumulé}$	Nu	Br (cm2)	a (cm)	a × b	V ≤ 0.3
Terrace	283.5	283.5	36	36	418.743	2.81	3.67	/	/
N1	147.24	430.74	63	99	290.07	1.95	3.39	30*40	

IV.3.5.4.Conclusion :

The pre-dimensioning of the primary and secondary components furnished us with the subsequent data on the diverse loads destined for specific segments of the framework. Upon completion of the pre-dimensioning process for the structural components and conducting all essential verifications, we proceeded to select the subsequent sections for said components.

Main beams: b × h= (30×60) cm^2.

Secondary beams: b × h= (30×50)cm^2 .

Columns:(30×40) cm^2.

Columns:(40×50) cm^2.

IV.4.CALCUL DES ELEMENTS SECONDAIRES :

IV.4.1.The parapet :

IV.4.2.Introduction:

Our publication exclusively contains a single form of acrotery, which serves as a safety feature at the terrace level. This particular structure acts as a barrier to prevent falls, functioning as a console integrated at its lower part, bearing its own weight and horizontal loads. The analysis involves utilizing composite bending in the embedding segment over a 1m continuous section. Given that the acrotery is exposed to outdoor conditions, the occurrence of cracks is undesirable. Therefore, the assessment will be conducted at the ultimate limit state and the serviceability limit state in composite flexure for a linear 1m strip.

IV.4.3.Combination of stresses :

ULS

Normal force : N_U=1.35×G=1.35×1.72=2.322KN

Moment of embedding : Mu=1.5×Q×h=1.5×1×0.6=0.9KN

Shear force : Tu=1.5×1=1.5 KN

SLS

Normal force : N_U=G=1.72KN

Moment of embedding : Mu=Q×h=1×0.6=0.6KN

Shear force : Tu= Q =1KN

IV.4.4.Reinforcement :

we are working on a rectangular section

h =10 cm b =100 cm

d = 8 cm d'= 2 cm

IV.4.4.1.Calculation of reinforcement at U.L.S:

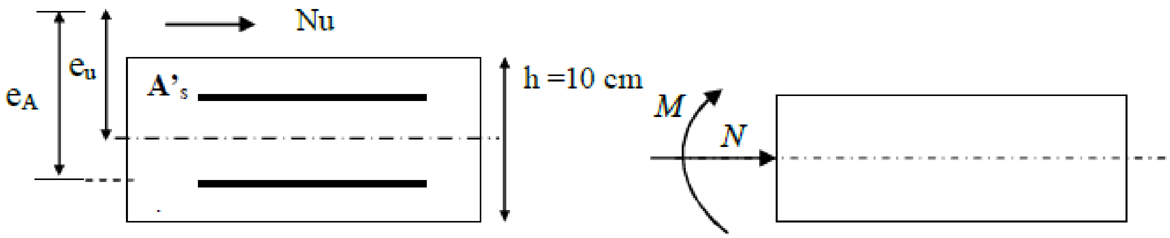
a. culation of eccentricity :

e_u= Mu/Nu=0.9/2.322=0.38m

h/2- d'=5-2=3cm => e_u=38cm >h/2- d'=3cm

b. Simple bending calculation:

e_A = e0 + (h/2 – d')=0.38 + (0.1/2–0.02) =0.41m
M_f= Nu× e_A=2.322×0.41 =0.952KN.m



FigIV. 14: Simple bending **[1].**

IV.4.4.2.Calculation of reduced moment:

$$u_{bu} = \frac{M_F}{F_{bu} \times d^2 \times b_0}$$

With : $f_{bu} = \frac{0.85 \times f_{c28}}{\theta \times \gamma_b} = 14.20MPA$

$$u_{bu} = \frac{0.952 \times 10^{-3}}{14.20 \times (0.08)^2 \times 1} = 0.010$$

IV.4.4.3.Calculation of reduced limit moments:

$$u_{lu} = (3440\gamma + 49 \times f_{c28} - 3050) \times 10^{-4}$$

$$\gamma = \frac{M_f}{M_s} = \frac{0.952}{0.6} = 1.85$$

$$u_{lu} = (3440 \times 1.85 + 49 \times f_{c28} - 3050) \times 10^{-4} = 0.36$$

$u_{bu} < u_{lu}$ **A′** = 0 no compressed reinforcement

$$z_b = d(1 - 0.4 \times u_{bu})$$

$$z_b = 0.08(1 - 0.4 \times 0.010) = 0.079m$$

$$A = \frac{M_f}{(z_b \times f_{ed})}$$

$$f_{ed} = \frac{f_e}{\gamma_s} = \frac{400}{1.5} = 348MPA$$

$$A = \frac{0.952 \times 10}{0.079 \times 348} = 0.34cm^2$$

IV.4.4.4.Calculation of the cross-section of real reinforcement in compound bending

$$A = A_{fs} - \frac{N_u}{f_{ed}} = 0.34 - \frac{2.233 \times 10}{348} = 0.27cm^2$$

$$A = 0.27cm^2$$

IV.4.4.5.Verifications:

Condition of non-fragility: (Art. A.4.2.1/BAEL91 modified 99)

The reinforcement of the parapet must satisfy the condition of non-fragility:

$A_{min} = 0.23b.d.\frac{f_{t28}}{f_e}$; $f_{t28} = 2.1MPA$ **[37].**

$A_{min} = 0.23(1 \times 0.08) \times \frac{2.1}{400} = 0.966cm^2$

We note that : $A_u < A_{min}$

So the reinforcement will be made with A_{min}

We take : As= 4HA8 with spacing St=100/4=25cm

IV.4.4.5.1. Verifications at the U.L.S:

a. Verification of shear force

Check that $\tau_{u\,max} < \tau_{\bar{u}}$

$\tau_u = \frac{T_u}{b.d} = \frac{1.5\times10^{-3}}{1\times0.08} = 0.019MPA$ with T_U =1.5×Q=1.5MPA

$\tau_{\bar{u}} = \min\left(\frac{0.15\times f_{c28}}{\gamma_b}, 4MPA\right)$

$\tau_{\bar{u}} = \min\left(\frac{0.15\times 25}{\gamma_b}, 4MPA\right) = 2.5MPA$

$\tau_{u\,max} < \tau_{\bar{u}}$ → Checked condition

a. Checking the adhesion of bars in shear

The adhesion stress must be less than the ultimate limit value.

$\tau_{se} < \tau_{\bar{s}} = \Psi_s . f_{t28}$

Ψ_s **:** Sealing coefficient

$\tau_{se} = \frac{T_u}{0.9d\sum u_i}$

$\Psi_s = 1.5(Fe400\ steel, high\ adhesion)$

τ_{se}: Adhesion stress

$\tau_{\bar{s}}$: Adhesion stress admissible

$\sum u_i = n\pi\emptyset$: Sum of the useful perimeter of the rods

n : number of rods

$\emptyset$: Diameter of rods $(\emptyset = 8mm)$

$\boldsymbol{\tau se}$ = 0.21MPa < 3.15 MPa → Checked condition

There is no risk of dragging the rods

IV.4.4.5.2.Calculation of reinforcement at S.L.S:

$N_{ser} = 1.72KN$

$M_{ser} = 0.6KN.m$

$M_{br} = \frac{1}{2}\alpha_1\left(1-\frac{\alpha_1}{3}\right) \times b_0 \times d^2 \times \sigma_{\overline{bc}}$

$\alpha_1 = \frac{15\sigma_{\overline{bc}}}{\sigma_s+15\sigma_{\overline{bc}}}$

$\sigma_{\overline{bc}} = 0.6 \times f_{c28} = 0.6 \times 25 = 15MPA$

$\sigma_{\bar{S}} = min\left(\frac{2}{3}f_e; 110\sqrt{\eta.f_{td}}\right)$

$\sigma_{\bar{S}} = min\left(\frac{2}{3} \times 400; 110\sqrt{1.6 \times 2.1}\right) = 201.63MPA$

$\alpha_1 = \frac{15\sigma_{\overline{bc}}}{\sigma_s+15\sigma_{\overline{bc}}}$ = 0.53

$M_{br} = \frac{1}{2} \times 0.53\left(1-\frac{0.53}{3}\right) \times 1 \times 0.08^2 \times 15 \times 10^3$

$M_{br} = 20.94KN.m$

$M_{br} > M_{ser} A' = 0$

$u_s = \frac{M_{ser}}{b_0.d^2\sigma_{\bar{S}}} = \frac{0.6\times10^{-3}}{1\times0.08^2\times201.63} = 0.00047$

$Z_{ser} = \frac{15}{16}d\frac{40u_s+1}{45u_s+1} = 7.45cm$

$A_{ser} = \frac{M_{ser}}{Z_b \times \sigma_{\bar{S}}} = \frac{0.6\times10^{-3}}{7.45\times201.63} = 0.40cm^2$

a. Conclusion of reinforcement

$A_s = max(A_{ser}, A_u, A_{min}) = (0.4; 0.27; 0.966) = 0.966cm^2$

We take 4HA8 (2.01cm^2) with 25cm spacing

b.Reinforcement distribution

$A_r = \frac{A_s}{4} = \frac{2.01}{4} = 0.5cm^2$

We take 3HA6(0.84cm^2) with 15cm spacin

IV.4.4.5.3.Verifications at the S.L.S:

Stress in concrete : $\sigma_{bc} \leq \sigma_{\overline{bc}}$

Stress in steel : $\sigma_s \leq \sigma_{\bar{S}}$

$$e_{sre} = \frac{M_{ser}}{N_{ser}} = \frac{0.6}{1.72} = 0.34m \geq 0.03m$$

$$\sigma_{bc} \leq \sigma_{\overline{bc}} = 0.6 \times f_{c28} = 15MPA$$

$\sigma_{bc} = Ky$ with K = $\frac{M_{ser}}{I}$

We have :

$\frac{b \times y^2}{2} + \eta(A + A') \times y - \eta(A.d + A'.d') = 0$ with $A' = 0$ $\eta = 15$

$50y^2 + (15 \times 2.01 \times y) - (15 \times 8 \times 2.01) = 0 \Rightarrow y = 1.92cm$

$$I = \frac{by^3}{3} + \eta A_s(d - y)^2 + \eta A'_s(y - d')^2$$

$$I = \frac{100 \times 1.93^3}{3} + 15 \times 2.01(8 - 1.93)^2 = 1350.47cm^4$$

K = $\frac{M_{ser}}{I} = \frac{0.6}{1350.47 \times 10^8} = 0.0444$

$\sigma_{bc} = K.y = 0.0444 \times 19.2 = 0.85MPA \leq \sigma_{\overline{bc}} = 15MPA$ ... Checked condition

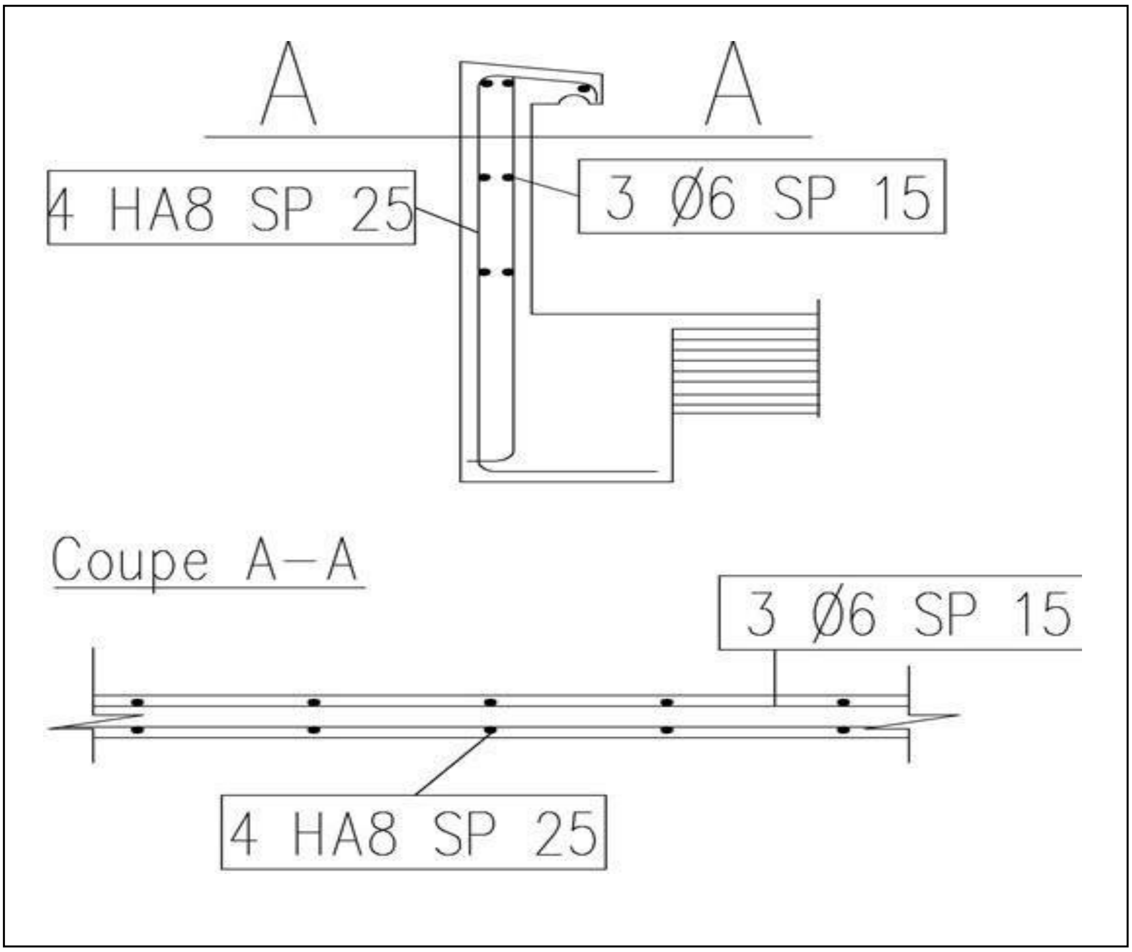
a.Verification of maximum stresses in steel

We need to check that : $\sigma_s \leq \sigma_{\bar{S}}$

$$\sigma_{\bar{S}} = min\left(\frac{2}{3}f_e; 110\sqrt{\eta.f_{td}}\right) = 201.63MPA$$

$$\sigma_S = \eta \times k \times (d - y) = 15 \times 0.044(80 - 19.2) = 40.49MPA$$

$\sigma_s = 40.49MPA \leq \sigma_{\bar{S}} = 201.63MPA$ ... ... Checked condition



FigIV. 15: Diagram of parapet reinforcement **[1].**

IV.5.Floor calculation :

IV.5.1Introduction :

Floors are flat horizontal elements that separate the different levels of a building.

The floors in a building are hollow floors (16+4), combined with beams.

The hollow-body floor consists of :

• Nervures called T-section beams, they provide load-bearing capacity.

distance between beam axes is 65cm.

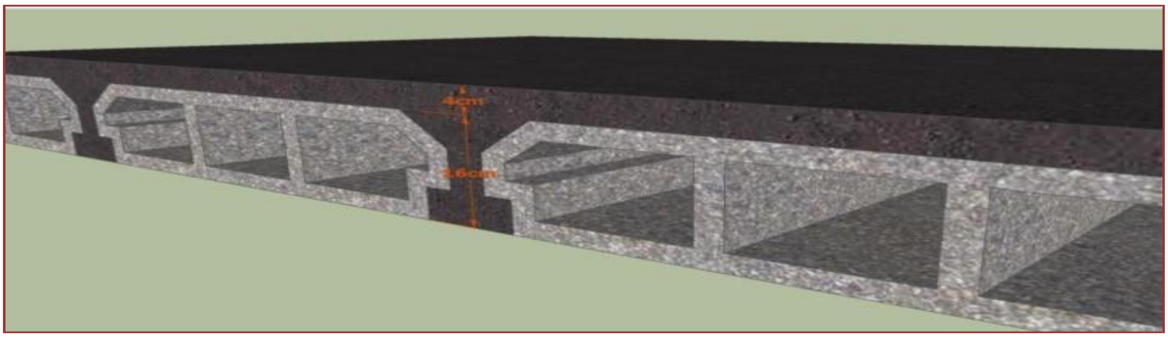
• Remplissage are used as lost formwork and soundproofing.

Its height is 16cm

4cm-thick concrete compression slab, reinforced with a grid of rebars to reinforcement grid:

- Limit the risk of cracking due to shrinkage.

- Withstand loads applied to small surfaces.



FigIV. 16: Hollow body floor **[1].**

IV.5.2. beam study:

If the beams in both directions are equal, then the direction in which there are more supports (continuity criterion) is chosen. as the supports relieve the moments in the span and reduce the deflection.

We have: b: total compression width

h: floor thickness.

Lx: maximum distance between two beams.

Ly: maximum distance between beams perpendicular to Lx.

L=300- 30=270 cm

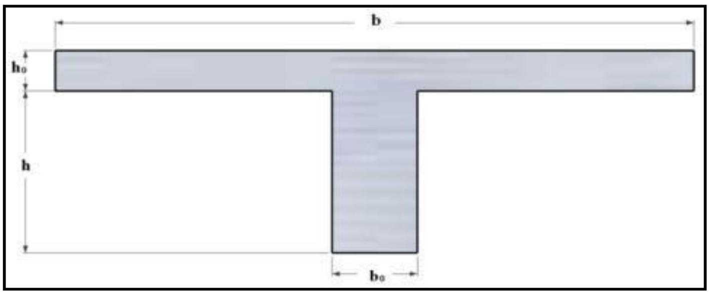
So : 0.4h≤ b_0 ≤0.6h ⇨ 8cm≤ b_0 ≤12cm

Takes: b0=12cm

We will have : b= $\frac{b-b_0}{2} \leq min\left\{\frac{Ix}{2},\frac{Iy}{10}\right\}$ ⇨ b= $2min\left\{\frac{Ix}{2},\frac{Iy}{10}\right\} + b_0$

Ix= 65-12=53cm , Iy=300cm ⇨ b≤ $2b_1 + b_0 = 2(26.5)$ +12=65cm

We take: b=65cm



FigIV. 17:T-beam **[1].**

IV.5.3.Evaluation of loads:

IV.5.3.1.Terrace floor :

G=6.5 KN/m^2 Q=1KN/m^2

IV.5.3.2.Current floor :

G=5.43KN/m Q=1.5KN/m^2

We calculate the most unfavorable floor and generalize the reinforcement for the other floors of the different levels:

The most unfavourable case is the terrace case: G=6.13KN/m^2 and Q=1 KN/m^2

IV.5.3.3.Choice of calculation method:

To calculate the internal forces in beams considered as continuous beams on several supports, we use one of the 02 simplified methods.

- The flat-rate method.

- Caquot's method.

Flat-rate method :

This method is applicable if the following four hypotheses are verified:

1- Q ≤max (2G ; 5 KN/m²)
2- T The length ratio between two successive spans must verify :

$$\begin{cases} 0.8 \leq \frac{l_n}{l_{n+1}} \leq 1.25 \\ 0.8 \leq \frac{l_n}{l_{n-1}} \leq 1.25 \end{cases}$$

3-Little harmful cracking.

If one of the conditions is verified, the applicable flat-rate method.

IV.5.3.4.Application:

IV.5.3.5.Terrace floor:

Q = 1 KN/m^2< 2G = 12.26 KN/m^2 …………………….. checked.

IV.5.3.6.Floor Current floor:

Q=1.5 KN/m^2< 2G =10.86KN/m^2 ………………………. Checked.

2) The moments of inertia of the cross-sections are the same in the different spans.

3) Cracking is not very detrimental
he moments of inertia of the cross-sections are the same in the different spans.

$$\begin{cases} 0.8 \leq \frac{l_n}{l_{n+1}} \leq 1.25 & \Rightarrow 0.8 \leq 1.22 \leq 1.25 \\ 0.8 \leq \frac{l_n}{l_{n-1}} \leq 1.25 & \Rightarrow 0.8 \leq 0.81 \leq 1.25 \end{cases}$$ …………………. Checked.

IV.5.4. Method forfaitaire :

IV.5.4.1.Application of the method :

α = Q / G + Q.

$M_0 = \frac{q.L^2}{8}$.

$M_t \geq \max\{1.05.\ M_0\ ;\ (1+0.3\ \alpha).M_0 - (M_w + M_e)\ /.$

$M_t \geq (1+0.3\ \alpha).M_0 / 2$.

$M_t \geq (1.2+0.3\ \alpha).M_0 / 2$.

The moment on chord supports of number of spans.

IV.5.4.2. Calculation of M0 :

E.L.U :

$$M_{0u} = \frac{q_u L^2}{8}$$

E.L.S :

$$M_{0s} = \frac{q_s L^2}{8}$$

IV.5.4.3. Calculation of the isostatic shear forc :

E.L.U :

$$T_{0u} = \frac{q_u L}{2}$$

E.L.S :

$$T_{0s} = \frac{q_s L}{2}$$

IV.5.4.4. The shear force:

$V_1 = T_0 + (M_e - M_w) / 1.$

$V_2 = - T_0 + (M_e - M_w) / 1.$

IV.5.4.5.Terrace floor :

- Permanent loads : G =6.5KN /m^2.
- Operating load Q = 1 KN /m^2.

a. Loads over 0.65 m (distances between beams):

- Q = 1 × 0.65 = 0.65 KN /ml.

- G = 6.5× 0.65 = 4.225KN /ml.

IV.5.4.6.Static calculation :

E.L.U : Pu = 1.35 × 4.225+ 1.5 × 0.65 = 6.678 KN / ml.

E.L.S : Pu = 4.225 + 0.65 = 4.875 KN / ml

IV.5.4.7.Floor 1st type :

- ELU : $=\frac{Q}{G+Q} = \frac{0.65}{4.225+0.65} \Longrightarrow \alpha = 0.133$

1+0,3α=1+0,3×0,133 = 1.04

Max (1.05 ;1+0.3α)=1.05

<u>IV.5.5.Calculation of span and support moments at ULS.</u>

IV.5.5.1.Isostatic moment :

a. Bay:

$M_0 = \frac{q_u L^2}{8} = \frac{6.678 \times 3^2}{8} = 7.512$KN.m

b. Support moments :

Support A : M_a= 0.20 × M_0= 0.20 × 7.512= 1.502

Support B : M_b = 0.6 × M_0 = 0.6 × 7.512 = 4.507

The moments of the span :

$M_t \geq (1.2+0.3\ \alpha).M_0/2$

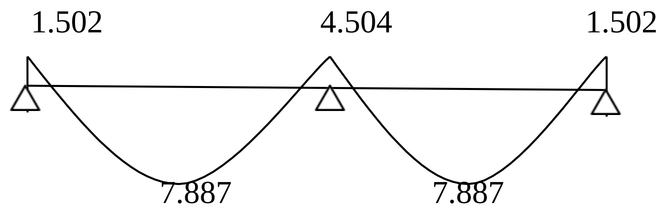
$M_t \geq (1.2+ (0.3 \times 0.14)).7.512/2=4.664$

$M_t \geq \max 1.05.\ M_0\ ;\ (1 +0.3\ \alpha).M_0 - (M_w + M_e) / 2.$

$M_t \geq$ max 7.887; 6.851= 7.887

Table IV. 3: The values of the moments in the span and on supports at the ULS

Span	01	
L (m)	3.00	
M_{0u}(N.m)	7.512	
Support	01	02
Coef. Flat rate	0.20	0.6
M_A (KN.m)	1.502	4.507
M_t (KN.m) (c.1)	7.887	
M_t (T.de rive)	4.664	
M_t (resultant)	7.887	



FigIV. 18: Diagram of moments in span and on supports at the ULS **(no need).**

IV.5.5.2.Calculation of the shear force:

E.L.U :

$T_{ou} = P_u.\, l/2 = 10.017$KN.

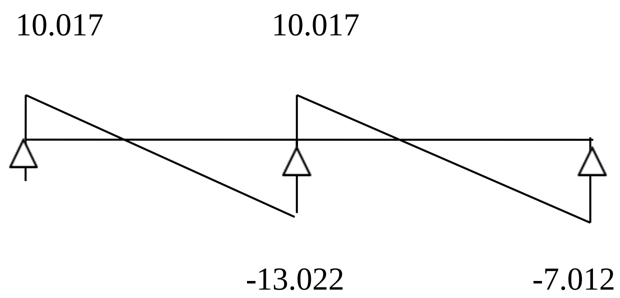
E.L.S :

$T_{ou} = P_s.\, l/2 = 10.017$KN. $V_1 = T_0 + (M_e - M_w) / 1.$

$V_2 = -\, T_0 + (M_e - M_w)$

Table IV. 4: The values of the shear force on supports at the ULS

Section	**M_e(KN.m)**	**M_w(KN.m)**	**T_0(KN)**	**L(m)**	**V_1(KN)**	**V_2 (KN)**
1-2	1.502	4.507	10.017	3.00	7.012	-13.022
2-3	4.507	1.502	10.017	3.00	13.022	-7.012



FigIV. 19:Diagram of A'ELU shear forces **(no need).**

<u>IV.5.6.Calculate the moments in the span and on the ELS supports:</u>

IV.5.6.1. Isostatic moment:

a. Bay01:

$M_0 = \frac{qL^2}{8} = \frac{4.875\times 3^2}{8} = 5.484$ KN.m

b. The moments of support:

Support A: $M_a = 0.20 \times M_0 = 0.20\times 5.484 = 1.096$

Support B: $M_b = 0.6 \times M_0 = 0.6 \times 5.484 = 3.29$

c. The moments of the span:

- Edge span AB:

$M_t \geq (1.2+0.3\ \alpha).M_0 /2$

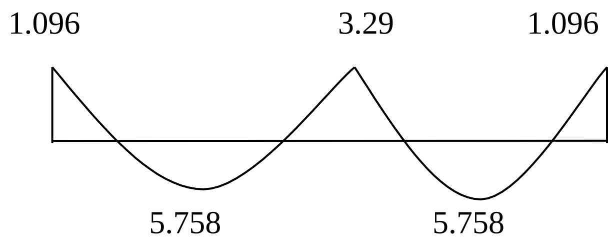
$M_t \geq 4.145$

$M_t \geq \max 1.05.\ M_0\ ;\ (1 +0.3\ \alpha).M_0 – (M_w + M_e) / 2.$

$M_t \geq \max 5.758;5.002= 5.758$

Table IV. 5: The values of the moments in the spanand on supports at the SLS

Span	**1**	
L (m)	3.00	
$M_{0u} = P_s.L^2 / 8$(KN.m)	5.484	
Support	**1**	**2**
Coef. Flat rate	0.20	0.6
M_A (KN.m)	1.096	3.29
M_t (KN.m)	5.758	
M_t (T.de rive)	4.145	
M_t (resultant)	5.758	



FigIV. 20:Diagram of span and support moments at SLS **(no need).**

IV.5.6.2.Shear force calculation :

U.L.S :

$T_{ou} = P_u.\ \bar{l} /2 = 7.312$KN

S.L.S :

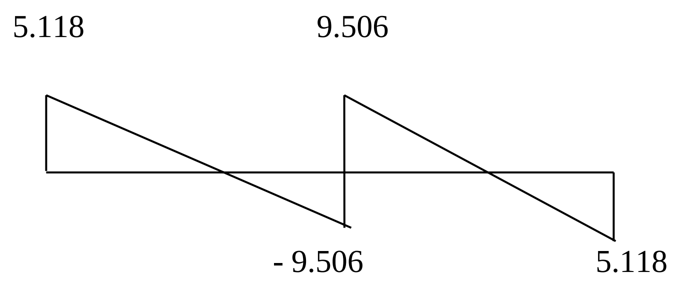
$T_{os} = P_s.\ \bar{l} / 2 = 7.312$ KN.

$V_1 = T_0 + (M_e - M_w) / 1.$

$V_2 = - T_0 + (M_e - M_w) / 1.$

Table IV. 5: The values of the shear force on supports at the SLS.

Section	M_e(KN.m)	M_w(KN.m)	T_0(KN)	L (m)	V_1(KN)	V_2 (KN)
1-2	1.096	3.29	7.312	3.00	5.118	-9.506
2-3	3.29	1.096	7.312	3.00	9.506	- 5.118



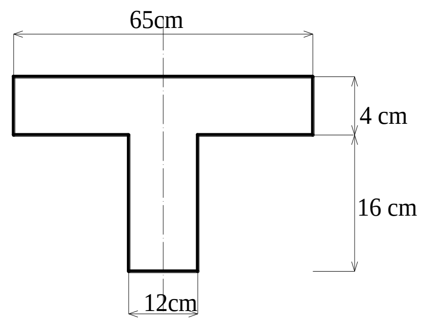
FigIV. 21: Diagram of A'S LS shear forces **(no need).**

IV.5.6.3.Calculation of longitudinal reinforcement :

Table IV. 5: Summary of results.

M_{au}(max) (KN.m)	M_{tu} (max) (KN.m)	V (max) (KN)
4.507	7.887	-13.022

b = 0.65 m ; b0 = 0.12 m ; h0 = 0.04 m ; h = 0.2 m ; d = 0.9h = 0.18 m



FigIV. 22:Truss formwork **[1].**

According to the simple bending flow chart, we find the following results:

IV.5.6.4.Reference time :

a. In span:

Reinforcement is calculated as for a T-section beam. The moment is balanced by the compression table:

M = b . h . $\sigma_{bc}\left(d\,\frac{h_0}{2}\right)$= 0.65 × 0.04 ×14.20 (0.18 ×$\frac{0.04}{2}$) 10^3 = 59.07 KN.m

M< Mt ⇨ L the neutral axis falls into the table, only one part of the table is compressed, and as tensioned concrete is not included in the calculations, the T-section will be calculated

as a rectangular section of dimensions (b*h), b =65cm and h=20cm

$\mu_{bu} = \frac{M_u}{b \times d^2 \times F_{bu}} = \frac{7.887 \times 10^{-3}}{0.65 \times (0.18)^2 \times 14.20} = 0.026$

$\alpha = 1.25 \times (1 - \sqrt{1 - 2\mu_{bu}}\ ) = 1.25 \times (1 - \sqrt{1 - 2 \times 0.026}\ ) = 0.0329$

$Z_b = d \times (1 - 0.4\alpha) = 0.13 \times (1 - 0.4 \times 0.0329) = 0.128m$

$A = \frac{M_u}{Z_b \times F_{ed}} = \frac{7.887 \times 10}{0.128 \times 348} = 1.77\ cm^2$

Or: 2HA12/ml =2.26cm^2 Au=2.26cm^2

• Condition of non-fragility: (Art. A.4.2.1/BAEL91modified 99)

$A_{min} = 0.23 \times b \times d\ \frac{F_{t28}}{Fe}$ $F_{t28} = 2.1$ Mpa

$A_{min} = 0.23(65 \times 18) \times \frac{2.1}{400} = 1.41\ cm^2$

We note that $A_{min} < A_u = 2.26\ cm^2$

b. On supports:

$\mu_{bu} = \frac{M_u}{b \times d^2 \times F_{bu}} = \frac{4.507 \times 10^{-3}}{0.65 \times (0.18)^2 \times 14.20} = 0.0150$

$\alpha = 1.25 \times (1 - \sqrt{1 - 2\mu_{bu}}\ ) = 1.25 \times (1 - \sqrt{1 - 2 \times 0.0150}) = 0.0188$

$Z_b = d \times (1 - 0.4\alpha) = 0.13 \times (1 - 0.4 \times 0.0188) = 0.129m$

$A = \frac{M_u}{Z_b \times F_{ed}} = \frac{4.504 \times 10}{0.129 \times 348} = 1\ cm^2$

Or: 1HA12/ml =1.13cm^2 Au=1.13cm^2

• Condition of non-fragility: (Art. A.4.2.1/BAEL91modified 99)

$A_{min} = 0.23 \times b \times d\ \frac{F_{t28}}{Fe}$ $F_{t28} = 2.1$ Mpa

$A_{min} = 0.23(20 \times 18) \times \frac{2.1}{400} = 0.434\ cm^2$

We note that $A_{min} < A_u = 1.13\ cm^2$

IV.5.6.5. The checks:

IV.5.6.5.1.Verification at the U.L.S:

a. In span :

- **Verification of the shear force (Art III.2/BAEL 91)**

T =13.022 KN

It is necessary to verify that $\tau_\mu < \bar{\tau_\mu}$

$\bar{\tau_\mu}$= min$\left\{0.15\frac{F_{c28}}{\gamma b}\ ;\ 5MPa\right\}$

$\bar{\tau_\mu}$= min $\{2.5Mpa\ ;\ 5\text{MPa}\}$= 2.5 MPa

$\tau_\mu = \frac{V_U}{b\ .d} = \frac{13.022\times10^3}{650\times180}$= 0.111 MPa

τ_μ = 0.111 MPa < $\bar{\tau_\mu}$= 2.5 MPa ………………… Checked.

- Adhesion check
- $\tau_{ser} < \bar{\tau_{se}}$

$\tau_u = \frac{\tau_u}{0.9d\varepsilon ui} \leq \bar{\tau_{se}} = \bar{\psi_s}\ .\ F_{t28}$ = 3.15MPa

- $\psi_s = 1.5(steel\ Fe400, high\ adhesion)$
- $\Sigma\text{ui} = \text{n}\pi\emptyset = 2\times3.14\times12 = 75.36\text{mm}$
- n: number of bars
- ØBar diameter (Ø=12mm)
- $\tau_{ser} = \frac{13.022\times10^3}{0.9\times180\times75.36}$ = 1.066MPa
- $\emptyset_{ser} = 1.066MPa < \bar{\tau_{se}}\ \emptyset = 3.15MPa$

Bar diameter (=12mm)

b. On Support: T=13.022KN

- It is necessary to verify that $\tau_{ser} < \bar{\tau_{se}}$

$\bar{\tau_\mu}$= min$\left\{0.15\frac{F_{c28}}{\gamma b}\ ;\ 5MPa\right\}$

$\bar{\tau_\mu}$= min $\{2.5Mpa\ ;\ 5\text{MPa}\}$= 2.5 MPa

$\tau_\mu = \frac{V_U}{b\ .d} = \frac{13.022\times10^3}{120\times180}$= 0.602MPa

τ_μ = 0.602MPa < $\bar{\tau_\mu}$= 2.5 MPa …………………. Checked

IV.5.6.5.2.Adhesion check :

- $\tau_{ser} < \bar{\tau_{se}}$

$\tau_u = \frac{\tau_u}{0.9d\Sigma ui} \leq \bar{\tau_{se}} = \bar{\psi_s} . F_{t28}$ = 3.15MPa

- $\psi_s = 1.5(steel\ Fe400, high\ adhesion)$
- $\Sigma ui = n\pi\emptyset = 1 \times 3.14 \times 12 = 37.68mm$
- n: number of bars
- ØBar diameter (Ø=12mm)
- $\tau_{ser} = \frac{13.022 \times 10^3}{0.9 \times 180 \times 37.68}$ = 2.133MPa
- $\emptyset_{ser} = 2.133MPa < \bar{\tau_{se}}\ \emptyset = 3.15MPa$

Bar diameter (Ø=12mm)

IV.5.6.6.Calculation of transverse reinforcement and spacing :

IV.5.6.6.1.Calculation of transverse reinforcement :

The transverse reinforcement is calculated according to the following regulations:

According to RPA99V2003 (Article.7.5.2.2)

$$\begin{cases} \frac{A_t}{S_t} \geq 0.003b_0 \\ S_t \leq (\frac{h}{4}, 12\emptyset_l) \ldots\ldots \emptyset_t \leq min(\frac{h}{35}; \frac{b_0}{10}; \emptyset_1) \\ S_t \leq \frac{h}{2} \ldots \ldots \end{cases}$$ **[37].**

$\emptyset_l$: Minimum diameter of longitudinal reinforcements.

$\emptyset_t \leq min[5.14; 12; 12]$ =5.14mm

We will take Ø =6 mm; the transverse reinforcements are: 2 Ø 6 (A = 0.57cm2).

IV.5.6.6.2. Calculation of spacing (St):

Current zone; St ≤min (0.9d. 40cm)

Nodal zone; St=St (Current zone) / 2

SO :

$S_t \leq$ min (0,9 ;40)= min (16,2 ;40)

$S_t \leq 16.2$cm

We take: St = 15 cm (except for the first plane of the transverse reinforcements which will be placed at).

$\frac{S_t}{2}$=7.5mm

IV.5.6.6.3. Verification at S.L.S:

IV.5.6.7.the compressive stress in concrete:

Since cracking is not very damaging, it should be checked.

a. In span:

M_{ser}=5.758KNm ; b =65 cm ; d= 18 cm ; A = 2.26cm^2

$\sigma_{bc} \leq \bar{\sigma_{bc}}$=0.6 × F_{c28} =15MPa

σ_{bc}= ky   k = $\frac{M_{SER}}{I}$

Neutral axis position:

(We have)$(A' = 0 \quad , \eta = 15)$

$$y = n\,\frac{A_s+A_s'}{b}\left(\sqrt{1+\frac{b.d.A_s}{7.5(A_s+A_s')^2}}-1\right)$$

$$y = 15\frac{2.26}{100}\left(\sqrt{1+\frac{18\times65}{7.5\times2.26}}-1\right)$$

y = 2.5 cm

$$I = \frac{by^3}{3} + nA_s(d-y)^2 + nA_s'(y-d)^2$$

$$I =\frac{65}{3}(2.5)^3 + 15 \times 2.26\,(18-2.5)^2 = 8483.01\ cm^4$$

$$K = \frac{M_{ser}}{I} = \frac{5.758\times10^6}{8483.01\times10^4} = 0.0678\ N/m^3$$

σ_{bc} = Ky= 0.0678× 25= 1.695 MPa≤ $\bar{\sigma_{bc}}$= 15MPa…….. Checked

IV.5.6.7.1.Verification of the maximum stress of the steel:

$\sigma_s \leq \bar{\sigma_s}$

$\sigma_s = \eta . k(d-y) = 15 \times 0.0678(180-25) = 157.635$MPa

$\bar{\sigma_s} = \frac{F_e}{\gamma s} = 348$ MPa

$\sigma_s = 157.635$MPa$<$ $\bar{\sigma_s} = 348$MPa …….. Checked

a. Support:

M_{ser}=3.29KN.m; b =12 cm; d= 18 cm ; A = 1.13 cm^2

$\sigma_{bc} \leq \bar{\sigma_{bc}} = 0.6 \times F_{c28}$ =15MPa

σ_{bc}= ky k = $\frac{M_{SER}}{I}$

Position de l'axe neutre :

(We have)($A' = 0$, $\eta = 15$)

y = n $\frac{A_s + A_s'}{b}$($\sqrt{1 + \frac{b.d.A_s}{7.5(A_s + A_s')^2}} - 1$)

y = 15$\frac{1.13}{12}$ ($\sqrt{1 + \frac{18 \times 12}{7.5 \times 1.13}} - 1$)

y = 5.85 cm

I = $\frac{by^3}{3}$ + nA_s(d-y)^2 +n$A_s'(y-d)^2$

I =$\frac{12}{3}$ (5.85)^3+ 15 ×1.13 (18-5.85)^2 = 3303.0078 cm^4

K = $\frac{M_{ser}}{I}$ = $\frac{3.29 \times 10^6}{3303.0078 \times 10^4}$= 0.0996 N/$m^3$

σ_{bc} = Ky= 0.0996 × 58.5= 5.826MPa$\leq$ $\bar{\sigma_{bc}}$= 15MPa………………. Checked

IV.5.6.7.2.Verification of the maximum stress of the steel :

$\sigma_s \leq \bar{\sigma_s}$

$\sigma_s = \eta . k(d-y) = 15 \times 0.0996(180-58.5) = 181.52$MPa

$\bar{\sigma_s} = \frac{F_e}{\gamma s} = 348$ MPa

$\sigma_s = 181.52$MPa$<$ $\bar{\sigma_s} = 348$MPa …………. Checked

• Checking the arrow

1) $\frac{h}{l} > \frac{1}{16}$ 2) $\frac{l}{h} > \frac{1M_t}{10M_0}$ 3)$\frac{A}{b.d} \leq \frac{4.2}{fe}$MPa

l: The span between bare supports (4.05m) h: height of the section (15cm)

Mt: maximum moment in span

M0: moment of the reference span

A: tensioned steel section in span

$$\frac{h}{l} = \frac{20}{330} = 0.0606 < \frac{1}{16} = 0.0625$$

$$E_i = 11000 . \sqrt[3]{F_{c28}} = 11000 . \sqrt[3]{25} = 32164,19 \text{ MPa}$$

$$E_i = 3700 . \sqrt[3]{F_{c28}} = 3700 . \sqrt[3]{25} = 10818,86 \text{ MPa}$$

$$\rho = \frac{A}{b_0 . d} = \frac{2.26}{12 \times 18} = 0.0104$$

$$\mu_g = 1 - [\frac{1.75 \times F_{t28}}{4 \times \rho . \sigma_{sg} + F_{t28}}] = 1 - [\frac{1.75 \times 2.1}{4 \times 0.0104 \times 348 + 2.1}] = 0.221$$

$$\lambda_i = \frac{0.05 \times f_{t28}}{\left(2 + 3\frac{b_0}{b}\right)\rho} = \frac{0.05 \times 2.1}{\left(2 + 3 \times \frac{120}{650}\right).0.0104} = 3.95$$

$$\lambda_v = \frac{2}{5} \lambda_i = 0.4 \times 3.95 = 1.58$$

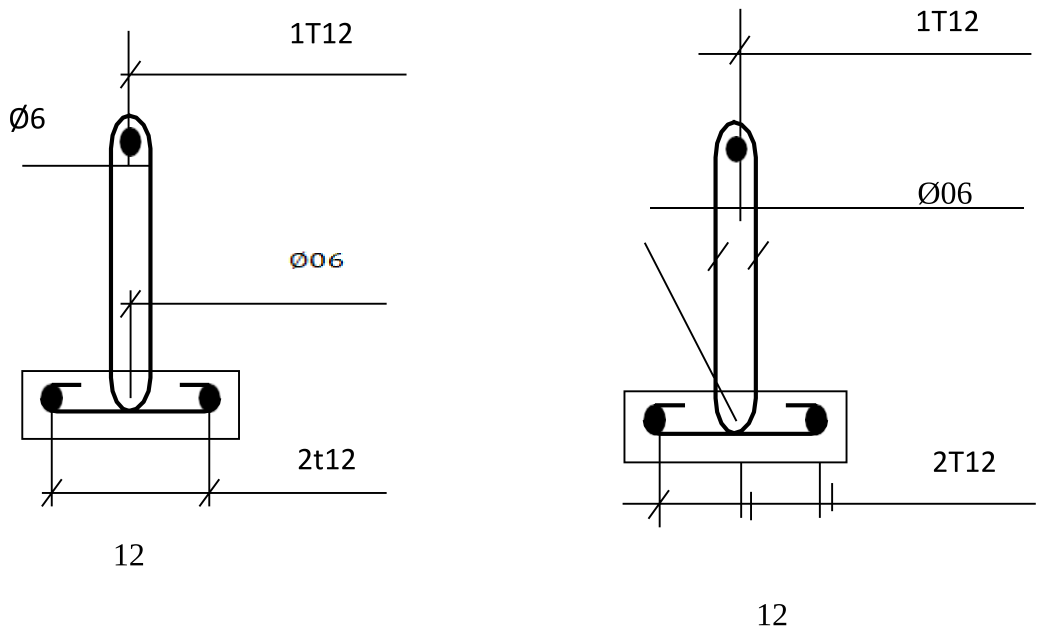
$$I_0 = \frac{bh^3}{12} + n[A_s(\frac{h}{2} - h)^2] = \frac{65 \times 20^3}{12} + 15 \times 2.26(\frac{20}{2} - 18)^2 = 45502.93 cm^4$$

$$I_{fi}^{g} = \frac{1.1\ I_0}{(1 + \lambda_I . \mu)} = \frac{1.1 \times 45502.93}{(1 + 3.95 \times 0.221)} = 26724.270\ Cm^4$$

$$I_{fv}^{g} = \frac{1.1\ I_0}{(1 + \lambda_v . \mu_g)} = \frac{1.1 \times 45502.93}{(1 + 3.95 \times 1.58)} = 6912.47 Cm^4$$

IV.5.6.9.admissible deflection:

L=270cm<400cm ⇨ $\Delta f_{max} = \frac{L}{500} = \frac{270}{400} = 0.675$cm



FigIV. 23: Beam reinforcements **[1].**

IV.5.7.Compression slab:

The compression slab is poured over the entire floor surface, with a thickness of 4cm. The reinforcement of the compression slab must be made by a grid in which the dimensions of the meshes must not exceed :

- 33cm: parallel to the joists.
- 20cm: perpendicular to the beams.

In practice (in Algeria, we consider a 20cm mesh)

IV.5.7.1.Reinforcement perpendicular to beams:

The cross-sectional area of steel perpendicular to the ribs, expressed in cm2/ml, must be at least equal to: $A_{Nervure} = 4.\frac{L_1}{f_e}$

with (L_1in cm) with: spacing :St=20cm

L1: distance between joist axes (L1=65cm)

Diameter perpendicular to the beam, Fe=400MPa

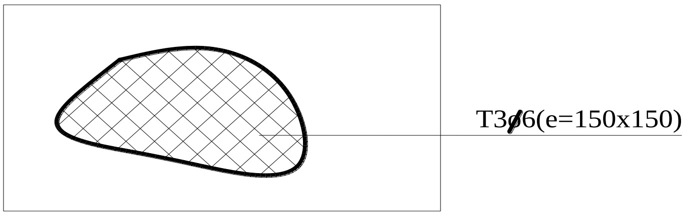
(Fe: yield strength of steels used)

Welded trellis grid (TLE520).

$$A_{Nervure} = 4 \times \frac{65}{400} = 0.65 \Rightarrow A_{Nervure} = 3\emptyset 6 \Rightarrow 0.85cm^2$$

IV.5.8.Conclusion:

For the reinforcement of the compression slab, a welded mesh is used with a mesh size of 20cm in both directions (15×15)



FigIV. 24: Reinforcement layout for compression slab

IV.6.Staircase calculation :

IV.6.1.Introduction :

Staircases are used to link the various levels of a building by successive steps.

Our building has a type of staircase with two flights consisting of a reinforced concrete bench and rectangular steps.

- floor height He=3.74m
- step height h=17cm
- number of steps n=22
- step width g = 30 cm

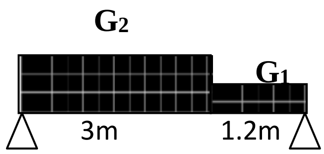
From which we adopt 11 steps per flight

IV.6.2.Loads and overloads:

The load is given for a strip 1ml wide.

IV.6.2.1.Valuation of loads :

a. The flight : G=7.55KN/ml Q =2.5KN/ml



b.The landing : G=5.35KN/ml Q =2.5KN/ml

IV.6.2.2.Load combinations:

a.The landing :

Q_u = 1,35 G + 1,5 q = (1,35 x 5.35) + (1,5 x 2.5) =10.972 KN/ml

Q_{ser}= G + q = 5.35 + 2.5=7.85KN/ml

b.The flight :

Q_u = 1,35 G + 1,5 q = (1,35 x 7.55) + (1,5 x 2.5) =13.94 KN/ml

Q_{ser}= G + q = 7.55 + 2.5=10.05KN/ml

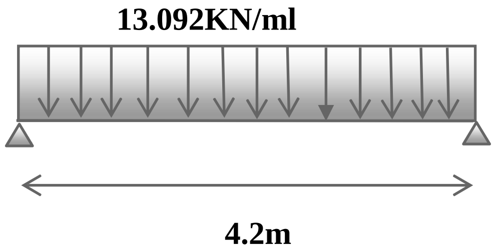
IV.6.2.3.The equivalent load :

$$Q_{eq} = \frac{Q_1 \times l_1 + Q_2 \times l_2}{l_1 + l_2}$$

a. In the ULS

$$Q_{eq} = \frac{13.94 \times 3 + 10.972 \times 1.2}{3+1.2}$$

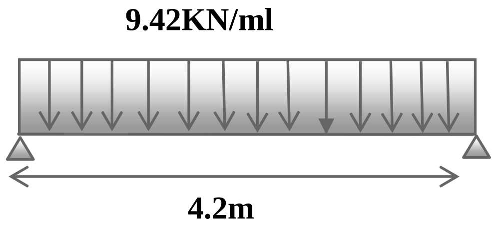
$$Q_{eq} = 13.092KN/ml$$



b. In the SLS

$$Q_{eq} = \frac{10.05 \times 3 + 7.85 \times 1.2}{3+1.2}$$

$$Q_{eq} = 9.42KN/ml$$



IV.6.3.The calculation of the moments:

IV.6.3.1.Isostatic moments :

a. In the ULS:

$$M_0 = q_{eq} \times \frac{l^2}{8} = 13.092 \times \frac{4.2^2}{8} = 28.86KN.m$$

b. In the SLS:

$$M_0 = q_{sq} \times \frac{l^2}{8} = 9.42 \times \frac{4.2^2}{8} = 20.77KN.m$$

IV.6.3.2.Bending moments:

IV.6.3.2.1.On supports:

a. In the ULS:

$$M_u = 0.3M_0 = 0.3 \times 28.86 = 8.65KN.m$$

b. In the SLS:

$$M_s = 0.3M_0 = 0.3 \times 20.77 = 6.23KN.m$$

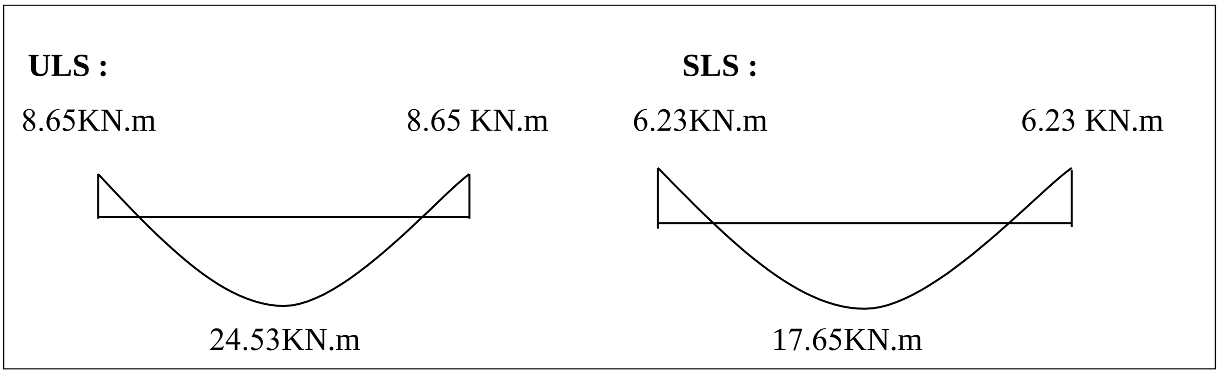
IV.6.3.2.2.In the span:

a. In the ULS:

$$M_u = 0.85M_0 = 0.85 \times 28.86 = 24.53KN.m$$

b. In the SLS:

$$M_s = 0.85M_0 = 0.85 \times 20.77 = 17.65KN.m$$



FigIV. 25: Diagrams of bending moments in spans and supports

<u>IV.6.4.Calculation of reinforcement</u>:

We consider a rectangular section subjected to simple bending, taking a strip of

Width b=1m

Table IV. 6: Reinforcement calculations .

b(cm)	d(cm)	F_{t28}(MPa)	h (cm)	σ_{su}(MPa)	F_{c28}(MPa)	F_e(MPa)	F_{bc}(MPa)	d'(cm)
100	13	2.1	15	348	25	400	14.2	2

a. In span :

$$u_{bu} = \frac{M_u}{bd^2 f_{bu}} = \frac{24.53\times10^{-3}}{1\times0.13^2\times14.2} = 0.102$$

$$u_{bu} < u_l$$

$$\alpha = 1.25\left(1-\sqrt{1-2u_{bu}}\right) = 0.134$$

$$z_d = d(1-0.4\alpha) = 13(1-0.4\times0.134) = 12.30cm$$

$$A_t = \frac{M_u}{z.\sigma_s} = \frac{24.53\times10}{0.123\times348} = 5.73cm^2$$

The choice : 6HA12/ml with :A_t =6.78cm^2 S_t=15cm

b. On supports:

$$u_{bu} = \frac{M_u}{bd^2 f_{bu}} = \frac{8.65\times10^{-3}}{1\times0.13^2\times14.2} = 0.036$$

$$u_{bu} < u_l$$

$$\alpha = 1.25\left(1-\sqrt{1-2u_{bu}}\right) = 0.045$$

$$z_d = d(1-0.4\alpha) = 13(1-0.4\times0.045) = 12.77cm$$

$$A_s = \frac{M_u}{z.\sigma_s} = \frac{8.56\times10}{0.127\times348} = 1.94cm^2$$

The choice : 4HA12/ml with :A_s =4.52cm^2 S_t=25cm

IV.6.4.1.Calculation of distribution reinforcement :

a. In span : $A = \frac{6.78}{4} = 1.70cm^2$ $\Rightarrow$ 4HA10/ml A=3.14cm^2 S_t=25cm

b. On supports: $A = \frac{4.52}{4} = 1.13cm^2$ $\Rightarrow$4HA10/ml A=3.14cm^2 S_t=25cm

IV.6.5.Verification:

IV.6.5.1.Verification of ULS:

IV.6.5.2.Verification of the non-fragility condition :

$$A_{min} = 0.23 \times b \times d \frac{f_{t28}}{f_e} = 0.23 \times 100 \times 13 \times \frac{2.1}{400} = 1.57cm^2$$

$$A_{span} = 6.78cm^2 > 1.57cm^2 \text{ ………………Checked condition}$$

$$A_{supp} = 4.52cm^2 > 1.57cm^2 \text{ ………………Checked condition}$$

IV.6.5.3.Verification of shear force :

For the shear force, verification of the shear will suffice in the worst case.

$$\tau_u = \frac{T_u}{b.d} = \frac{27.49}{1000 \times 130} = 0.21MPA$$

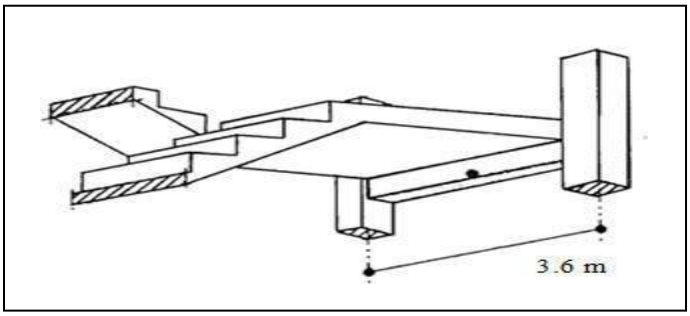
$$\bar{\tau}u = min\left(\frac{0.15 \times f_{td}}{\gamma_b}; 5MPA\right) = 2.5MPA$$

$$\tau_u = 0.21MPA < \bar{\tau}u = 2.5MPA \text{ ……………..Checked condition}$$

IV.7.Landing beam study :

If the landing beam is recessed at 2 ends and embedded in the thickness of the landing designed to support the stairs
be a staircase support, the calculation will be carried out as for a rectangular beam of dimensions
dimensions (b x h), subjected to stresses due to its own weight, to the load it receives from the stairs (support reactions), and subjected to simple bending.
According to the fundamental combinations ELU and ELS, and considering cracking

of little
detrimental cracking.



FigIV. 26:3D view of the landing beam **[1].**

IV.7.1.Evaluation of expenses :

The calculation is performed in simple bending for a beam partially embedded at the end in columns
and uniformly loaded.

IV.7.1.1.Permanent loads :

$$\frac{L_{mqx}}{15} \leq h_p \leq \frac{L_{mqx}}{10} \rightarrow \frac{595}{15} \leq h_p \leq \frac{595}{10} \rightarrow 39.66 \leq h_p \leq 59.5$$

According to RPA99 /V2003 h_p= 45cm

$0.4\, h_p \leq b \leq 0.8\, h_p \longrightarrow 18 \leq b \leq 36 \Longrightarrow b = 30cm$

dead weight of landing beam: 0.30*0.45*25 = 3.375 KN/m

Wall weight: = 2.85 ×1.87=5.33KN/ml

IV.7.1.2.Operating overload:

Q=2.50KN/m

IV.7.1.3.Load combination :

Bearing weight: Gp=5.35

G= 3.375+5.33+5,35 =14.055 KN/ml.

ELU : Pu = 1,35 x 14.055 + 1,5 x 2,5 = 22.724KN/m

ELS : Ps = 14.055+2.50 = 16.555 KN/m

$fc28$ = 25 MPa

γb = 1.5

fbu = 14.17 MPa

σb = 15MPa

Fe = 400 MPa

$ft28$ = 2.1 MPa

γs = 1.15 σs = 348MPa

IV.7.2.Calculating moments :

'ELU: $\frac{L^2}{8}$ **M0**=qu. $\frac{L^2}{8}$ = 22.724. $\frac{5.95^2}{8}$ = 100.56 KN.m. L =5.95m

Span → **Mt** = 0,80 M0 = 0,80 x100.56 = 80.448KN.m

Support → **Ma** = 0,30M0=0,30 x100.56= 30.168 KN.m à l'ELS:**M0**= qu. $\frac{L^2}{8}$ = 16.555. $\frac{5.95^2}{8}$ = 73.261KNm.

Span → **Mt** = 0, 80 M0 = 0,80 x73.261= 58.608KN.m

Support → **Ma** = 0,30M0= 0,30 x73.261 = 21.978KN.m

Table IV. 7: Result of reinforcement at ULS for beam

Position	**Mu**(KNm)	**b**(m)	**d** (m)	μ_t	α	**Z** (m)	**As** (cm²/ml)
Span	80.448	0.30	0.405	0.115	0.153	0.380	6.083
Support	30.168	0.30	0.405	0.0432	0.0552	0. 396	2.189

IV.7.2.1.Armatures adopted :

4HA14 (6.15cm²/ml) As span reinforcement

2HA12 (2.26 cm²/ml) As supporting reinforcement

IV.7.3.Verification of shear force at ULS :

IV.7.3.1.Shear force calculation :

Tu = $\frac{qu.l}{2}$ = $\frac{22.724*5.95}{2}$ = 67.60 KN   Tu = 67.60KN

Low-impact cracking**:** : $\tau u \leq \overline{\tau u}$ = min ($\frac{0.20*fc28}{\gamma b}$; 5MPa) = min ($\frac{0.20*25}{1.5}$; 5MPa)

= min (3.33MPa ; 5MPa)

$\overline{\tau u}$: Permissible shear stress

τu = $\frac{Tu}{b*d}$ = $\frac{67.60*10^{-3}}{0.3*0.405}$ = 0.556**MPa**   1 MPa = 10^3KN / m²

τu= 0.556 MPa < 3.33MPa → condition Checked → Transverse reinforcements are

straight reinforcement. The diameter of the crossbars is directly related to the diameter of the reinforcement longitudinal bars according to the expression:

Core reinforcement diameter: According to RPA/2003:

$\emptyset t \geq \frac{\emptyset l}{3} = \frac{12}{3}$ = 4 mm ⟹ *soit*, $\emptyset t$= 6 mm

The transverse reinforcements will consist of a frame and a 6 mm pin.

diameter, i.e. a total section of: At = 4HA8= 2.01cm^2

IV.7.3.2.Spacing of transverse reinforcement:

Spacing conditions according to RPA/2003:

a. **In the nodal zone**: St ≤ min (12$\emptyset t$)

; h/4) = min (12×1.2; 45/4) = min (14.4; 11.25) = 11.25cm.

a chosen spacing of : St = 8 cm. The length of the nodal zone: 2× h = 2×45=90 cm.

b. In the running zone: St ≤ h/2 = 45/2 = 22.5 cm So a chosen spacing of : St = 20 cm.

The cross-sectional area of transverse reinforcement is deduced from the following

Expression :

At = 0,003 ×s ×b = 0,003 ×20 × 30 = 1, 80cm^2 .

The cross-section of reinforcement adopted verifies this condition:

A (Adopted) = 1.80 cm^2 > 1, 51cm^2

IV.7.4.À l'SLS :

a. In span:

Since cracking is not harmful, and the steel used is FeE400, then the SLS stress check will be will be simplified as follows

$\boldsymbol{\alpha\mu t} \leq \frac{\gamma-1}{2} + \frac{f_{C28}}{100}$ avec $\gamma = \frac{Mu}{Ms} = \frac{80.448}{58.608} = 1.37$

$\boldsymbol{\alpha\mu t} \leq \frac{1.37-1}{2} + \frac{25}{100} \leq 0.435$

$\boldsymbol{\alpha\mu t} = 0.115 \leq \mathbf{0.435}$ → Checked

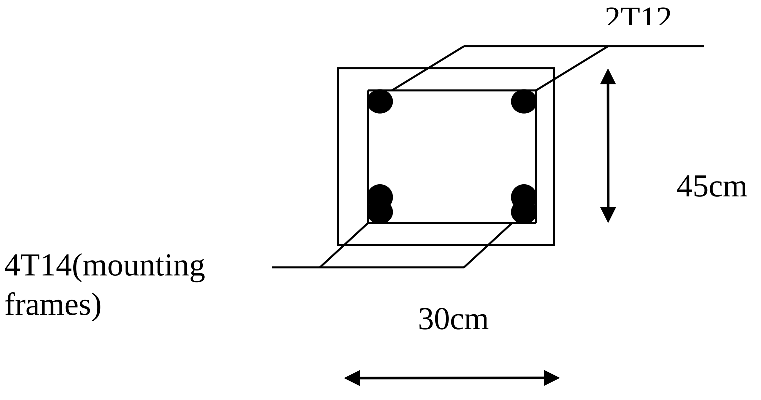
b. In Support

$\boldsymbol{\alpha\mu t} \leq \frac{\gamma-1}{2} + \frac{f_{c28}}{100}$ avec $\gamma = \frac{Mu}{Ms} = \frac{30.168}{21.978} = 1.37$

$\boldsymbol{\alpha\mu t} \leq \frac{1.37-1}{2} + \frac{25}{100} \leq 0.435$

$\boldsymbol{\alpha\mu t} = 0.0432 \leq \mathbf{0.435}$ → Checked

So there's no need to check the concrete stress σbc< σbc



FigIV. 27: Landing beam reinforcement

This modelling made the structural engineer's task easier, as we were able to exploit

that the internal forces at all points of the building according to the various

desired combinat ions

IV.8.Column reinforcement :

IV.8.1.Reinforcement :

When calculating the reinforcement of our structure's load-bearing elements, we must take the following into account

the following combinations :

- Fundamental combinations : $\begin{cases} 1{,}35\ G + 1.5p \\ G + P \end{cases}$

We have only one type of post to study: Type (30 × 40) cm^2 Type (40 × 50) cm^2

IV.8.2.Minimum reinforcement required by BAEL :

Amin = max (0,2 . b .h/100 ; 4 cm^2)

IV.8.3.Determining the nodal zone :

- The nodal zone consists of the beam-column node itself and the ends of the

ends of the contributing bars.

- The lengths to be taken into account for each bar are given below

h' = max (he/6 ; b1 ; h1 ; 60 cm)

L' = 2 h'

IV.8.4.Column (ground floor) :

S = (30 × 40) cm^2

S = (40× 50) cm^2

IV.8.4.1. 1st case: columns : S = (30 × 40) cm^2

ELU R : (1,35 G + 1,5 P)

Solicitations taken into account

Nmax = 900.86KN

Mmax= 15.46 KN.m

e $= \frac{M}{N} = \frac{15.46}{900.86}$ = 0,0171cm

Fc28 = 25 MPa ⇒ σbc = 0,85 .$\frac{25}{1.5}$ = 14,20 MPa

A= $\frac{N-100\sigma_{bc}\times B^2}{\sigma_s\times 100} = \frac{900.86\times 10^3-100\times 14.20\times 40^2}{348\times 100}$ = -39.40

A1= -39.40 < 0 ⇒ A = 0 cm^2

$\lambda = 3{,}46 \cdot \frac{Lf}{b} = 3{,}46 \cdot \frac{0.7\times374}{40} = 22.64$

$\lambda < 50 \Rightarrow \alpha = \frac{0.85}{1+0.25(\frac{22.64}{35})^2} = 0.76$

α = 0,76

$A2 = \frac{\gamma_s}{F_e}[\frac{N_u}{\alpha} - \frac{B_r \times F_{c28}}{0.9\times\gamma_b}]$

Br = (h – 2) (b – 2) = (30 – 2) (40 – 2) = 1064 cm^2

$A2 = \frac{1.15}{400}[\frac{900.86\times10^3}{0.76} - \frac{1064\times 25\times100}{0.9\times1.5}]\frac{1}{100} = -22.56 cm^2$

IV.8.4.1.1.Minimum reinforcement :

1. According to BAEL 91 :

A1 min = max (0,2 . b . h / 100 ; 4 cm^2) = max (2.4 ; 4) cm^2

A1 min = 4 cm^2

2 Next RPA 99 version 2003

A2 min = 0,80 % . b . h ⇒ A2 min = 0,80 % . 40 . 30 = 9.6 cm^2

⇒ Amax = max (A1 ; A2 ; A1min ; A2min) cm^2

Amax = max (0 ; 0; 4 ; 9.6), Amax = 9.6 cm^2

The reinforcement cross-section adopted for

the columns (40 × 30) cm^2 A = 9.6cm^2 Choix : 6T14⟶ A = 9.18cm^2

IV.8.4.1.2.Checking the cutting effect :

Tu max = 36.31KN

$\tau u = \frac{T_{umax}}{b\times d} = \frac{36.31\times10^3}{40\times27\times100}$ =0.336 MPa

Low-damage cracking: $\overline{\tau u}$< min (0,13 fc28 ; 4 MPa) = 3,25 MPa

So τu < $\overline{\tau u}$ ⇒ according to the shear force calculation, the shear condition is Checked

IV.8.4.1.3.Determining transverse reinforcement :

∅t ≥ ∅max ⇒ ∅t ≥ $\frac{1}{3}$. 14 = 4.66 mm we take ∅t = 5 mm

IV.8.4.1.4.Spacing of transverse reinforcement :

1. According to BAEL 91 :

St = min (15 ∅min ; 40 cm ; b + 10 cm)

= min (15 . 1,4 ; 40 cm ; 50 cm)

St = 10 cm

In the current zone : St ≤ 15 ∅Lmin = 15 . 1,4 = 21 cm St = 15 cm

IV.8.4.1.5.Determining the nodal zone :

According to RPA99 article 7.5.22. RPA 99 version 2003

L' = 2×h = 2 × 30= 60 cm

h' = max ($\frac{he}{6}$; b1 ; h1 ; 60 cm)

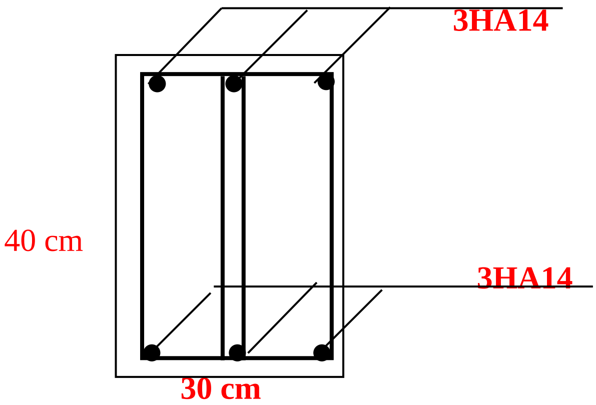
h' = max ($\frac{374}{6}$; 40 ; 30 ; 60) cm = 62.33cm

So : h' = 62.33cm L'= 60cm

Longitudinal bar overlap :

LR = 40 ∅max = 40 . 1,4 = 56 cm

We take LR = 60 cm



FigIV. 28:Column reinforcement **(no need)0.**

<u>IV.8.4.2. 2 nd case: columns</u> : S = (40× 50) cm^2

ELUR : (1,35 G + 1,5 P)

Solicitations taken into account

Nmax = 562.29KN

Mmax=12.01 KN.m

e =$\frac{M}{N}$ = $\frac{12.01}{562.29}$ = 0,0213cm

Fc28 = 25 MPa ⇒ σbc = 0,85 .$\frac{25}{1.5}$ = 14,20 MPa

A= $\frac{N-100\sigma_{bc}\times B^2}{\sigma_s\times 100}$ = $\frac{562.29\times 10^3-100\times 14.20\times 50^2}{348\times 100}$ = -85.85

A1= -85.85 < 0 ⇒ A = 0 cm^2

λ = 3,46 . $\frac{Lf}{b}$ = 3,46 . $\frac{0.7\times 374}{50}$= 18.11

λ < 50 ⇒ α = $\frac{0.85}{1+0.25(\frac{18.11}{35})^2}$ = 0.79 α = 0,79

A2 = $\frac{\gamma_s}{F_e}[\frac{N_u}{\alpha} - \frac{B_r\times F_{c28}}{0.9\times\gamma_b}]$

Br = (h – 2) (b – 2) = (40 – 2) (50 – 2) = 1824 cm^2

A2 = $\frac{1.15}{400}[\frac{562.29\times 10^3}{0.79} - \frac{1824\times 25\times 100}{0.9\times 1.5}]\frac{1}{100}$ = -76.64cm^2

IV.8.4.2.1. Minimum reinforcement :

1. According to BAEL 91 :

A1 min = max (0,2 . b . h / 100 ; 4 cm^2) = max (4 ; 4) cm^2

A1 min = 4 cm^2

2 Next RPA 99 version 2003

A2 min = 0,80 % . b . h ⇒ A2 min = 0,80 % . 50 . 40 = 12cm^2

⇒ Amax = max (A1 ; A2 ; A1min ; A2min) cm^2

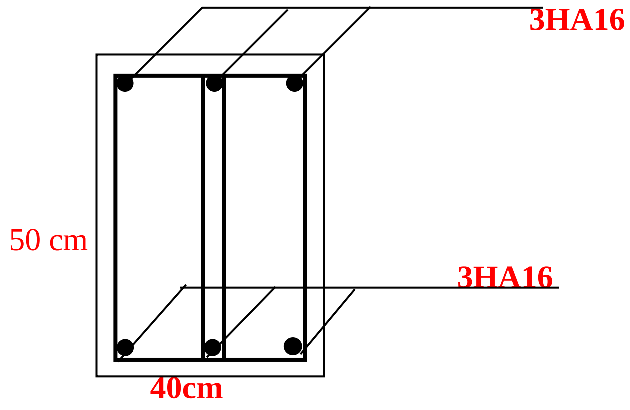
Amax = max (0 ; 0; 4 ; 12), Amax = 12cm^2

The reinforcement cross-section adopted for

the columns (50 × 40) cm^2 A = 12cm^2 Choix : 6T16 → A = 12

Table IV. 8: calculation of column reinforcement **(no need).**

Type	A1	A2	A1min	A2min	Amax	The choice
Column	0	0	4	9.6	9.6	6T14
Gantry column	0	0	4	12	12	6T16



FigIV. 29:Column reinforcement **(no need).**

IV.8.4.2. 2.Checking the cutting effect :

Tu max = 0.17KN

$\tau u = \frac{T_{umax}}{b \times d} = \frac{0.17 \times 10^{3}}{50 \times 36 \times 100}$ =0.94×10^{-3}MPa

Low-damage cracking: $\overline{\tau u}$< min (0,13 fc28 ; 4 MPa) = 3,25 MPa

So τu < $\overline{\tau u}$ ⇒ according to the shear force calculation, the shear condition is verified

IV.8.4.2. 3.Determining transverse reinforcement :

∅t ≥ ∅max ⇒ ∅t ≥ $\frac{1}{3}$. 16 = 5.33mm we take ∅t = 6mm

IV.8.4.2. 4.Spacing of transverse reinforcement :

1. According to BAEL 91 :

St = min (15 ∅min ; 40 cm ; b + 10 cm) **[36]**

= min (15 . 1,6 ; 40 cm ; 60 cm)

St = 10 cm

In the current zone : St ≤ 15 ∅Lmin = 15 . 1,6 = 24 cm St = 15 cm

IV.8.4.2. 5.Determining the nodal zone :

According to RPA99 article 7.5.22. RPA 99 version 2003

L' = 2×h = 2 × 40= 80 cm

h' = max ($\frac{he}{6}$; b1 ; h1 ; 60 cm)

h' = max ($\frac{374}{6}$; 50 ; 40 ; 60) cm = 62.33cm

So : h' = 62.33cm L'= 60cm

Longitudinal bar overlap :

LR = 40 ∅max = 40 . 1,6= 64 cm

We take LR = 64 cm

IV.9. Beam reinforcement :

We have 2 types of beams to study:

- Main beam (30 × 60)

- Secondary beam (30 × 50)

For beam reinforcement, the extreme steel percentages given in RPA99.

1. The minimum percentage of longitudinal reinforcement over the entire length of the beam is

0.5% in cross-section.

2. Maximum total percentage of longitudinal steels of :

- 4% in the running zone .

- 6% in the overlap zone .

Since normal forces are zero, beams are calculated in simple bending.

IV.9.1.Calculation of reinforcement:

a. Determining forces :

Combinations taken into account

b. Sustainable situation and transaction :

ELU : 1,35 G + 1,5 P

ELS : G + P

IV.9.1.1.Reinforcement of spans: (sustainable situation) ULS

- Main beam (30 × 60)

M_{ut}=84.20 KN.m

According to B.A.E.L 91 :

$$u_{bu} = \frac{M_u}{bd^2 f_{bu}} = \frac{84.20}{0.30 \times 0.54^2 \times 14.2 \times 10^3} = 0.0677$$

$$\alpha = 1.25\left(1 - \sqrt{1 - 2u_{bu}}\right) = 0.0877$$

$$z_d = d(1 - 0.4\alpha) = 54(1 - 0.4 \times 0.0877) = 52.10cm$$

$$A_t = \frac{M_u}{z_d . \sigma_s} = \frac{84.20}{0.521 \times 348 \times 10^3} = 0.000447m = 4.64cm^2$$

A= 4.64cm^2

Choice of bars: 3HA14. As =4.62.cm^2

IV.9.1.1.2.Reinforcement on support: (accidental situation) G+Q

M_a= -66.99 KN.m

$$u_{bu} = \frac{M_u}{bd^2 f_{bu}} = \frac{66.99}{0.30 \times 0.54^2 \times 14.2 \times 10^3} = 0.0539$$

$$\alpha = 1.25\left(1 - \sqrt{1 - 2u_{bu}}\right) = 0.0692$$

$$z_d = d(1 - 0.4\alpha) = 54(1 - 0.4 \times 0.0692) = 52.50cm$$

$$A_t = \frac{M_u}{z_d.\sigma_s} = \frac{66.99}{0.525 \times 348 \times 10^3} = 0.000366m = 3.66cm^2$$

A = 3.66 cm^2

Choice of bars: 3HA12. As =3.39.cm^2.

IV.9.2.Checks required for beams :

IV.9.2.1.The condition of non-fragility :

Amin =0.23×b×d×$\frac{F_{t28}}{F_e}$ = 1.956cm^2

Amin > 0.23× 30×54×2.1/400 = 1.956 cm^2
Amin = 1.956 cm^2 (condition Checked)

Percentage required by RPP99

The minimum total percentage of longitudinal reinforcement over the entire length of the beam.

0.5% in all sections: Amin > 0.5%.b.h **[37].**
A_{min}> 0.5%.(30×60)=9 cm^2

Note that As min is greater than the previously calculated cross-section of the span.

So we adopt As min.

Choice of bars: 6HA14. As = 9.23 cm^2

IV.9.2.2.Transverse reinforcement: (tangential stress)

V_{umax} = 35.63 KN.m

$$\tau_{umax} = \frac{V_{umax}}{b \times d} = \frac{0.03563}{0.30 \times 0.54} = 0.219MPa$$

τ_u = min(0.15×F_{c28} ; 4 MPa) = 3.75 MPa
$\tau < \bar{\tau}$ Condition Checked

IV.9.2.3.Construction layout :

St: spacing of transverse reinforcement courses**.**

a. In nodal zones

$S_t \leq$ min (h/4; 12Ø; 30cm) =10cm

b. In the current zone

$S_t \leq h/2$ => S_t =20cm

h: beam height.

$S_t \leq$ min (0.9d; 40cm)
St ≤min (0.486; 40cm) => the condition is Checked

Minimum cross-section of transverse reinforcement BAEL A.5.1.2 3 :
$S_t \leq A_t \times F_e/0.4 \times b_0$
$A_t \geq 0.4 \times b_0 \times S_t/F_e$

A_t= 0.4× 0.30× 0.2/400 = 0.6cm^2

Condition required by RPP99

The minimum amount of transverse reinforcement is given by :

A_t= 0. 003.St. b **[37].**
A_t=0.003×0.10×0.30=0.9cm^2

(Ø 8" frame + Ø 8" stirrup) = 1.79 cm^2

IV.9.1.2.Reinforcement of spans: (sustainable situation) ULS

- Secondary beam (30 × 50)

M_{ut}=44.66 KN.m

According to B.A.E.L 91 :

$$u_{bu} = \frac{M_u}{bd^2 f_{bu}} = \frac{44.66}{0.30 \times 0.45^2 \times 14.2 \times 10^3} = 0.0517$$

$$\alpha = 1.25\left(1-\sqrt{1-2u_{bu}}\right) = 0.0663$$

$$z_d = d(1-0.4\alpha) = 54(1-0.4 \times 0.0877) = 43.80cm$$

$$A_t = \frac{M_u}{z_d.\sigma_s} = \frac{44.66}{0.438 \times 348 \times 10^3} = 0.000292m = 2.92cm^2$$

A= 2.92cm^2

IV.9.1.2.1.Reinforcement on support: (accidental situation) G+Q

M_a= -51.67 KN.m

$$u_{bu} = \frac{M_u}{bd^2 f_{bu}} = \frac{51.67}{0.30 \times 0.45^2 \times 14.2 \times 10^3} = 0.0596$$

$\alpha = 1.25(1 - \sqrt{1 - 2u_{bu}})$ =0.0768

$z_d = d(1 - 0.4\alpha) = 45(1 - 0.4 \times 0.0768) = 43.61cm$

$A_t = \frac{M_u}{z_d \cdot \sigma_s} = \frac{51.67}{0.4361 \times 348 \times 10^3} = 0.000334m = 3.34cm^2$

A = 3.34 cm^2

Choice of bars: 3HA12. As = 3.39cm^2.

IV.9.3.Checks required for beams :

IV.9.3.1.The condition of non-fragility :

Amin =0.23×b×d×$\frac{F_{t28}}{F_e}$ = 1.63cm^2

Amin > 0.23×30×45×2.1/400 = 1.63 cm^2
Amin = 1.63 cm^2 (condition Checked)

Percentage required by RPP99

The minimum total percentage of longitudinal reinforcement over the entire length of the beam.**[38]**

0.5% in all sections: Amin > 0.5%.b.h **[38].**
A_{min}> 0.5%.(30×50)=7.5 cm^2

Note that As min is greater than the previously calculated cross-section of the span.

So we adopt As min.

Choice of bars: 3HA12. 3HA14 As = 8.01 cm^2

IV.9.3.2.Transverse reinforcement: (tangential stress)

V_{umax} = 49.09 KN.m

$\tau_{umax} = \frac{V_{umax}}{b \times d} = \frac{0.04909}{0.30 \times 0.45}$= 0.363MPa

τ_u = min(0.15×F_{c28} ; 4 MPa) = 3.75 MPa

$\tau < \bar{\tau}$ Condition verified

IV.9.3.3.Construction layout :

St: spacing of transverse reinforcement courses**.**

a. In nodal zones

$S_t \leq$ min (h/4; 12Ø; 30cm) =12.5cm

b. In the current zone

$S_t \leq h/2$ => S_t =20cm

h: beam height.

$S_t \leq$ min (0.9d; 40cm)
St $\leq$min (0.405; 40cm) => the condition is Checked

Minimum cross-section of transverse reinforcement BAEL A.5.1.2 3 :
$S_t \leq A_t \times F_e / 0.4 \times b_0$
$A_t \geq 0.4 \times b_0 \times S_t / F_e$

A_t= 0.4× 0.30× 0.2/400 = 0.6cm^2

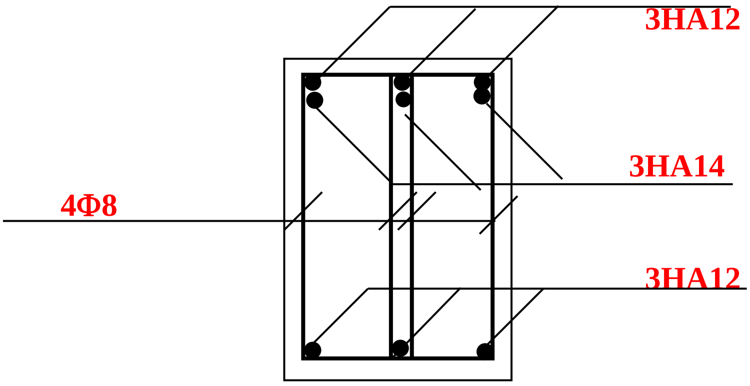
Condition required by RPP99

The minimum amount of transverse reinforcement is given by :

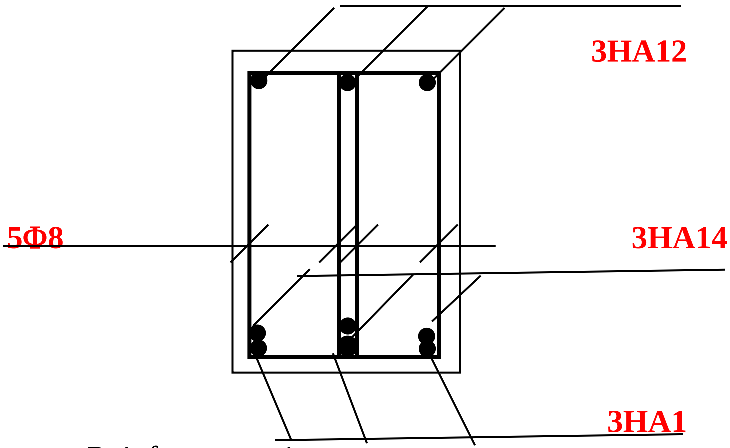
A_t= 0. 003.St. b **[38].**
A_t=0.003×0.125×0.30=1.25cm^2

(Ø 8" frame + Ø 8" stirrup) = 2.49 cm^2.

- Main beam (30 × 60)



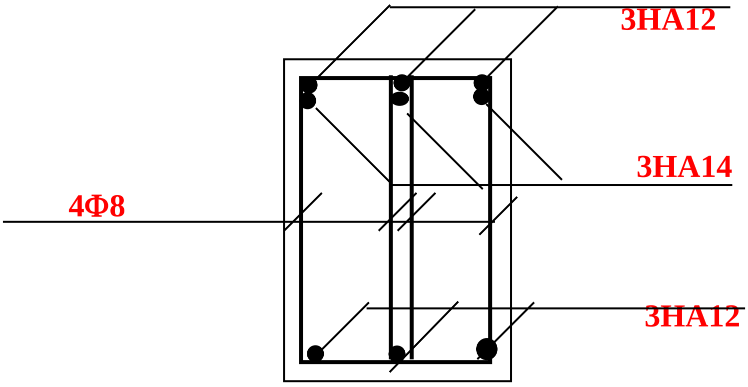
Reinforcement on supports



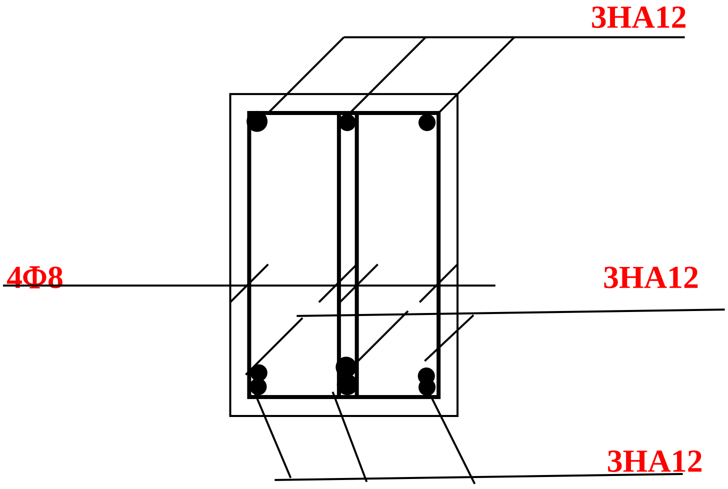
Reinforcement in spans

FigIV. 30: Reinforcement diagram for 30×60 main beam **(no need).**

- Secondary beam (30 × 50)



Reinforcement on supports



Reinforcement in spans

FigIV. 31: Reinforcement diagram for 30×50 secondary beams **(no need).**

IV.10.Foundations:

IV.10.1.Introduction:

Foundations are structures used to transmit the loads from the superstructure from the superstructure to the ground: Dead weight or permanent loads, operating loads, climatic and seismic loads.

The choice of foundation type depends on :

- Type of structure to be built.
- The nature and homogeneity of the soil.

- The bearing capacity of the foundation soil.

- Economic reasons.

- Ease of construction.

With a bearing capacity of the ground equal to 3 bar, the following should be planned

a priori, shallow foundations of the : insulated footings

IV.10.2. Pre-dimensioning of insulated footings:

If we call A and B the sides of the footing at sides a and b of the column, there are two conditions to be met to be satisfied in order to design a rigid footing under centred loading

IV.10.2.1. calculation of the insulated footing Type 1 :

The surface area of the footing Ss must satisfy the following relationship :

$$S_s \geq \frac{N}{\bar{\sigma}_{sol}}$$

Ss : surface area of the footing in cm2 = A × B

N : the normal force acting on the baseplate obtained by the ROBOT N=656.96 KN =65.696t

$\bar{\sigma}_{sol}$: Permissible soil stress. $\bar{\sigma}_{sol} = 3\ bar$ =30t/m^2

$$S_s = \frac{65.696}{30} = 2.18m^2$$

We assume that the footings are square, so we have

$\Rightarrow A = B = \sqrt{S_s}$=$\sqrt{2.18}$ =1.47m

We select A = B = 1.5 m

IV.10.2.2.The height of the footing :

$h_t \geq d + 0.05$ m

with :

$$d \geq MAX\left(\frac{A-a}{4};\frac{B-b}{4}\right) \Rightarrow d \geq MAX\left(\frac{150-30}{4};\frac{150-40}{4}\right)$$

We then have :

$h_t \geq 0.30 + 0.05$ m $\Rightarrow$ $h_t \geq 0.35$ m $\Rightarrow$ h_t =0.35cm

$$\frac{h_t}{3} \leq h_1 \leq \frac{h_t}{2} \quad \Rightarrow \frac{35}{3} \leq h_t \leq \frac{35}{2} \Rightarrow 11.66 \leq h_1 \leq 17.5$$

We take $h_1 = 15cm$

$h_2 = h_t - h_1 = 35 - 15 = 25cm$

IV.10.2.3.Verification of ELS stresses: G+Q

The average stress in the soil σ_{moy} must satisfy the following condition

$$\sigma_{moy} = \frac{3\sigma_1 + \sigma_2}{4} \leq \sigma_{sol}$$

With :

$$\sigma_{1,2} = \frac{N}{S}\left(1 \pm \frac{6.e}{A}\right)$$

$$e = \frac{M}{N}$$

Table IV. 9: Verification of the S1 footing at SLS **(no need).**

Footing (A*B)	**M (KN.m)**	**N (KN)**	**e (m)**	**Ss (m^2)**	σ_1 KN/m^2	σ_2 KN/m^2	σ_{moy} KN/m^2	σ_{sol} KN/m^2	**check**
1.5×1.5	-4.75	656.96	-0.0071	2.25	283.69	300.27	287.83	300	Yes

IV.10.2.3.Verification of ELU stresses: 1.35G+1.5Q

Table IV. 10: Verification of the S1 footing at ULS **(no need).**

Footing (A*B)	**M (KN.m)**	**N (KN)**	**e (m)**	**Ss (m^2)**	σ_1 KN/m^2	σ_2 KN/m^2	σ_{moy} KN/m^2	σ_{sol}*1.5 KN/m^2	**check**
1.5×1.5	-6.57	900.68	-0.0072	2.25	411.83	388.77	406.06	450	yes

IV.10.3.Reinforcement of insulated footings :

For the reinforcement of insulated footings, we use the connecting rod method.

of steel is determined using the following formula :

$$A_X = \frac{N(A-a)}{8(h-c)\sigma_{st}} \qquad \text{Such as : } \sigma_{st} = \frac{f_e}{\gamma_s} = \frac{400}{1.15} = 348MPA$$

With :

N : normal stress at ELU returning to the footing

A : footing dimensions (in cm)

H : footing height

C : steel cover (in cm)

fe: yield strength of steel

Ys: safety coefficient = 1.15

Nu = 900.68 KN

Weight of footing = 1.5×1.5×0.35×25=19.68 KN

Weight of column starter = 0.3×0.4×1.25×25 = 3.75 KN

Weight of soil above the footing = (1.5×1.5×1.25×18) = 50.62KN

Nu = 974.73 KN

Table IV. 11: Footing reinforcement **(no need).**

footing (m)	Column (m)	S (m)	N (KN)	h-c (m)	A_P (cm^2)	Choice of rods		Choice of rods	
1.5×1.5	0.3×0.4	2.25	974.73	0.30	12.69	11HA12	S_t=15	11HA12	S_t=15

IV.10.3.1.calculation of the footing type 2 :

The surface area of the footing Ss must satisfy the following relationship :

$$S_s \geq \frac{N}{\bar{\sigma}_{sol}}$$

Ss : surface area of the footing in cm2 = A × B

N : the normal force acting on the baseplate obtained by the ROBOT N=402.61KN =40.261t

$\bar{\sigma}_{sol}$: Permissible soil stress. $\bar{\sigma}_{sol} = 3\ bar$ =30t/m^2

$$S_s = \frac{40.26}{30} = 1.34\ m^2$$

We assume that the footings are square, so we have

$\Rightarrow A = B = \sqrt{S_s} = \sqrt{1.34}$ = 1.15m

We select A = B = 1.**15** m

IV.10.3.2.The height of the footing :

$h_t \geq d + 0.05$ m

with :

$$d \geq MAX\left(\frac{A-a}{4};\frac{B-b}{4}\right) \Rightarrow d \geq MAX\left(\frac{115-30}{4};\frac{115-40}{4}\right)$$

We then have :

$h_t \geq 0.25 + 0.05$ m $\Rightarrow$ $h_t \geq 0.30$ m $\Rightarrow$ h_t =0.30cm

$$\frac{h_t}{3} \leq h_1 \leq \frac{h_t}{2} \Rightarrow \frac{30}{3} \leq h_t \leq \frac{30}{2} \Rightarrow 10 \leq h_1 \leq 15$$

We take $h_1 = 15cm$

$$h_2 = h_t - h_1 = 30 - 15 = 15cm$$

IV.10.3.3.Verification of ELS stresses: G+Q

The average stress in the soil σ_{moy} must satisfy the following condition

$$\sigma_{moy} = \frac{3\sigma_1 + \sigma_2}{4} \leq \sigma_{sol}$$

With :

$$\sigma_{1,2} = \frac{N}{S}\left(1 \pm \frac{6.e}{A}\right)$$

$$e = \frac{M}{N}$$

Table IV. 12: Verification of the S1 footing at SLS **(no need).**

Footing (A*B)	**M (KN.m)**	**N (KN)**	**e (m)**	**Ss (m^2)**	σ_1 **KN/m^2**	σ_2 **KN/m^2**	σ_{moy} **KN/m^2**	σ_{sol} **KN/m^2**	**check**
1.15×1.15	9.79	402.61	0.024	1.32	371.84	238.17	338.42	300	yes

IV.10.3.4. Verification of ELU stresses: 1.35G+1.5Q

Table IV. 13: Verification of the S1 footing at ULS **(no need).**

Footing (A*B)	**M (KN.m)**	**N (KN)**	**e (m)**	**Ss (m^2)**	σ_1 **KN/m^2**	σ_2 **KN/m^2**	σ_{moy} **KN/m^2**	σ_{sol}***1.5 KN/m^2**	**check**
1.15×1.15	13.43	552.26	0.024	1.32	470.76	365.99	444.56	450	yes

<u>IV.10.4.Reinforcement of footings :</u>

For the reinforcement of footings, we use the connecting rod method.

of steel is determined using the following formula :

$$A_X = \frac{N(A-a)}{8(h-c)\sigma_{st}}$$ Such as : $\sigma_{st} = \frac{f_e}{Y_s} = \frac{400}{1.15} = 348MPA$

With :

N : normal stress at ULS returning to the footing

A : footing dimensions (in cm)

H : footing height

C : steel cover (in cm)

fe: yield strength of steel

Ys: safety coefficient = 1.15

Nu = 552.26 KN

Weight of footing = 1.15×1.15×0.30×25=9.91 KN

Weight of column starter = 0.3×0.4×1.25×25 = 3.75 KN

Weight of soil above the footing = (1.15×1.15×1.25×18) = 29.75KN

Nu = 595.67KN

Table IV. 14: Footing reinforcement **(no need).**

footing (m)	Column (m)	S (m)	N (KN)	h-c (m)	A_P (cm^2)	Choice of rods		Choice of rods	
1.15×1.15	0.3×0.4	2.25	595.67	0.25	11.72	10HA12	S_t=15	10HA12	S_t=15

IV.10.4.1.calculation of the footing Type3 :

The surface area of the footing Ss must satisfy the following relationship :

$$S_s \geq \frac{N}{\bar{\sigma}_{sol}}$$

Ss : surface area of the footing in cm2 = A × B

N : the normal force acting on the baseplate obtained by the ROBOT N=185.84KN =18.584t

$\bar{\sigma}_{sol}$: Permissible soil stress. $\bar{\sigma}_{sol} = 3\ bar$ =30t/m^2

$$S_s = \frac{18.584}{30} = 0.62\ m^2$$

We assume that the footings are square, so we have

$\Rightarrow A = B = \sqrt{S_s}=\sqrt{0.62}$ =0.79 m

We select **A** = B = 1 m

IV.10.4.2.The height of the footing :

$h_t \geq d + 0.05$ m

with :

$$d \geq MAX\left(\frac{A-a}{4};\frac{B-b}{4}\right) \Rightarrow d \geq MAX\left(\frac{100-30}{4};\frac{100-40}{4}\right)$$

We then have :

$h_t \geq 0.20 + 0.05$ m $\Rightarrow$ $h_t \geq 0.25$ m $\Rightarrow$ h_t =0.25cm

$$\frac{h_t}{3} \leq h_1 \leq \frac{h_t}{2} \quad \Rightarrow \frac{30}{3} \leq h_t \leq \frac{30}{2} \Rightarrow 8 \leq h_1 \leq 15$$

We take $h_1 = 15cm$

$$h_2 = h_t - h_1 = 25 - 15 = 10cm$$

IV.10.4.3.Verification of ELS stresses: G+Q

The average stress in the soil σ_{moy} must satisfy the following condition

$$\sigma_{moy} = \frac{3\sigma_1 + \sigma_2}{4} \leq \sigma_{sol}$$

With :

$$\sigma_{1,2} = \frac{N}{S}\left(1 \pm \frac{6.e}{A}\right)$$

$$e = \frac{M}{N}$$

Table IV. 15: Verification of the S3 footing at SLS **(no need).**

Footing (A*B)	M (KN.m)	N (KN)	e (m)	S_s (m^2)	σ_1 KN/m^2	σ_2 KN/m^2	σ_{moy} KN/m^2	σ_{sol} KN/m^2	check
1×1	12.36	185.84	0.066	1	259.43	238.17	254.11	300	yes

IV.10.4.4.Verification of ELU stresses: 1.35G+1.5Q

Table IV. 16: Verification of the S3 footing at ULS **(no need).**

Footing (A×B)	M (KN.m)	N (KN)	e (m)	S_s (m^2)	σ_1 KN/m^2	σ_2 KN/m^2	σ_{moy} KN/m^2	σ_{sol}×1.5 KN/m^2	check
1×1	17.06	254.39	0.067	1	356.65	152.12	305.51	450	yes

<u>IV.10.5.Reinforcement of footings :</u>

For the reinforcement of footings, we use the connecting rod method.

of steel is determined using the following formula :

$$A_X = \frac{N(A-a)}{8(h-c)\sigma_{st}}$$ Such as : $$\sigma_{st} = \frac{f_e}{Y_s} = \frac{400}{1.15} = 348MPA$$

With :

N : normal stress at ULS returning to the footing

A : footing dimensions (in cm)

H : footing height

C : steel cover (in cm)

fe: yield strength of steel

Ys: safety coefficient = 1.15

Nu = 254.39KN

Weight of footing = 1×1×0.25×25= 6.25KN

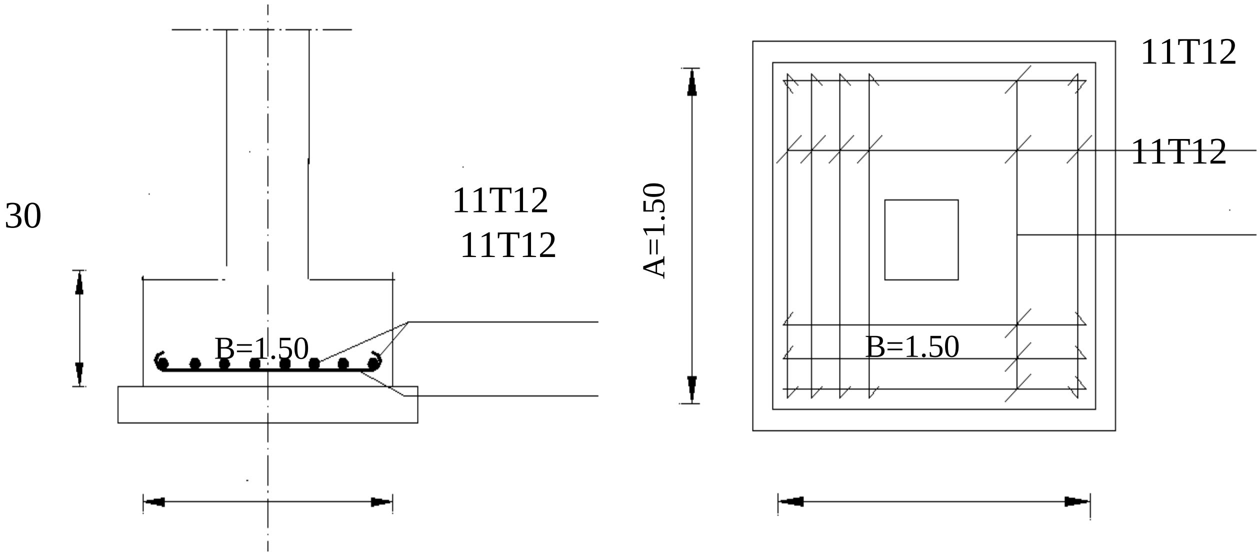
Weight of column starter = 0.3×0.4×1.25×25 = 3.75 KN

Weight of soil above the footing = (1×1×1.25×18) =22.5 KN

Nu = 286.69KN

Table IV. 17: Footing reinforcement **(no need).**

footing (m)	Colum n (m)	S (m)	N (KN)	h-c (m)	A_P (cm^2)	Choice of rods		Choice of rods	
1×1	0.3×0.4	1	286.69	0.20	3.60	6HA12	S_t=20	6HA12	S_t=20



FigIV. 32*:* Diagram of the reinforcement of a footing Type

IV.11.The sill beam :

IV.11.1Definition :

Stringers are beams that connect columns at infrastructure level.

calculated as a member subjected to a moment from the base of the column and a tensile force.

IV.11.2.Sizing of the sill beam :

According to RPA 99 (art.10-1-1), the minimum dimensions of the cross-section of the

are :

- (25×30) cm² .... S2, S3 sites

- $(30\times30)\ cm^2$ .... S4 sites

we take a section of $(25\times30)\ cm^2$.

IV.11.2.1.Reinforcement of the sill beam :

The reinforcement section is given by the minimum reinforcements $A=0,6\%\times b\times h$

$A= 0,006x25\times30 = 4.5cm^2$

Le choix : 6HA12 ($A=6,79cm^2$)

IV.11.2.2.Non-fragility condition :

$As\geq 0.23\times b\times d\times (ft/fe)$

$As \geq 0.23\times25\times27\times(2.1/400)=0.81cm2$

Condition checked

Transverse reinforcement

Constructive layout

Spacing

$St < \min (0.9d\ ;\ 40\ cm)$.

So: $St < 27cm$. $St = 15cm$.

Calculation of the minimum section

$At \geq 0.4\times b\times St / fe$

$At \geq 0.4\times25\times15/400$

Donc on adopte $At = 0.37cm^2$ $\Longrightarrow$2HA6

IV.12.CONCLUSION :

The study demonstrates that analyzing a building's structure is a critical step in ensuring the long-term safety and sustainability of the building. Through the effective use of reference documents and careful application of engineering concepts, the team was able to design a strong and stable structure at minimal cost, reflecting high planning and execution skills.

In addition, the study highlights the importance of adhering to local and international regulations and guidelines in the design and construction processes, as this contributes to ensuring the quality of the structure and minimizing the risk of potential failures in the future.

Overall, the project's success in applying engineering concepts and reference documents reflects a deep understanding of technical and organizational knowledge, which confirms the team's ability to carry out complex and sophisticated engineering projects with high efficiency and professionalism.

CHAPTER V:

APPLICATION OF AHP

V.I. method:

V.1.Evaluation criteria:

By analogy to the literature review, and for a sustainable assessment and suitable solutions in this context the criteria of the current research are chosen and clustered under three main dimensions respectively Economic factors, factors related to the environmental Impact of structures technical factors .

V.1.1. Economy :

V.1.2. Construction cost:

Based on design and calculation BOQ for each alternative have been prepared (appendix 2) total cost of both alternatives was calculated

Construction cost refers to the expenses incurred from the beginning of the building process until the project is fully completed **[39].**

V.1.3. Maintenance cost :

Life cycle cost (LCC) reflects all costs over the life of a building, including construction and maintenance costs

The maintenance cost encompasses expenses related to inspection, repair, and replacement. Consequently, the properties of building materials play a crucial role in minimizing these maintenance costs **[39].** the maintenance cost were scored directly by experts .

V.1.4. Construction duration :

The fast execution of construction projects can help the rapid growth of the economy . Efficient project management will be crucial to meet the project time lines **[39].**

Construction duration have been estimated by both Project Owner, and the design firm

Concrete structure 12 months / steel Structure 8 months Efficient project management will be crucial to meet these timeline **[39].**

V.2. Sustainability and Environmental impact:

V.2.1.Energy consumption :

In terms of total energy consumption, steel and concrete (as primary building materials)

have equivalent energy requirements in the pre-use phase.

V.2.2.CO2 emissions :

emission, in particular, at the forefront of environmental policy issues. The facts are that

the building industry is the largest contributor to the total upstream CO_2 emissions.

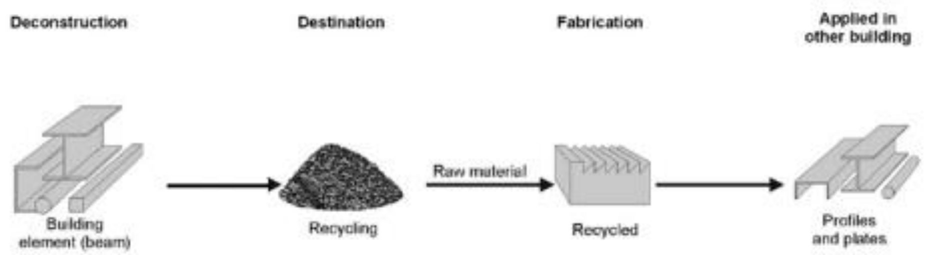
Table V. 1: Co_2 emission and Energy consumption by both structures **(no need).**

	$C0_2$ Emissions (Kg/SF)		Energy Consumption (MJ/SF)	
	Steel	Concrete	Steel	Concrete
Karimizadeh 2015	14.4	16.4	102.1	102.5
Bjorklund et al., 1996	8.1	11.9	84.7	110.6
Guggemos et al., 2005	57.6	51.1	882.6	771.1
Oladazimi et al 2020	3,56	5,75		

V.2.3.Recyclability:

Recyclability assesses the potential of a material to serve as a resource in the fabrication of novel products. Steel is recognized as a commonly recycled material, the average recycled content across the United States steel industry is 96% (Steel Recycling Institute, 2005) **[39].**

Karimizedah (2015) considered Steel structure as high recyclable structure than RC structure, Zurut el al (2022) **[39].** in a pairwise comparison regarding the recycling opportunities of steel structure and RC structure had considered the steel structure highly recyclable with a value of 5 compared to steel structure. In our study we opt for values of 3 for steel structure which means that the steel Structure is three times better than RC **[39].**



FigV. 1 : Possible destinations for steel at the end of building life cycle **[39].**

V.3.Technical :

Defined as the ability of a structure or its components to perform their intended functions within the given operational context over a long period, without facing unexpected costs for maintenance or repair ,Stiffness is assessed based on expert evaluations and their judgments.

The lifespan of concrete and steel can vary considerably depending on factors such as environmental conditions, maintenance practices, and the specific application.

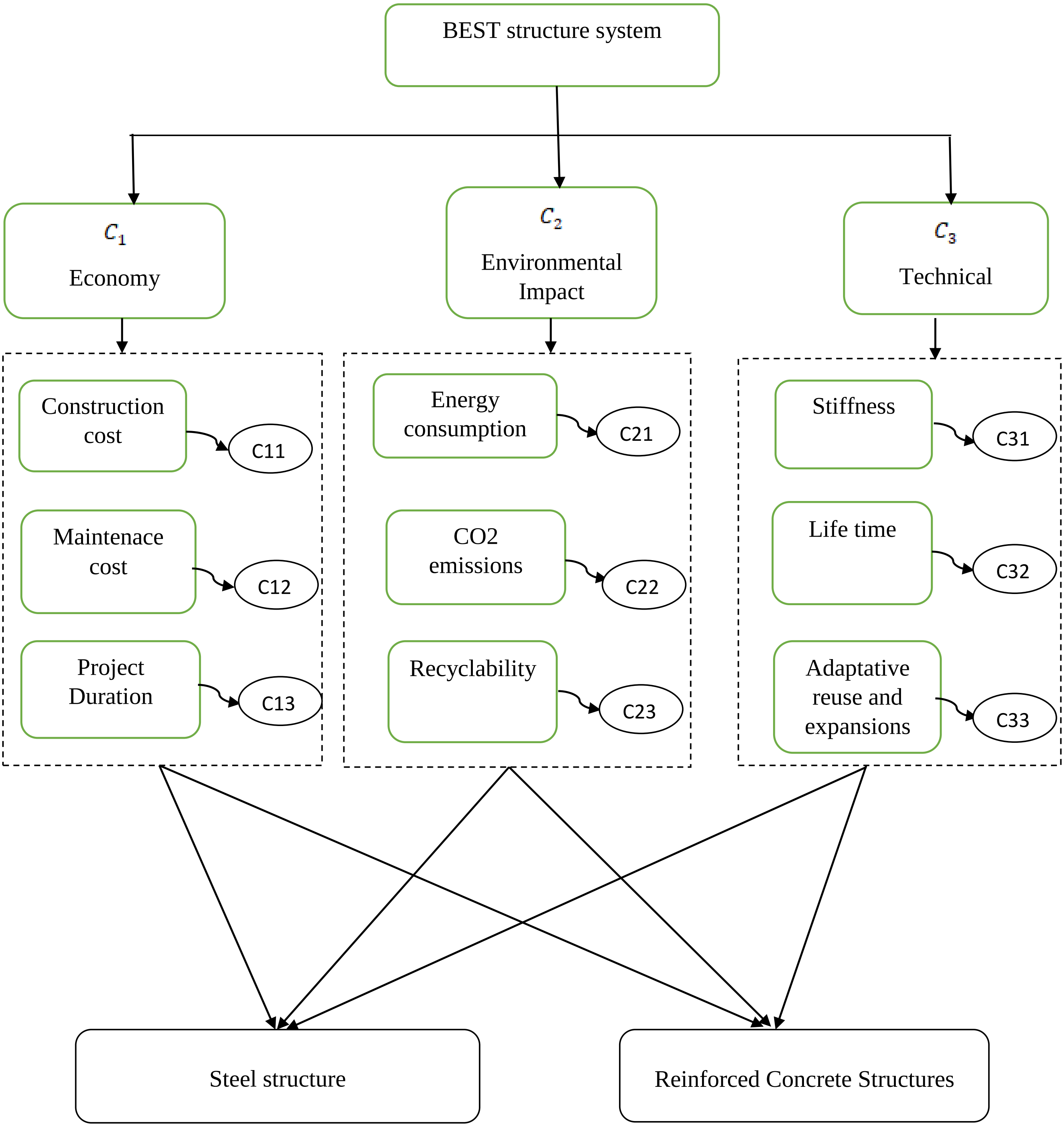
generally has a of RC 50 Steel 80 .

V.3.1. Adaptative and expansions:

The ability to adapt is gaining increasing importance in the field of sustainable design, as it is understood that user needs and technology change over time, necessitating flexible building designs. Additionally, creating a framework for recycling materials used in construction enhances the future users' ability to reuse these materials. This contributes to sustainability and reduces waste, supporting the circular economy in the built environment. By focusing on adaptability and material reuse, we can prepare our buildings and cities to face future challenges.

Table V. 2: Table code **(no need).**

Criteria	Code	Sub Criteria	Code
Economy	C_1	Construction cost	C_{11}
		Maintenance cost	C_{12}
		Project Duration	C_{13}
Environmental Impact	C_2	Energy consumption	C_{21}
		CO_2 emissions	C_{22}
		Recyclability	C_{23}
Technical	C_3	Stiffness	C_{31}
		Life time	C_{32}
		Adaptative reuse and expansions	C_{33}



FigV. 2 : Decision Hierarchy of the problem **(no need).**

V.II.Data collection:

V.1.Documentation and previous studies:

-The steel structure documents; design and calculations (appendix 1)

-Previous studies data such as environmental data and technical criteria

V.1.1.Design and calculation:

using autodesk **Robot** Structural Analysis -14- presented in details in **chapter 4** to determine aspects as elements dimensions and stiffness and some of the economics aspects of the project regarding the reinforced concrete alternative .

V.2.Questionnaire :

V.2.1.Questionnaire :

Based on the selected evaluation criteria and the proposed structural systems two questionnaire forms had been addressed to our experts

- Firstly, a pairwise comparison using Saaty's nine-point significance scale was employed to assess the importance of each criterion relative to the others.

Table V. 2: Scores for the importance of variable **(no need).**

Importance Scale	Definition of Importance Scale
1	Equally Important Preferred
2	Equally to Moderately Important Preferred
3	Moderately Important Preferred
4	Moderately to Strongly Important Preferred
5	Strongly Important Preferred
6	Strongly to Very Strongly Important Preferred
7	Very Strongly Important Preferred
8	Very Strongly to Extremely Important Preferred
9	Extremely Important Preferred

Table V. 3: Questionnaire used **(no need).**

Economy																			
1	Construction cost	9	8	7	6	5	4	3	2	1	2	3	4	5	6	7	8	9	Maintenace cost
2	Construction cost	9	8	7	6	5	4	3	2	1	2	3	4	5	6	7	8	9	Project Duration
3	Maintenace cost	9	8	7	6	5	4	3	2	1	2	3	4	5	6	7	8	9	Project Duration
Environmental Impact																			
1	Energy consumption	9	8	7	6	5	4	3	2	1	2	3	4	5	6	7	8	9	CO_2 emissions
2	Energy consumption	9	8	7	6	5	4	3	2	1	2	3	4	5	6	7	8	9	Recyclability
3	CO_2 emissions	9	8	7	6	5	4	3	2	1	2	3	4	5	6	7	8	9	Recyclability
Technical																			
1	Stiffness	9	8	7	6	5	4	3	2	1	2	3	4	5	6	7	8	9	Life time
2	Stiffness	9	8	7	6	5	4	3	2	1	2	3	4	5	6	7	8	9	Adaptative reuse and expansions
3	Life time	9	8	7	6	5	4	3	2	1	2	3	4	5	6	7	8	9	Adaptative reuse and expansions

- Secondly, experts were asked to evaluate each of the two alternatives with respect to the established evaluation criteria

Table V. 4: Basic data for structural system Alternatives Experts **(no need).**

Alternatives	C_{11}(DA)	C_{12}	C_{13} (months)	C_{21}	C_{22}	C_{23}	C_{31}	C_{32}	C_{33}
RC Structure	19330000	Points	12	1	1	1	Scoring	50	Scoring
Steel Structure	25244000	Scoring	8	1	1	3	Scoring	80	Scoring

V.2.2.Our groups of experts were divided as follow :

Expert 1, expert 2, and expert 3 are directly involved in the design and implementation process

- Expert 1: The project owner; presented by the manager and vice manager of the bitumen centre NAFTAL Ghardaïa
- Expert 2: Control services CTC; the point of view of two civil engineers in the control service CTC
- Expert 3: The designer; civil engineer with more than 15 years of experience in the field
- Expert 4: External evaluators; which are an expert in civil engineering with more than 25 years as practiser in the domain, as well as an associate Professor at civil engineering department at the university of Ghardaïa specialised in structure systems

V.3.AHP:

AHP Is one of most commonly used technique of DM in several domain among other civil engineering projects it has been ranked as the first technique of DM Between 1995 to 2015. In this study we opted for the use of AHP to select the best structural system for a multipurpose hangar.

AHP is popular because it allows for intuitive pairwise comparisons of factors and attributes, making the division of decision-making problems appear straightforward. Users of AHP start by breaking down their decision problem into a hierarchy of smaller, more manageable sub-problems, each of which can be analysed separately

Pairwise matrix is a mathematical tool used to represent relationships or comparisons between two or more sets of objects. It can be used in many areas such as decision making, data analysis, experimental design, and more. **(no need)**

Pairwise matrix :

$$A = \begin{bmatrix} a_{11} & a_{12} & \dots & a_{1n} \\ a_{21} & a_{22} & \dots & a_{2n} \\ \dots & \dots & & \dots \\ a_{n1} & a_{n2} & \dots & a_{nn} \end{bmatrix} = \begin{bmatrix} 1 & a_{12} & \dots & a_{1n} \\ 1/a_{21} & 1 & \dots & a_{2n} \\ \dots & \dots & & \dots \\ 1/a_{n1} & \dots & 1 & 1 \end{bmatrix}$$

Table V. 5: Pairwise matrix based on experts' feedback **(no need).**

E1		C1	C2	C3		C11	C12	C13
	C1	1	5	4	C11	1	4	0,5
	C2		1	0,33	C12		1	0,33
	C3			1	C13			1
		CR = 0,08				CR = 0,02		
		C21	C22	C23		C31	C32	C33
	C21	1	2	2	C31	1	4	2
	C22		1	3	C32		1	1
	C23			1	C33			1
		CR = 0,01				CR = 0,05		
E2		C1	C2	C3		C11	C12	C13
	C1	1	1	0,25	C11	1	4	0,5
	C2		1	0,25	C12		1	0,33
	C3			1	C13			1
		CR = 0				CR = 0,1		
		C21	C22	C23		C31	C32	C33
	C21	1	1	2	C31	1	1	2
	C22		1	2	C32		1	1
	C23			1	C33			1
		CR= 0				CR= 0,05		
E3		C1	C2	C3		C11	C12	C13
	C1	1	1	0,33	C11	1	5	2
	C2		1	0,5	C12		1	0,25
	C3			1	C13			1
		CR= 0,02				CR = 0,02		
		C21	C22	C23		C31	C32	C33
	C21	1	1	0,33	C31	1	3	2
	C22		1	0,5	C32		1	4
	C23				C33			1
		CR= 0,02				CR = 0,35		
E4		C1	C2	C3		C11	C12	C13
	C1	1	1	0,5	C11	1	3	2
	C2		1	0,5	C12		1	0,5
	C3			1	C13			1
		CR = 0				CR = 0,01		
		C21	C22	C23		C31	C32	C33
	C21	1	0,5	0,33	C31	1	1	1
	C22		1	2	C32		1	1
	C23				C33			1
		CR = 0,13				CR = 0		

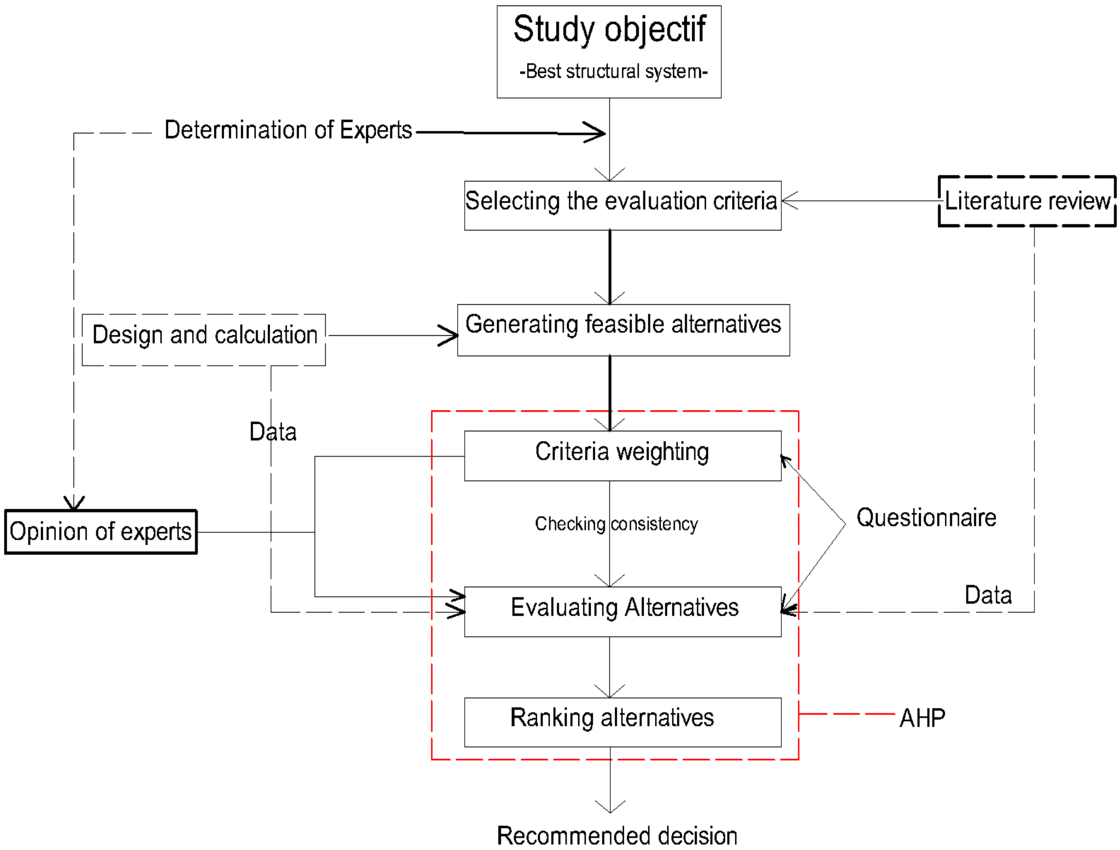
"Verification of consistency formula" refers to a method used to ensure that the judgments or comparisons made in a pairwise matrix are logically consistent. This is crucial when using methods like the Analytic Hierarchy Process (AHP) to prioritize or make decisions based on multiple criteria

"Criteria weight using Geometric mean" refers to a method of calculating the weights of criteria in a pairwise comparison matrix using the geometric mean. This method is commonly used in decision making processes such as Analytic Hierarchy Modeling (AHP) .

Alternatives Ranking Formula

$$CI = \frac{(I_{max} - n)}{(n - 1)}$$

$$CR = {CI}/{RI}$$



FigV. 3: Flowchart of the AHP Method.

V.III.Results and discussion:

V.1.Criteria weghts :

Table V. 6: example of pairwise

	Economy	Environnemental Impact	Technical factors	Geometric mean	Normelized weights
Economy	1	5	4	2,71	0,674
Environnemental Impact	0,2	1	0,33	0,4	0,101
Technical factors	0,25	3	1	0,91	0,226
			Total	4,02	1.81

Table V. 7: Criteria weights **.**

	Weight of Criteria				Weight of Sub- Criteria								
					Local Weight					Global Weight			
criteria/ experts	E1	E2	E3	E4		E1	E2	E3	E4	E1	E2	E3	E4
C1	**0,674**	0,167	0,21	0,25	C11	**0,655**	0,359	**0,57**	**0,540**	**0,441**	0,060	0,120	0,135
					C12	0,095	0,124	0,097	0,163	0,064	0,021	0,020	0,041
					C13	0,25	**0,517**	0,333	0,297	0,169	0,086	0,070	0,074
C2	0,101	0,167	0,24	0,25	C21	0,297	0,4	0,21	0,168	0,030	0,067	0,050	0,042
					C22	0,163	0,4	0,24	**0,484**	0,016	0,067	0,058	0,121
					C23	**0,54**	0,2	**0,55**	0,349	0,055	0,033	0,132	0,087
C3	0,226	**0,667**	**0,55**	**0,5**	C31	0,149	**0,413**	**0,532**	0,333	0,034	**0,275**	**0,293**	**0,167**
					C32	**0,474**	0,327	0,322	0,333	0,107	0,218	0,177	**0,167**
					C33	0,376	0,26	0,146	0,333	0,085	0,173	0,080	**0,167**
Total	1	1	1			/	/	/	/	1	1	1	1

The results presented in Table 8 show a contrast in the weights of the main criteria between the opinion of the project owner (E1) on one side and that of the CTC, the designer, and the external experts on the other side. A weight of [0.674] for Economy is considered the most important criterion among the main criteria by the project owner, whereas for the CTC experts, the designer, and the external experts, the technical factors were considered the most important criteria, with weights of [0.667], [0.55], and [0.5], respectively. The results also show that the project owner does not place significant importance on the environmental impact of the structure[0.101], whereas the other experts consider it of medium importante

On the global weight the economic criterion (0.441) remained the most important criterion according to the project owner, while the technical factors remained the most important factors according to the CTC experts, the designer and external experts (0.275), (0.293) and (0.167) The results also show that no importance was given to the environmental damage caused by the structure **(no need).**

V.2.Priority and alternatives ranking by each expert:

* Evaluation of project owner:

Table V. 8: Priority matrix depending on the evaluation of the project owner**.**

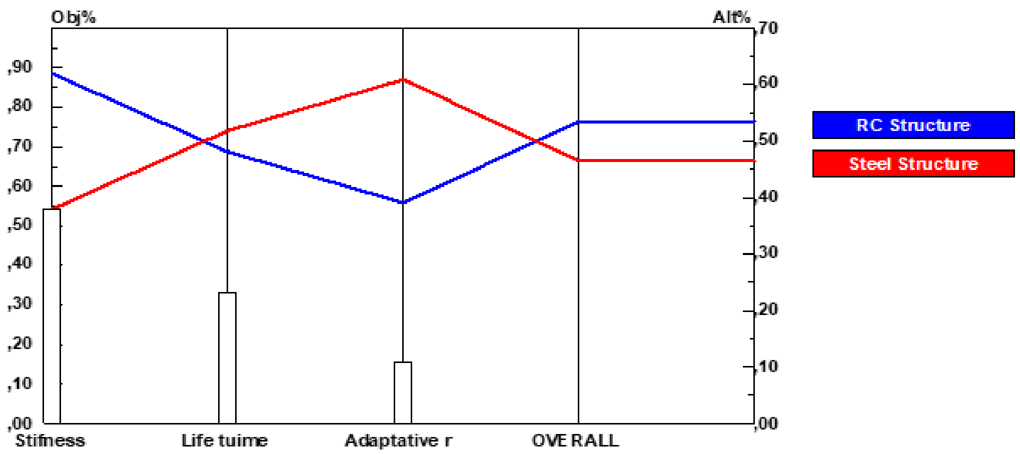
	economy			Priority	Environmental Impact			Priority	Technical Factors			Priority	Overal Priority
Ci w		**0,674**			0,101				0,226				
	C11	C12	C13		C21	C22	C23		C31	C32	C33		
Ci w	0,655	0,095	0,25		0,297	0,163	0,54		0,149	0,474	0,376		
A 1 RC	0,567	0,667	0,4	**0,533**	0,5	0,5	0,25	0,39	0,667	0,385	0,333	0,407	0,494
A 2 Steel	0,433	0,333	0,6	0,467	0,5	0,5	0,75	**0,61**	0,333	0,615	0,667	**0,593**	**0,506**

Based on the evaluation results, the project owner appears to have a clear preference for concrete structures when considering economic factors, as indicated by a higher rating (RC=0.533 for concrete versus RC=0.467 for steel). This suggests that concrete structures may offer better cost-effectiveness or lower overall expenditure in the context of this project.

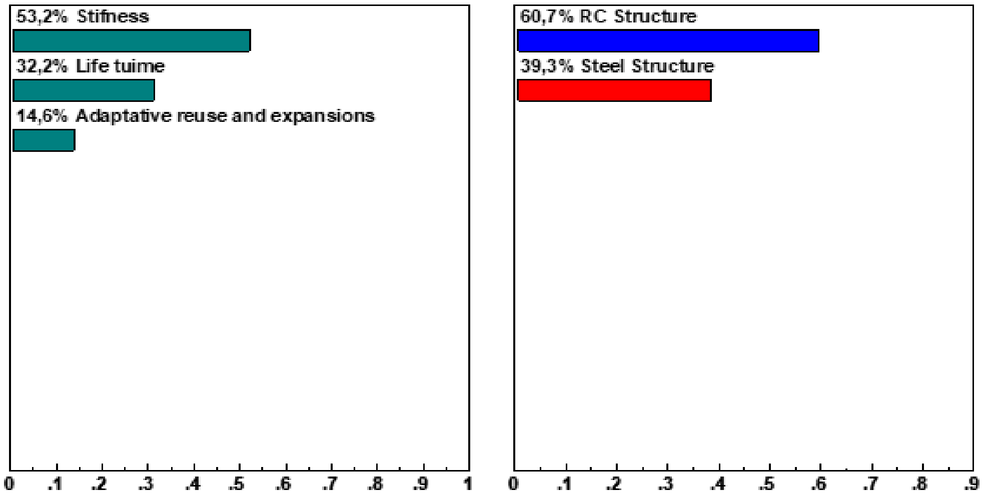
However, when evaluating environmental and technical criteria, metal structures were favoured. In particular, metal structures had a slight advantage in both areas, with ratings of RC=0.506 for steel compared to RC=0.494 for concrete. This suggests that from an environmental perspective, metal structures may have advantages such as lower carbon emissions during production or better recyclability **(no need).**

In short, the assessment highlights a balance: While concrete structures are more economically viable, metal structures excel in environmental and technical aspects. The final decision may depend on the criteria ranked by the project owner, as well as the specific context and objectives of the project.

⁎ Evaluation of CTC:



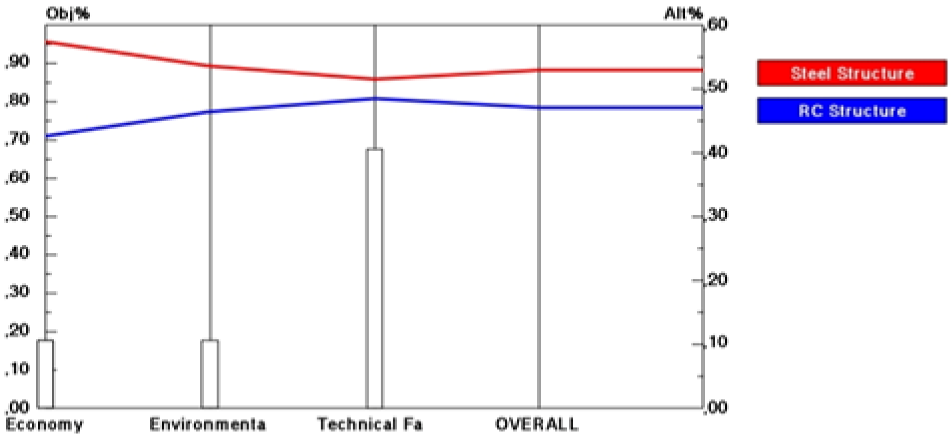
FigV. 4: Project Management performance sensitivity.



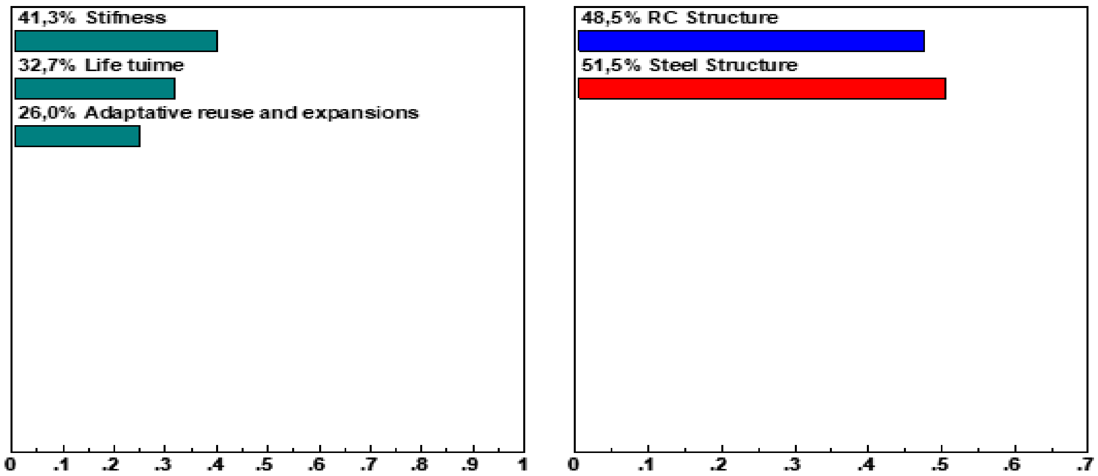
FigV. 5: CTC assassement of Systems with respect to main criterion - Technical factors

We can see from the results of the curve that concrete structures are better than steel structures, and this is the view of the CTC, where concrete structures are 0.533 and steel structures are 0.467, and for the most important criterion, concrete structures are better with 60.7% and steel structures with 39.3%, and we notice that there is no contradiction and in all cases, concrete structures are considered better than steel structures, and this is due to their handling of concrete structures **(no need).**

⁎ **Evaluation of Designer:**



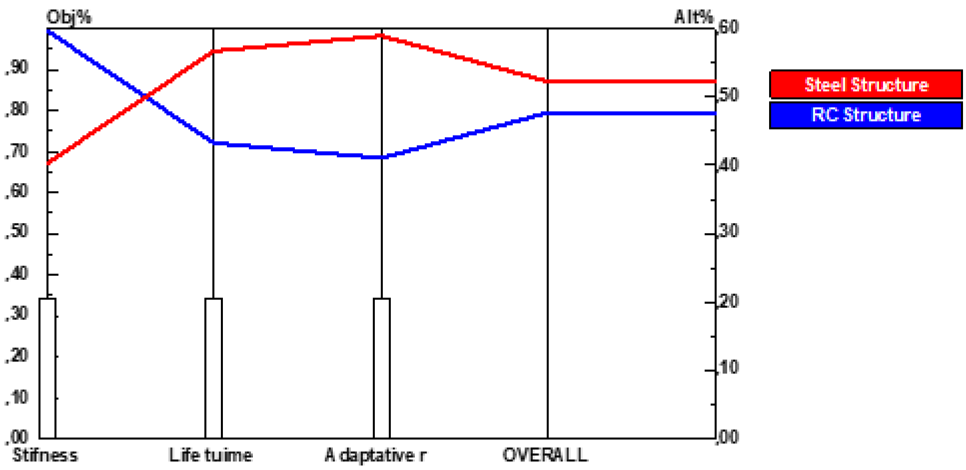
FigV. 6: performance sensitivity of alternatives -Designer



FigV. 7: Designer assassement of the structural systems with respect to main criterion - Technical Factors

After evaluation, we can see that the proportion of steel structures is 48.5% greater than that of concrete structures (51.5%), which is what the designer chose, and he believes that steel structures are better than concrete structures **(no need).**

⁎ **Evaluation of External Experts:**



FigV. 8: External experts sensitivity performance.

We note that external experts consider steel structures 0.593 to be better than concrete structures 0.407 due to the ease of implementation and speed of construction.

V.3.Global priority and alternatives ranking:

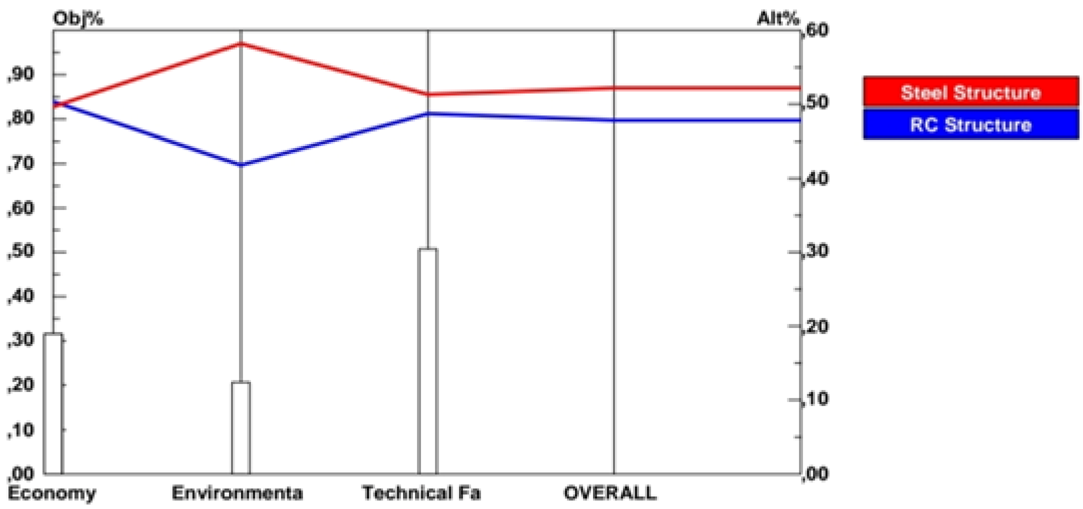
Table V. 10: Combined preference value and alternatives ranking.

Alternatives	E1		E2		E3		E4		combined	
	Priority	Ranking	Priority	Ranking	Priority	Ranking	Priority	Ranking	Priority	Ranking
RC Structure	0,494	2	0,471	2	0,535	**1**	0,477	2	0,478	2
Steel Structure	0,506	**1**	0,529	**1**	0,465	2	0,523	**1**	0,522	**1**

The results indicate a clear consensus between the project owner, designer and external experts, who all rank steel structures as the first choice, followed by concrete structures. This suggests a strong belief in the benefits of steel, perhaps due to its advantages in terms of flexibility, weight and speed of construction.

However, (CTC) offers a contrasting view, ranking concrete structures first. This gap may stem from a lack of designers specialising in steel structures, which may lead to concrete being favoured due to familiarity or the perception that it is easier to design and build

(no need).



FigV. 9: Combined alternatives ranking

After collecting the opinions, we noticed that the common opinion among them is that steel structures are better than concrete structures, and this reflects the project owner's choice, which was successful even though it is a standard that does not apply to the chosen structure.

General conclusion:

This study focused on evaluating the efficiency of structural system selection through a comparison between two systems namely; concrete structures and steel structures. The comparison covered several interrelated criteria using Multi-Criteria Decision-Making (MCDM) methods, with emphasis on economic, environmental, and technical factors. The Analytical Hierarchy Process (AHP) was employed to assess the different options, and the methodology was applied to a multi-service hangar owned by Naftal in Ghardaïa.

Initially, design documents and calculations for the hangar were collected, and we redesigned it using a reinforced concrete structural system. We also utilized Autodesk Robot software for analyzing and designing the new structure.

The results highlighted the importance of evaluating all criteria in a balanced manner. However, it was observed that Experts sometimes select a system that doesn't align with the criterion they deemed most important. In other words, there is a contradiction between what they considered the key factor and the system they ultimately chose. Furthermore, experts tend to focus solely on cost and technical characteristics, which was evident when the project owner assessed the criteria giving the high weight for economy, overlooking the environmental factors that play a crucial role in achieving sustainability. This trend could lead to unsustainable choices that negatively impact the environment and society.

Therefore, it is essential to guide investors and project owners toward making informed decisions that consider all criteria without bias. A comprehensive evaluation of all criteria contributes to improving sustainability and ensures a balance between economic, environmental, and technical factors, thereby supporting the development of more sustainable projects in the future.

other Multi-Criteria Decision-Making (MCDM) methods such as TOPSIS and ELECTRE can also be used. Each of these methods has its own advantages and can provide different insights into the available options.

The comparison between steel and concrete in construction depends on the specific requirements of the project, as both materials have distinct advantages and disadvantages. - *Steel*is known for its high strength, flexibility, speed of construction and ability to span large distances with minimal materials. It is ideal for structures that require precision, such as skyscrapers and bridges. However, steel can be more expensive, requires corrosion protection, and has less fire resistance than concrete.

List of references

1. Cheyma, B., *ETUDE D'UN BATIMENT R+ 4 A USAGE D'HABITATION CONTREVENTEMENT MIXTE (PORTIQUE+ VOILE)*. 2020, Faculté des Sciences et Technologies.
2. *Ghoneim MA, El-Mihilmy MT. Design of reinforced concrete structures2008.*
3. *Hadjar, K. and L. Chaima, Etude d'un hangar réalisé en Charpente métallique. 2020, Abdelhafid boussouf university Centre mila.*
4. *Jamle S. Flat Slab Shear Wall Interaction for Multistoried Building under Seismic Forces. IJournals: International Journal of Software & Hardware Research in Engineering. 2017.*
5. *Aldabagh S, Abed F, Yehia S. Effect of types of concrete on flexural behavior of beams reinforced with high-strength steel bars. ACI Structural Journal. 2018.*
6. *Taucer F, Spacone E, Filippou FC. A fiber beam-column element for seismic response analysis of reinforced concrete structures: Earthquake Engineering Research Center, College of Engineering, University …; 1991.*
7. *Varghese PC. Design of reinforced concrete foundations: PHI Learning Pvt. Ltd.; 2009.*
8. Bertero VV, Aktan AE, Harris HG, Chowdhury AA. Mechanical characteristics of materials used in a 1/5 scale model of a 7-story reinforced concrete test structure: Earthquake Engineering Research Center, College of Engineering, University …; 1983.
9. *Fabrication, C.C.P.B.W.o.S.S.*
10. *Landowski, M. and B. Lemoine, Concevoir et construire en acier.*
11. *Ozbasaran, H., A Parametric Study on Lateral Torsional Buckling of European IPN and IPE Cantilevers. 2014.*
12. https://www.phd.eng.br/wp-content/uploads/2015/12/en.1993.1.1.2005.pdf
13. *Bounouara, L. and S. Dendani, Etude d'un hangar en charpente métallique en décrochement avec mezzanine. 2021, Université Mouloud Mammeri Tizi-Ouzou.*
14. *Abu·Saba, E.G., DESIGN OF STEEL STRUCTURES.*
15. *Moussi, H., Etude d'un hangar en charpente métallique à usage de stockage. 2021, Université Mouloud Mammeri Tizi-Ouzou.*
16. *Structural Steelwork: Design to Limit State Theory. Third edition ed.*
17. *Multi-Criteria Decision Making Methods: A Comparative Study2000.*
18. *Aruldoss M, Lakshmi TM, Venkatesan VP. A survey on multi criteria decision making methods and its applications. American Journal of Information Systems. 2013.*
19. *Bhole GP, Deshmukh T. Multi-criteria decision making (MCDM) methods and its applications. International Journal for Research in Applied Science & Engineering Technology (IJRASET). 2018.*
20. *Asadabadi MR, Chang E, Saberi M. Are MCDM methods useful? A critical review of analytic hierarchy process (AHP) and analytic network process (ANP). Cogent Engineering.*
21. *AHPDevice.jpg/credit.*
22. *Uzun B, Taiwo M, Syidanova A, Uzun Ozsahin D. The technique for order of preference by similarity to ideal solution (TOPSIS). Application of multi-criteria decision analysis in environmental and civil engineering. .*
23. *Beg I, Rashid T. Group decision making using intuitionistic hesitant fuzzy sets. International Journal of Fuzzy Logic and Intelligent Systems.*
24. *Emamat MSMM, Mota CMdM, Mehregan MR, Sadeghi Moghadam MR, Nemery P. Using ELECTRE-TRI and FlowSort methods in a stock portfolio selection context. Financial Innovation. 2022.*
25. *Starfield T. Simple multi-attribute ranking technique smart. Decis Anal. 2005.*
26. *Deshmukh SC. Preference ranking organization method of enrichment evaluation (promethee). International Journal of Engineering Science Invention. 2013.*
27. *Rizka A. METODE MULTI-ATTRIBUTE UTILITY THEORY (MAUT) UNTUK PEMILIHAN PRODUK TERLARIS. Penerbit Tahta Media. .*

28. Antucheviciene, J., et al., *Solving civil engineering problems by means of fuzzy and stochastic MCDM methods: current state and future research.* Mathematical problems in engineering, 2015. **2015**(1): p. 362579.
29. Zumrut, I.B., H.B. Baran, and T.G. Ozbalta, *Multi-criteria decision-making approach for selecting a structural system of an industrial facility.* Organization, Technology and Management in Construction: an International Journal, 2022. **14**(1): p. 2656-2665.
30. Haddad, M. and D. Sanders, *Selection of discrete multiple criteria decision making methods in the presence of risk and uncertainty.* Operations Research Perspectives, 2018. **5**: p. 357-370.
31. ZEHRO, K.Z., *COMPARATIVE CONSTRUCTION COST FOR STEEL STRUCTURES AND REINFORCED CONCRETE STRUCTURES WITH MOMENT-RESISTING FRAME (MRF)*. 2021, NEAR EAST UNIVERSITY.
32. Johnson, T.W., *Comparison of environmental impacts of steel and concrete as building materials using the life cycle assessment method*. 2006, Massachusetts Institute of Technology.
33. Saba, D.D.W., *Investigating the durability of structures*. 2013, Massachusetts Institute of Technology.
34. Azzi, A. and M. Ait Taleb, *Etude d'un bâtiment (R+ 9+ attique) A usage d'habitation et commercial en contreventement mixte*. 2018, Université Mouloud Mammeri TiziOuzou.
35. Khaled, B.S., *Etude d'un bâtiment R+ 8 contreventement mixte voile et portique*. 2019, Faculté des Sciences et Technologies.
36. OUEDRAOGO, A.A.T., et al., *Comparative numerical and experimental studies of the tensile strengths of concrete incorporating recycled and natural aggregates.* World Journal of Advanced Research and Reviews, 2023. **20**(3): p. 1535-1549.
37. Labed, A., T. Benmansour, and A.M. Abu halaweh, *A numerical investigation of the inelastic cyclic behaviour of short and long links designed according to RPA 99 provisions.* Asian Journal of Civil Engineering, 2020. **21**(2): p. 217-228.
38. Madi, R., A. Bordjiba, and M. Guenfoud. *Compliance with RPA of an Old Building*. in *Proceedings of the 4th International Symposium on Materials and Sustainable Development: Volume 2: Waste Recycling and Environment 4*. 2020. Springer.
39. Karimizadeh, A., *Comparison of Steel and Reinforced Concrete as a Sustainable Building Material in Northern Cyprus*. 2015, Eastern Mediterranean University (EMU)-Doğu Akdeniz Üniversitesi (DAÜ).

- **RPA99version 2003:** Algerian seismic regulations.
- **CBA93:** design and calculation rules for reinforced concrete structures.
- **BAEL91:** reinforced concrete in limit states.
- **DTR B.C.2.2:** technical regulations (loads and overloads).

Appendices

Appendix 1

STEEL STRUCTURE DESIGN AND CALCULATION

Presentation of the work. :

Introduction :

This project is composed of a single R+1 block in metal frame with a mezzanine in collaborating floor at +3.00 m, located in the commune of BOUNOURA wilaya of GHARDAÏA which is a Saharan region classified in ZONE Ozone of negligible seismicity according to the R.P.A 99 version 2003, and on firm ground.

The purpose of this calculation note is to calculate and design the elements of the project structure.

The study of the elements is a study, carried out using Autodesk Robot Structural Analysis 2011 software.

CHARACTERISTICS OF THE MATERIALS USED

BUILDING DIMENSIONS:

Height:	7.25m
Width:	18.00 m
Depth:	24.00 m
Ground position:	0.00m
Building altitude:	7.30 m

CONCRETE: Concrete is a mixture of aggregates (sand, gravel), hydraulic binders (cement), water and additives. The resulting mixture is called "fresh concrete", and begins to harden after a few hours, gradually reaching its characteristic strength.

Cement dosage:	Concrete of cleanliness	150.00 Kg/m³
	Large concrete	250.00 Kg/m³
	Reinforced concrete	350.00 Kg/m³
Admissible stresses:	Controlled concrete	F_{c28} =22.00 MPa

a) **Ultimate Limit State :** The ultimate limit states concern safety, such as resistance to loads, stability and equilibrium, when the structure reaches the point where it becomes dangerous for its intended use.

ELU: F_{bu}= 11.33MPa

b) **Serviceability limit state:** Serviceability limit states refer to states in which the structure, although "functional", begins to behave in an unsatisfactory manner due, for example, to vibrations or excessive deformations or deflections.

Design or verification at ELS is based on limiting deformations (or deflections for beam spans, and horizontal displacements for column heads).

ELS: σb = 15.00 MPa

F_{tj} = 0.6+0.06 × fc28 = 02.10 MPa

Density of concrete = 2.50 t/m³ = 25KN/m³

NB: The concrete of the infrastructure must be made of sulfate-resistant cement (HTS OR CRS).

STEEL:

Standard Fe E 400 Natural high adhesion: fe = 400.00 MPa

Slightly harmful cracking:

Durable and transitional situation σs = fe/1.15 MPa

Accidental situation: σs = fe / 1.00 MPa

Detrimental cracking: σs = min (2/3 fe , 150.00 η)

With η = 1.6 and σs = 240.00 MPa

LOWERING LOADS:

The aim of the load reduction is to determine for each supporting element, the load which falls on it at the level of each floor up to the foundation. DTR BC 2-2

COLLABORATIVE FLOOR:

- Floor tile……………………………………………65.00 Kg/ m²

-Full slab………………………………………………340.00 Kg/ m²

G = 405.00 Kg/m² **Q =250.00 Kg/m²**

1.35G +1.50 ≈ 9.22 KN/m²

CALCULATIONS OF SNOW AND WIND LOADS according to RNV2013

WIND DATA:

Region: 1

Wind type: normal

Site: normal ks= 1,000

Basic pressure: 50.99 KG/ m²

WIND RESULTS:

Load case: Wind 0 deg sur.(+)

Gamma: 1,000

Loading coefficients

area:282 Ce: 0.800 Ci: 0.300 Ce-Cl = 0.500

qh: 47.92 KG/ m² Delta:

qr(z) 23.96 KG/ m²

area:283 Ce: -0.500 Ci: 0.300 Ce-Ci = -0.800

qH: 47.92 KG/ m² Delta: 1.000

qr(z)-38.34 KG/ m²

area:284 Ce: -0.500 Ci: 0.300 Ce-Ci = -0.800

qH: 47.92 kG/ m² Delta: 1,000

qr(z)= -38.34 kG/ m²

area :285 Ce: -0.500 Ci: 0.300 Ce-Ci= -0.800

qH: 47.92 kG/ m² Delta: 1,000

gr(2)-38.34 kG/ m²

Load case: **Wind 90 deg on.(+)**

Gamma: 1,000

Loading coefficients

area:282 Ce: -0.500 Ci: 0.300 Ce-Ci = -0.800

qH: 47.92 kG/ m² Delta 1.000

qr(z)= -38.34 kG/ m²

area:283 Ce: -0.500 Ci 0.300 Ce-Ci = -0.800

qH: 47.92 kG/ m² Delta: 1.000

qr(z)= -38.34 kG/ m²

area:284 Ce: -0.500 Ci: 0.300 Ce-Ci= -0.800

qH: 47.92 kG/ m² Delta :1.000

qr(z)= -38.34 kG/ m²

area:285 Ce: 0.800 Cl: 0.300 Ce-Ci= 0.500

qH: 47.92 kG/ m² Delta: 1.000

qr (z) = 23.96 kG/ m² local

View of the structure

2 CAE 60x6

2 CAE 70x7

2 CAE 70x7 mont

2 CAE 90x9

2 CAE 90x9 sup

CV 2 CAE 60x6

HEA 100

HEA 180

HEA 200,

HEA 280

IPE 120,

UPN 200

HEA 180

HEA 200

HEA 280

UPN 200

Coordinates of the center of gravity of the structure:

X = 8.837 (m)

Y = 10.355 (m)

Z = 4.659 (m)

Central moments of inertia of the structure:

Ix = 2336549.013 (kg. m²)

ly =1439738.356 (kg.m²)

Iz =3523663.898 (kg. m²)

Mass = 32858.006 (kg)

List of load cases/calculation types:

Case 1 : PP

Analysis type: Linear static

Case 2 : G

Analysis Type: Linear Static

Case 3 : Q

Analysis Type: Linear Static

Case 4 : Wind 0 deg on.(+)

Type of analysis: Linear static

Case 5 : Wind 90 deg on.(+)

Type of analysis: Linear static

Case 6 : ULS V90

Type of analysis: Linear combination

Case 7 : ELU V0

Type of analysis: Linear combination

Case 8 : ELS V90

Analysis type: Linear combination

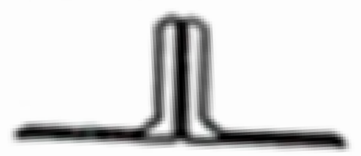
Case 9 : ELS V0

Type of analysis: Linear combination

profile properties:

Section Features:

2 CAE 90x9



HY=19.0 , HZ=9.0 [cm]

AX=31.04 [c m²]

IX=8.31 , IY=231.60 , IZ=518.46 [cm4]

Material = STEEL

2 CAE 90x9 sup

HY=19.0 , HZ=9.0 [cm]

AX=31.04 [c m²]

IX=8.31 , IY=231.60 , IZ=518.46 [cm4]

Material = STEEL

2 CAE 70x7 mount

HY=15.0 , HZ=7.0 [cm]

AX=18.79 [c m²]

IX=3.04 , IY=84.60 , IZ=199.26 [cm4]

Material = STEEL

2 CAE 70x7

HY=15.0 , HZ=7.0 [cm]

AX=18.79 [c m²]

IX=3.04 , IY=84.60 , IZ=199.26 [cm4]

Material = STEEL

HEA 280

HY=28.0 , HZ=27.0 [cm]

AX=97.26 [c m²]

IX=56.50 , IY=13673.30 , IZ=4762.64 [cm4]

Material = STEEL

IPE 120

HY=6.4 , HZ=12.0 [cm]

AX=13.21 [c m²]

IX=1.71 , IY=317.75 , IZ=27.67 [cm4]

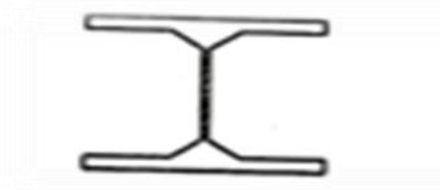
Material = STEEL

HEA 100

HY=10.0 , HZ=9.6 [cm]

AX=21.24 [c m²]

IX=4.69 , IY=349.22 , IZ=133.81 [cm4]



Material = STEEL

HEA 180

HY=18.0, HZ=17.1 [cm]

AX=45.25 [c m²]

IX=14.20 , IY=2510.29 , IZ=924.60 [cm4]

Material = STEEL E24

2 CAE 60x6

HY=12.8 , HZ = -6.0 [cm]

AX=13.82 [c m²]

IX=1.64 , IY=45.58 , IZ=105.94 [cm4]

Material = STEEL

CV 2 CAE 60x6

HY=12.8 , HZ=6.0 [cm]

AX=13.82 [c m²]

IX=1.64 , IY=45.58 , IZ=105.94 [cm4]

Material = STEEL

UPN 200

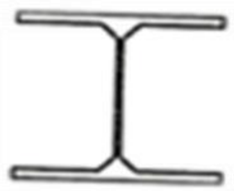
HY=7.5, HZ=20.0 [cm]

AX=32.02 [c m²]

IX=11.03 , IY=1910.50 , IZ=147.81 [cm4]

Material =STEEL E24

HEA 200



HY=20.0, HZ=19.0 [cm]

AX=53.83 [c m²]

IX=18.60 , IY=3692.15 , IZ=1335.51 [cm4]

Material= STEEL E24

weights

Weightings according to the regulations: BAEL 91

Parameters for creating weights:

Type of weights: complete

List of active cases:

1: PP	permanent G1	
2:G	permanent G1	
3:Q	operating	Q1
4: Wind 0 deg on.(+)	wind	W1
5: Wind 90 deg on.(+)	wind	W1
EPS	Fire	

List of defined groups:

permed:	G1	And,
operating:	Q1	Or,
wind:	W1	or excl.,

List of defined relationships:

Characteristics – Materials

	Material	E [MPa]	G [MPa]	NU	LX [1/°C]	RO [kG/m3]	Re [MPa]
1	STEEL	210000.00	80800.00	0.30	0.00	7852.83	235.00
2	STEEL E24	210000.00	80800.00	0.30	0.00	7852.83	235.00

Loads:

Case: 1A9

Case	Load type	List			
1:PP	proper weight	All	Entire structure	-Z	Coef=1.00
2:G	(EF) uniform surface	237	PX=0.0	PY=0.0	PZ=- 6.00
3:Q	(EF) uniform surface	237	PX=0.0	PY=0.0	PZ=- 50.00
2:G	(EF) uniform surface	903	PX=0.0	PY=0.0	PZ=- 550.00
3:Q	(EF) uniform surface	903	PX=0.0	PY=0.0	PZ=- 250.00
4:Wind 0 deg on.(+)	surface on object	282	PX=0.0	PY=0.0	PZ=- 19.17
4:Wind 0 deg on.(+)	surface on object	283	PX=0.0	PY=0.0	PZ=- 30.67
4:Wind 0 deg on.(+)	surface on object	284	PX=0.0	PY=0.0	PZ=- 30.67
4:Wind 0 deg on.(+)	surface on object	285	PX=0.0	PY=0.0	PZ=- 30.67
5: Wind 90 deg on.(+)	surface on object	282	PX=0.0	PY=0.0	PZ=- 30.67
5: Wind 90 deg on.(+)	surface on object	283	PX=0.0	PY=0.0	PZ=- 30.67
5: Wind 90 deg on.(+)	surface on object	284	PX=0.0	PY=0.0	PZ=- 30.67
5: Wind 90 deg on.(+)	surface on object	285	PX=0.0	PY=0.0	PZ=- 19.17

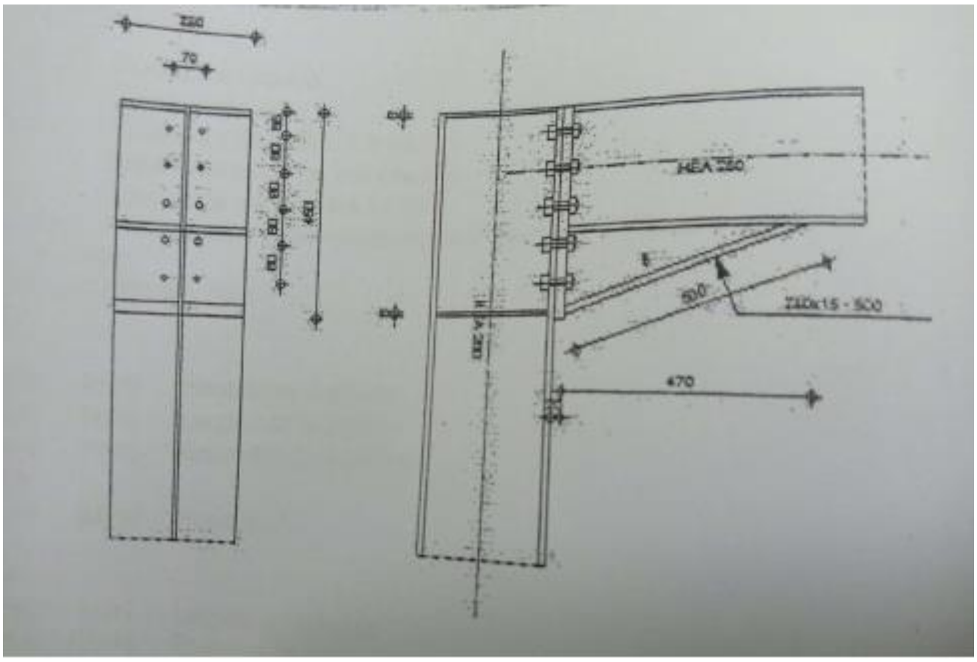
- Case: 6A9

Combination	Name	Analysis type	Nature of the combination	Nature of the case	Definition
6(C)	ELU V90	Linear combination	ELU	Permed	(1+2)×1.35+3 +1.50+5×1.80
7(C)	ELU V0	Linear combination	ELU	Permed	(1+2)×1.35+3 +1.50+5×1.80
8(C)	ELU V90	Linear combination	ELS	Permed	(1+2+3+5)×1. 00
9(C)	ELU V0	Linear combination	ELS	Permed	(1+2+3+5)×1. 00

Verificatiopn des pieces

Profil	Lay	Laz	Reason	Cas	Profil	Lay	Laz	Reason	Cas
2 CAE 90x9	39.54	29.36	0.22	6 ELU V90	2 CAE 70x7	52.57	42.82	0.13	6 ELU V90
2 CAE 90x9	39.54	29.36	0.22	6 ELU V90	2 CAE 70x7	51.46	41.91	0.10	6 ELU V90
2 CAE 90x9	39.54	29.36	0.22	6 ELU V90	2 CAE 70x7	50.42	41.07	0.07	7 ELU V0
2 CAE 90x9	39.54	29.36	0.22	6 ELU V90	2 CAE 70x7	49.46	40.29	0.04	7 ELU V0
2 CAE 90x9	39.54	29.36	0.22	6 ELU V90	2 CAE 70x7	48.59	39.58	0.06	6 ELU V90
2 CAE 90x9	39.54	29.36	0.22	7 ELU V0	2 CAE 70x7	47.82	38.95	0.14	6 ELU V90
2 CAE 90x9	39.54	29.36	0.22	6 ELU V90	HEA280	25.30	42.87	0.34	6 ELU V90

Assembly No.:



Assembly Name: Gantry Angle

Structure node: 211

Structural bars: 507, 492

Geometry:

Colenmn :

Profile: HEA 280

Bar N*; 507

$\alpha =$ -90.0[Deg] Tilt angle

h_c = 270 [mm] Height of post section

b tc = 280[mm] Width of post section

t ac = 8[mm] Thickness of the web of the column section

tc= 13[mm] Thickness of the flange of the column section

r_c = 24[mm] Fillet radius of column section

A c = 97.26[cm²] Column section area

$L_{xc} =$ 13673.30 [cm^4] Moment of inertia of the column section

Material: STEEL

σ_{ec}= 235.00 [MPa] Resistance

Beam :

Profile: HEA 280

Bar No: 492

α = 0.0 alpha [Deg] Tilt angle

h{b} = 270[mm] Height of beam section

bf = 280[mm] Width of beam section

t wb = 8[mm] Thickness of the web of the beam section

Tfb = 13[mm] Thickness of the aisle of the beam section

α = 0.0 [Deg] Tilt angle

rb = **2**4 [mm] Fillet radius of beam section

rb = 24 [mm] Fillet radius of beam section

Ab= 97.26 [cm²] Sectional area of the beam

lxb = 13673.30 [cm4] Moment of inertia of the beam

Material: STEEL

σ_{eb} = 235.00 [MPa] Resistance

Bolts:

d = 16 [mm] Bolt diameter

Class = 8.8 Bolt class

Fb = 69.08 [kN] Bolt breaking strength

Nv = 5 Number of bolt rows

Nh = 2 Number of bolt columns

h_1 = 55 [mm] First bolt clamp-upper end of end plate

Gauge,ei= 70 (mm)

Center distance pi = 80, 80, 80, 80 (mm)

Platinum:

hp = 450 [mm] Height of the plate

bp = 280 [mm] Width of the plate

tp= 20 [mm] Thickness of the plate

Material: STEEL

σ_{eb}= 235.00 [MPa] Resistance

Lower hock:

Wd= 280 [mm] Width of the plate

Tfd = 15 [mm] Aisle thickness

hd = 170 [mm] Height of the plate

twd = 8 [mm] Thickness of the core

ld = 470 [mm] Length of the plate

α = 19.9 [Deg] Tilt angle

Material: STEEL

σ_{ebu} = 235.00 [MPa] Resistance

Coulonmn stiffener:

Superior:

hsu = 244 [mm] Height of the stiffener

bsu = 136 [mm] Width of thu ralizer

Thu = 8 [mm] Thickness of the stiffener

Material: STEEL

σ_{eb} = 235.00 [MPa] Resistance

Lower:

had = 244 [mm] Height of stiffener

bsu= 136 [mm] Width of the stiffener

thd **=** 8 [mm] Thickness of the stiffener

Material: STEEL

σ_{eb}= 235.00 [MPa] Resistance

Fillet welds:

aw = 6 [mm] Core weld

af = 10 [mm] Sole weld

as= 6 [mm] Welding of the heat sink

ard = 5 [mm] Horizontal weld

Efforts: Case: 6: ELU V90 (1+2)*1.35+3*1.50+5*1.80

My = 70.69 [kN*m] Bending moment

FZ = -83.25 [kN] Shear force

Fx = 32.22 [kN] Axial force

Results:

Calculation distances:

Bolt	Type	A1	A2	A3	A4	A5	A6	A’1	A’2	A’3	A’4	A’5	A’6	S	S1	S2
1	interiors	23	31			28	42	7	31			39	47			
2	central	23	31					7	31							80
3	central	23	31					7	31							80
4	Central	23	31					7	31							80

X= 89 [mm] Compressed area

Forces per bolt - Forces per bolt - plastic method:

Ball N	Di	Ft	fa	fs	Fp	fb	fI	Pi{%}
1	369	159.80	0.00	439.64	245.08	69.08	49.79	100.00
2	289	74.43	75.20	113.95	155.60	69.08	39.70	100.00
3	209	74.43	75.20	113.95	155.60	69.08	29.60	100.00
4	129	74.43	75.20	113.95	155.60	69.08	19.51	100.00

Di -bolt arrangement

Ft - force transferred by the plate of the abutting element

Fa - transferred by the core of the abutment element

Fs -orce Fforce transferred by the weld

Fp -force transferred by the carrier's wing

Fb -force transferred by the bolt

Fi -Real requesting effort

Fis$\leq$ min(Fu, F. Fl. Ft) 49.79<69.08 checked (0.72)

Bolt Pull:

1.25 Fimax/As $\leq \sigma$red 396,431< 550.00 checked (0.72)

Simultaneous action of tensile and shear force in the bolt

Fimax2+2.36 Ti /As $\leq \sigma$red 1327,441< 550.00 verified (0.60)

T_1 = 8.32 [kN] Shear force in the bolt

Tb = 56.07 [kN] Bolt resistance to shear

Shear force

T1$\leq$Tb 8.32<56,07 checked (0.15)

Checking the beam:

Fres = 212.75 [kN] Compression force Reduced sole compression

Ncadm = 1170.10 [kN] Resistance of beam section

Fres ≤ Ncadm 212.75<1170.10 verified (0.18)

Compression of the column :

web Fress <Fpo l 212.75 < 977.60 checked (0.22)

Shearing of the column web - (C.T.I.C.M recommendation)

VR = 350.62 [kN] Shear force in the web

VR = 0.47"Av σo

Fres ≤VR 1212.751 <350/62 checked (0.61)

Satisfactory assembly with respect to the Standard:

Gusset assembly calculation:

General:

Structure node: 47

Structure bars: 9, 10, 34, 24,

Geometry: **Bars:**

		Barre 1	**Barre 2**	**Barre 3**	**Barre 4**	
Barre N		9	10	34	24	
Profile		2CAE90×9	2CAE90×9	2CAE70×7	2CAE70×7 mont	
	H	90	90	70	70	Mm
	Bf	90	90	70	70	Mm
	Tw	9	9	7	7	Mm
	Tf	9	9	7	7	Mm
	R	11	11	9	9	Mm
	A	31.04	31.04	18.79	18.79	cm²
Matières		STEEL	STEEL	STEEL	STEEL	
	σ_e	235.00	235.00	235.00	235.00	MPa
	Fu	365.00	365.00	365.00	365.00	MPa
Angle	α	0.0	0.0	36.6	90.0	Deg
Length	1	1.20	1.49	0.83	1.20	M

Welds:

Bar welds:

Bar 1-2:

a = 7 [mm] Sole side

b= 7 [mm] Angled edge

Gusset:

Lp = 600 [mm] Plate length

hp= 250 [mm] Height of the plat

tp= 8 [mm] Thickness of the plate

Settings:

H1= 0 [mm] Grugeago

V_1= 0 [mm] Notch

h_2= 0 [mm] Notch

V2= 0 [mm] Notch

h3= 0 [mm] Notch

V3 = 0 [mm] Notch

H4= 0 [mm] Notch

V4= 0 [mm] Notch

Center of gravity of the sheet relative to the center of gravity of the bars

eh= 300 [mm] Horizontal distance of the end of the gusset from the point of intersection of the axes of the bars

Material: STEEL

$\sigma =$ 235.00 [MPa] Resistance

Efforts:

Case: 6: ELU V90 (1+2)*1.35+3*1.50+5*1.80

N_1= -9.97 [kN] Axial force

N2 = 3.74 [kN] Axial force

N3 = 16.61 [kN] Axial force

N4= -9.47 [kN] Axial force

Results:

Platinum:

Left section:

Ni = 3.37 [kN] Axlal force in the bar

Mi = 0.48 [kN*m] Moment in the bar

Ai = 20.00 [cm²] Gusset section

Wi= 0.08 [cm³] Elastic factor of the section

f< σe 7,45<235.00 checked (0.03)

f< σe 11,43 <235.00 checked (0.05)

Straight section:

Nr = 3.74 [kN] Axlal force in the bar

Mr = 0.25 [kNm] Moment in the bar

Ar = 20.00 [cm²] Gusset section

Wr= 0.08 [cm]Elastic factor of the section

f< σe 4,85 < 235.00 checked (0.02)

f< σe 10.94 < 235.00 checked (0.05)

Bars:

T_1= 1086.03 [kN] Weld strength

N1,+ΔN<T1 -16.25 < 1086.03 checked (0.01)

M_1 = 729.43 [kN]Bar resistance

N1<M1 -9,97<729.43 checked (0.01)

T2 = 1086.03 [kN] Weld strength

N2+ ΔN < T2 14.46 < 1086.03 checked (0.00)

M2 = 437.51 [kN] Bar resistance

N2 < M2 3,74< 437.51 checked (0.00)

T3= 1347.23[kN] Weld strength

N3 + ΔN 20.11 < 1347.23 checked (0.01)

M3= 264.53[kN] Bar resistance

N3 < M3 16.61 <264.53 checked (0.01)

T4 = 1318.06[kN] Weld resistance

N4 + ΔN <T4 -11.54 < 1318.06 checked (0.01)

M4= 441.65 [Kn] Bar resistance

N4 < M4 - 9.471 < 441.65 checked (0.01)

Satisfactory assembly with respect to the Standard:

Gusset assembly calculation:

General:

Assembly No.: 4

Assembly Name: Truss Chord Gusset

Structure node: 46

Structure bars 23, 292, 34,

Geometry:

Bars:

		Barre 1	Barre 2	Barre 4	Barre 5	
Baare N			23	292	34	
Profiled		2CAE 90×9 sup	2 CAE 70×7 mont	2 CAE 90×9 sup	2 CAE 70×7	
	H	90	70	90	70	Mm
	Bf	90	70	90	70	Mm
	Tw	9	7	9	7	Mm
	Tf	9	7	9	7	Mm
	R	11	9	11	9	Mm
	A	31,04	18,79	31,04	18,79	Cm2
Material:		STEEL	STEEL	STEEL	STEEL	
	σe	235,00	235,00	235,00	235,00	MPa
	Fu	365,00	365,00	365,00	365,00	MPa
Angle	α	0,0	-0,0	87,1	53,4	Deg
Length	1	1,20	1,49	0,89	0,00	M

Welds:

Bar welds:

Bar 1-2:

a= 0 [mm] Sole side

b= -1 [mm] Corner edge

par 4:

a= 7 [mm]Sole side

b= 7 [mm]Angled edge

Bar 5:

a =7 [mm]Sole side

b =7 [mm]Angled edge

Gusset:

Ip= 600 [mm]Plate length

hp= 300 [mm]Height of the plate

Tp= 8 [mm]Plate thickness

Settings:

h_1 = 0 [mm]Notching

V_1= 0 [mm]Notching

h2 = 0 [mm]Notching

V2= 0 [mm]Notching

h3 = 0 [mm]Notching

V3 = 0 [mm]Notching

h4 = 0 [mm]Notching

V4= 0 [mm] Notching

Center of gravity of the sheet relative to the center of gravity of the bars (50/150)

ev= 0 [mm] Vertical distance of the end of the gusset from the point of Intersection of the axes of the bars

eh= 250 [mm] Horizontal distance of the end of the gusset from the point of Intersection of the axes of the bars

eo=0 [mm] Hor. frame axis distance.

Material: STEEL

σ=235.00 [MPa] Resistance

Efforts:

Case: 6: ELU V90 (1+2)×1.35+3×1.50+5×1.80

N1= -28.93 [kN]Axial force

N_2 = -18.91 [kN] Axial force

N3 = -28.93 [kN]Axial effort

N_1 = -28.93 [kN] Axial force

N5 = 16.79 [kN] Axial force

Results:

Platinum:

Left section:

NI = -28.93 [kN] Axlal force in the bar

MI = -2.47 [kNm] Moment in bar

AI = 24.00 [cm²] Gusset section

WI = 0.12 [cm³]Elastic factor of the section

f< σe 37.06 < 235.00 checked (0.16)

t< σe 0.00 < 235.00 checked (0.00)

Straight section:

Nr= -10.35 [kN] Axial force in the bar

Mr = -3.77 [kN*m] Moment in the bar

Ar = 24.00 [cm²] Gusset section

Wr= 0.12 [cm³]Elastic factor of the section

f< σe 40,34< 235.00 checked (0.17)

t< σe 14.83<235.00 checked (0.06)

Bars:

T_1 = -42.73 [kN] Weld resistance

[N_1 +ΔN1] <T1 -35.23 > -42.73 checked (0.00)

M_1 = 729.43 [kN] Resistance of the bar

N1<M1 -28.93 < 729.43 checked (0.00)

T2 = 1155.24 [kN] Weld resistance

[N2 +ΔN2] < T2 -23.41 < 1155.24 checked (0.02)

M2= 441.65 [kN] Resistance of the bar

N2<M2 -18.91|< 441.65 checked (0.02)

T4 = 1663.24 [kN] Weld resistance

N4 +ΔN <T4 -35.351< 1663.24 checked (0.02)

M 4= 729.43 [kN] Resistance of the bar

N4 < M4 -28,93<729.43 checked (0.02)

T5 = 1507.15 [kN] Weld strength

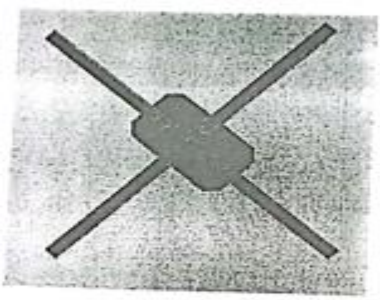
N5 +ΔN <T5 21.34 < 1507.15 checked (0.01)

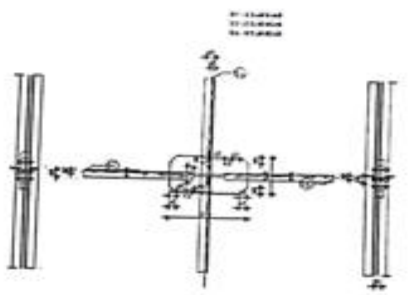
M5 = 264.53 [kN] Bar resistance

N5<M5116,791 264.53 checked (0.01)

Satisfactory assembly with respect to the Standard:

Gusset assembly calculation:





General:

Assembly N*: 5

Assembly Name: Gusset Bracing

Structure node: 323

Structure bars: 744, 737, 743, 738,

Geometry:

Bars

		Baare 1	Baare 2	Baare 3	Baare 4	
Baare		744	737	743	738	
Profile		2CAE60 ×6	2CAE60 ×6	2CAE60 ×6	2CAE60 ×6	
	h	60	60	60	60	Mm
	bf	60	60	60	60	Mm

	tw	6	6	6	6	Mm
	tf	6	6	6	6	Mm
	r	8	8	8	8	mm
	A	13.82	13.82	13.82	13.82	cm^2
Material		Steel	Steel	steel	steel	
	σe	235.00	235.00	235.00	235.00	MPa
	fu	265.00	365.00	365.00	365.00	MPa
Angle	α	0.1	90.00	0.1	90.00	Deg
Length	1	0.00	0.00	0.00	0.00	m

Lungs:

Bar 1

Class = 4.8 Bolt class

D = 16 [mm] Bolt diameter

do= 18 [mm] Bolt hole diameter

AS= 1.57 [cm²] Alre of the effective section of the bolt

Av = 2.01 [cm²] Bolt cross-sectional area

Fyb = 280.00 [MPa] Plasticity limit

Fub = 400.00 [MPa] Bolt tensile strength

n= 2 Number of bolt columns

Bolt spacing: 60 [mm]

e1= 40 [mm] Distance of the center of gravity of the first bolt from the end of the bar

e2= 30 [mm] Distance of bolt axis from bar edge

ec= 100 [mm] Distance from the end of the bar to the point of intersection of the bar axes

Bar 2-4:

Class = 4.8 Bolt class

d = 16 [mm] Bolt diameter

do= 18 [mm] Bolt hole diameter

A = 1.57 [cm²] Effective cross-sectional area of bolt

Av= 2.01 [cm²] Bolt cross-sectional area

fvo= 280.00 [MPa] Plasticity limit

Fub= 400.00 [MPa] resistance of the bolon to traction

n = 3 Bolt tensile strength Number of bolt columns

Bolt spacing :70/70 [mm]

Bar 3:

e0= 30 [mm]Distance of the bolt axis from the edge of the bar

Class = 4.8 Bolt class

d = 16 [mm] Bolt diameter

do = 18 [mm] Bolt hole diameter

As= 1.57 [cm²] Alre of the effective section of the bolt

Av = 2.01 [cm²] Alre of bolt section

fyb = 280.00 [MPa] Plasticity limit

fub = 400.00 [MPa] Bolt tensile strength

n = 2 Number of bolt columns

Bolt spacing: 60 [mm]

e_1= 40 [mm] Distance of the center of gravity of the first bolt from the end of the bar

e2 = 30 [mm] Distance of the bolt axis from the edge of the bar

ec = 100 [mm] Distance from the end of the bar to the point of intersection of the baraxes

Gusset:

1 = 500 [mm] Plate length

hp= 300 [mm] Height of the plate

tp = 50 [mm] Plate thickness

Settings:

h_1 = 50 [mm] Notching

V1 = 50 [mm]Notching

h2= 50 [mm]Notching

V2= 50 [mm] Notching

h3 = 50 [mm] Notching

V3 = 50 [mm] Notching

h4 = 50 [mm] Notching

V4= 50[mm] Notching

Center of gravity of the sheet relative to the center of gravity of the bars (-0.0)

ev = 150 [mm] Vertical distance of the end of the gusset from the point of Intersection of the axes of the bars

eh= 250 [mm] Horizontal distance of the end of the gusset from the point of intersection of the axes of the bars

Material: STEEL

$\sigma =$ 235.00 [kN] results

Efforts:

Case 6: ELU V90 (1+2)*1.35+3 1.50+5*1.80

N= -4.53 [kN] Axial force

N= -3.59 [kN] Axial force

N = -4.08 [kN] Axial force

N= -3.09 [kN] Axial force

Results:

Platinum:

Gusset section

N1<Nres -4.53 <111.66 checked (0.04)

N2<Nres -3,591 < 494963998579698800.00 checked (0.00)

Bars:

T_1 = 114.18 [kN] Shear resistance of bolts

M1= 324.71 [kN] Bar resistance

N1<min(T1 M1). -4,53< 114, 18 checked (0.04)

T2 = 171.27 [kN] Resistance of bolts to shearing

M2= 324.71 [kN] Resistance of the bar

N2 < min(T2: M2) -3.59 < 171.27 checked (0.02)

T3= 114.18 [kN] Shear resistance of bolts

M = 324.71 [kN] Resistance of the bar

N3<min(T3.M3) -4.08 < 114.18 checked (0.04)

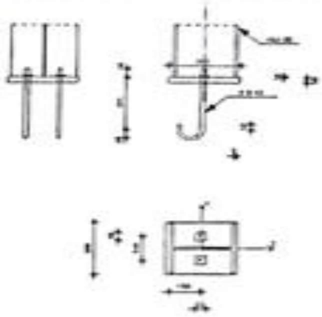
T4 = 171.27 [KN] Bolt resistance to shear

M4= 324.71 [kN] Bar resistance

N4 < min(T4; M4)-3.89 < 171.27 checked (0.02)

Satisfactory assembly with respect to the Standard:

Calculation of the articulated column base:



General:

Assembly No.: 6

Assembly Name: Hinged Post Base

Structure node: 209

Structural bars: 203

Geometry:

Column:

Profile: HEA 280

Bar No.: 203

α = 0.0[Deg] Tilt angle

hc = 270[mm] Height of columc section

bfc = 280[mm] length of columc section

twc= 8 [mm] Thickness of the web of the column section

trc= 13 [mm] Thickness of the flange of the post section

rc= 24[mm] Fillet radius of the column section

Ac = 97.26[cm²] Column section area

lys = 13673.30 [cm4] Moment of inertia of the column section

Material: STEEL

σec = 235.00[MPa] Resistance

Main column base plate:

Iod= 300[mm] Length

bpd= 300 [mm]Width

tpd= 30 [mm] Spreader

Material: STEEL

σe = 235.00[MPa] Resistance

Presealing plate:

Ipp= 405 [mm] Length

bpp= 308 [mm] Width

Tpp= 5 [mm] Thickness

Anchoring:

Class= 4.6 Anchor rod class

d= 16 [mm] Bolt diameter

do = 16 [mm] Diameter of holes for anchor rods

n= 2 Number of anchor rods in column

ev= 140 [mm] Center distance

Dimensions of anchor rods:

L_1=48 [mm]

L2=300 [mm]

L3=96 [mm]

L4=32 [mm]

Brochure:

Iwd= 40 [mm] Length

bwd= 48 [mm] Width

Twd= 10 [mm] Thickness

Insulated sole:

L= 2700 [mm] Sole length

B= 4000 [mm] Width of sole

H= 900 [mm] Height of the sole

Concrete:

f_{c28} = 20.00 [MPa] Resistance

σ_{bc} = 11.33 [MPa] Resistance

n= 15.00 Acler/Concrete ratio

Welds:

ap = 10 [mm] column pled main plate

Efforts:

Case: 6: ELU V90 (1+2)*1.35+3*1.50+5*1.80

Nc= 338.69 [kN] Axial compressive force

Nt = 0.00 [kN] Axial tensile force

Qy = -0.00 [kN] Shear force

Qz= 0.00 [kN] Shear force

N(Qy) = -338.69[kN] Axial force

N(Qz) = -338.69 [kN] Axial force

Results:

Concrete:

Pm= 3.76 [MPa] Maximum stress in concrete

hb= 2700 [mm]

bb= 4000 [mm]

K = max(1.01+(3-bp/bo-ly/hb) √[(1-bp/bo)*(1-lps/ho)]), K<=4.0

K= 3.55 Dimetric pressure zone coefficient

pm≤ 0.5*K* 3.76 < 20.1. checked

COLUMN:

S= 140 [mm] Vertical distance between anchor bolts

N≤ $\sigma_{ec} \times t_{wc} \times \pi(s - t_{wc})/2$ 0.00< 389.81 checked (0.00)

Core welding:

N≤ $(s - t_{wc})\ \sigma_{ec}$×ap/(k×$\sqrt{(0.2)}$) -71.61<990.90 checked (0.07)

Sole welding:

N≤ $(2 \times ap(2 \times bfc - t_{wc}) \times \sigma_{ec})/(tfc \times k \times \sqrt{2})$ -267,08<964, 15 checked (0.28)

Anchoring:

Adhesion:

checked

N≤ π× d ×τs(L2 + 6.4× r+3.5L4) 0.00 <39.04, checked (0.00)

N≤0.8× As × σ_e 0.00<30.14 checked (0.00)

Platinum:

Compression:

N≤1.185× $\sigma_e (tpd)^2/6 \times \mathrm{bpd} \times \mathrm{Ipd}/(0.8 \times (\mathrm{Mmax/p} \times a^2) \times a^2)$ -338, 691 < 196267

checked (0.39)

Flexion 1-1

N≤ $(tbd \times \text{bpd} \times \text{Ipd} \times \sigma_s)/(0.8^2 \times (\text{bpd} - \text{bf}c)^2)$338.69<74355.47 checked (0.00)

Flexion 2-2

N≤ $(tpd^2 \times \text{bpd} \times \text{Ipd} \times \sigma_s)/(0.8^2 \times 1.22^2 \times (\text{Ipd} \times \text{hc})^2$ 338.59<22202.95 checked (0.02)

Satisfactory assembly with respect to the Standard:

CALCULATION OF FOUNDATIONS:

The project is located in the area of NOUMERATE commune of ghardaia in a semi-rocky terraia. An admissible stress of 5.00 bars given by the laboratory for the sizing of foundations is limited to 3.00 bars

$A_a = \frac{N_U\ (A-a)}{8 \times d_a \times \partial_s}$

NSER max =191.55 KN

S=191.55/300 = 0.64m²

For the footings under HEA280 posts we take A=B=2.00m×2.00m

Numax=268.65 KN

As = 3.00 m²

SOLE

a/b= 1,000 A/B=1,000 S=4,000

contr.sol = 300.00 S.so l= 0.64 A=B = 0.8

Appendix 2

Bill of Quantities

N°	Désignation des Ouvrages	U	Qua.	P. U	Montant
	A- Travaux Gros Œuvres				
	01 - Lot : Terrassement				
1.01	Excavation des Fouilles en puits ou en rigole en terrain ordinaire	m³	28.500		
1,02	Excavation des Fouilles en puits ou en rigole en terrain dur ou semi-rocheux	m³	37.700		
1,03	Excavation des Fouilles en puits ou en rigole en terrain rocheux	m³	39.200		
1,04	Remblai des fouilles	m³	18.000		
1,05	Remblais d'apport (sable fin et homogène) pour rattrappage	m³	280.000		
	Sous -Total 01 ………………………… ……………………………………………….. =				
	02 - Lot : Travaux Infrastructure				
2.01	Gros béton dosé à 250 kg/m3 pour fondation (HTS)	m³	8.200		
2.02	Béton de propreté sous longrines dosé 200 KG/M3 en ciment HTS	m³	11.160		
2.03	Béton Arme pour semelles isoleé dosé à 350 Kg/M3 en ciment (HTS)	m³	32.000		
2.04	Béton arme pour futs dosé à 350 Kg/M3 en ciment (HTS)	m³	12.800		
2.05	Béton arme pour longrines 30*40 dose à 350 kg/m3 en ciment (CPJ)	m³	21.690		
2,06	mur en voil e=15 pour fausse	m²	51.840		
2,07	Gros béton dosé à 250 kg/m3 pour accés d'hangar	m³	3.970		
2,08	Mûr de rattrappage en béton banché ep:30cm en ciment (HTS)	m³	21.600		
2,09	Hérisson en pierre seche ep : 20 Cm	m²	393.24		
2,10	Dalle flottate en béton légèrement armée ep:10cm	m²	379.10		
2,11	Regard en béton armé de (0,60x0,60) en ciment (HTS)	U	2.00		
2,12	F/P de tube PVC type écoulement PN4				
	Diam : 40	Ml	3.00		
	Diam : 63	Ml	8.80		
	Diam : 110	Ml	15.00		
	Sous -Total 02 ………………………… ……………………………………………….. =				
	03 - Travaux de Superstructure et charpente métalique				

3,01	Tiges d'ancrage Rond 27mm	U	56.000		
3,02	Tiges d'ancrage Rond 20mm	U	12.000		
3,03	Poteaux HEA 280	Kg	7311.480		
3,04	Fermes HEA 280	Kg	8388.632		
3,05	Sablieres HEA 100	Kg	820.032		
3,06	Pannes IPE 120	Kg	4050.176		
3,07	C/V horizontales et verticales	Kg	3237.456		
	Potelets IPE 200	Kg	986.430		
3,09	Montant HEA 280	Kg	350.000		
3,10	Poutres IPE 200	Kg	3342.616		
3,11	Solives IPE 180	Kg	2344.518		
3,12	Coiffes UPN 200	Kg	2377.000		
3,13	Couverture	m²	460.000		
3,14	Bardage	m²	140.000		
3,15	Bandeaux	Ml	170		
3,16	Chenaux	Ml	24.000		
3,17	DEP	Ml	32.000		
3,18	Béton arme pour chainage sur mur brique dosé à 350 kg/m3	m³	10.116		
3,19	delle de compression légerement armé sur panneau sandwich pour mézanine	m²	140.00		
	Sous -Total 03 ………………………….. …………………………….…………….. =				
	04 - Travaux de maçonnerie				
4,01	Construction en brique double parois 10+5+10 Cm	m²	498.00		
4,02	Construction en brique simple parois 10cm	m²	34.65		
4,03	Construction en brique simple parois 15cm pour terrasse	m²	241.70		
4,05	Enduit exterieur en mortier de ciment tyrolienne	m²	506.80		
4,06	Enduit interieur en platre sur mur	m²	411.20		
4,07	Enduit interieur en mortier de ciment pour salle d'eau , lavage, et atelier	m²	605.40		
	Sous -Total 04 ………………………….. ……………………….………… …….. =				
N°	**Désignation des Ouvrages**	**U**	**Qua.**		
	05 - Travaux de revêtement				
5,01	F/P de carrelage granito mono-couche 1er choix 33 x 33 cm couleur aux choix du Maitre de L'ouvrage	m²	140.12		
5,02	F/P Resine sol	m²	239.00		

	époxy pour revetement de sol				
5,03	F/P de faience 1er choix pour salle d'eau	m²	99.33		
5,04	F.P de plinthe vernissée 8*33 cm	Ml	75.00		
5,05	F/P faux plafond en placoplatre blanc pour espace bureaux avec points lumineux LED et toutes sujétions de la bonne éxecution	m²	68.56		
5,06	F/P faux plafond en PVC evec points lumineux LED pour salle d'eau avec toute sujétions de la bonne exécution	m²	18.72		
	Sous -Total 05 …………………………. ………………………….……………….. =				
		Total Gros Œuvres …………………….………… …….. =			
	B- Travaux Secondaires				
	06 – Menuiserie				
7,01	F/P Porte garage electrique Dim:4,00*4,50	U	3		
	F/P Porte garage electrique Dim:3,00*4,50	U	1		
7,02	F/P d'une porte en aluminum y compris serrure et toute sujétions de la. bonne exécution				
	Dim:90 x 217 cm	U	7		
	Dim: 78 x 217 cm	U	6		
7,03	F/P d'une porte en aluminum en deux ventaux y compris serrure et toute sujétions de la.bonne execution Dim: 1,30 x 217 cm	U	1		
7,04	F/P d'une grille d'aeration en aluminum Dim: 1,50x 0,70 m	U	14		
7,05	F/P panneau amovible en aluminum pour séparation des bureaux	m²	42		
7,06	F/P panneau amovible en aluminum avec partie superieur vitrée y compris stores		18		
7,07	F/P garde corps en fer forgé h=1,20	Ml	7		
	Sous -Total 06 …………………………. ………………………….……………….. =				
	07 - Plomberie sanitaire				
7.01	F/P Cuvette wc à la turque y compris branchement et accessoires	U	1		

7.02	F/P Lavabo sur socle y compris mélangeur, syphon, mirroir, branchement et accessoires	U	3		
7.03	F/P siege wc à l'anglaise y compris branchement et accessoires	U	1		
7.04	F/P cuvette douche y compris mélangeur, syphon, branchement et accessoires	U	4		
7.05	F.P de Tuyauterie en PVC				
	Ø : 160	ML	7.00		
	Ø : 110	ML	2.00		
	Ø : 80	ML	7.00		
	Ø : 63	ML	6.00		
	Ø : 40	ML	2.00		
7.06	F.P siphon de sol Diam 63 mm	U	2		
7.07	F.P Caniveau avec une couvert Grillé				
	Dim : 20 x 20 x L	ML	40.00		
7.08	F.P Regard de viste en HTS avec tampon				
	Dim : 80 x 80 cm	U	8		
	Dim : 60 x 60 cm	U	3		
	Dim : 60 x 60 cm (Avaloir)	U	1		
7.09	F.P Regard Dégraisseur en HTS avec tampon				
	Dim : 100 x 100 cm	U	2		
7.10	F.P Tuyauterie en MULTI-COUCHE				
	Ø : 14/16	ML	20.00		
	Ø : 12/14	ML	12.00		
	Ø : 20	ML	pm		
7.11	F.P Robinet avec applique				
	Ø : 14/16	U	4		
7.12	F.P Robinet d'arrêt				
	Ø : 14/16	U	4		
	Ø : 12/14	U	4		
	Ø : 20	U	1		

7.13	F.P Armoir d'incendie de 25 m	U	3		
7.14	F.P Tuyauterie en PPR Ø : 50	ML	25.00		
7.15	F.P Robinet d'arrêt Ø : 50	U	3		
	Sous -Total 07 …………………………… …………………………………………….. =				
N°	**Désignation des Ouvrages**	**U**	**Qua.**		
	08 – Electricité				
8.01	F/P Luminaire fluorescent avec Diffuseur "vasque" Équipe de 02tubes néon2x 40W/220V /1.20m	U	60		
8.02	F/P Luminaire fluorescent avec Diffuseur "vasque" Équipe de 01tube néon 1x40W/220V /1.20m	U	5		
8,03	F/P lampe LED 15 W avec douille	U	6		
8,04	F/P prise de courant encastré10 A/220v	U	46		
8,05	F/P Càble de terre en cuivre nu 1*28mm2	ML	110.00		
8,06	F/P Boite de dérivation	U	16		
8,07	F/P tableau de distribution divisionnaire composé de:	ENS	02		
8,08	06 disjoncteurs divisionnaires unipolaire 10 A				
8,09	06 disjoncteurs divisionnaires unipolaire 16 A				
8,10	F/P Prise de courant triphasé Etanche 400V/32A.	U	02		
8,11	F/P interrupteur S.A 6-10 A encastré	U	10		
8,12	F/P bouton poussoire 6-10 A encastré	U	08		
8,13	F/P interrupteur D,A 6-10 A encastré	U	04		
8,14	F/P tableau de distribution divisionnaire y compris piéces de raccordement, pose, main d'œuvre et toutes sujétions de bonne éxécution 01disjoncteur divisionnaire bipolaire 32 A 01disjoncteur divisionnaire bipolaire 40 A 01disjoncteur diffirencielle Tetra polaire 63 A 02 disjoncteurs divisionnaire	ENS	01		

	tripolaire 32 A				
8,15	F/P disjoncteur unipolaire 16 A avec prise de courant	U	02		
8,16	F/P disjoncteur unipolaire 25 A avec prise de courant	U	01		
8,17	F/P de câble U1000 R02V, y compris toute sujétion de mise en œuvre et de bonne exécution suivant les règles de l'art.				
	Ø 4 x10 mm2 + T	ML	15.00		
	Ø 2 x6mm2 + T	ML	18.00		
	Sous -Total 08 ………………………… ………………………….……………….. =				
	09 - Peinture Vitrerie				
9.01	Peinture vinylique sur mûrs exterieurs	m²	1036.00		
9.02	peinture laquée sur portes metalique	m²	128.00		
9.03	Peinture intérieur vinylique sur toute surfaces	m²	760.00		
	Sous -Total 10 ………………………… ………………………….……………….. =				
TOTAL H .T : + 2 + 3 + 4 + 5 + 6 + 7+ 8 + 9					
T . V . A 19 %					

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Ministère de l'Enseignement Supérieur et de la Recherche Scientifique
جـــامعة غــــــــرداية

Faculté des Sciences et de la Technologie Département Hydraulique et Génie Civil		كلية العلوم و التكنولوجيا قسم العلوم والتكنولوجيا

Université de Ghardaïa

Filière : Génie Civil
Spécialité : Structures.

Autorisation d'impression d'un mémoire du Master

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2. ELOTHMANI Abdelmadjid

Thème
Assessing the effectiveness of structural system selection using Multicriteria decision making approach. A case study of a multipurpose hangar

J'autorise les étudiants mentionnés ci-dessus d'imprimer et déposer leur manuscrit final au niveau du département.

Le chef de département :

بولمسيز الطيب
رئيس قسم الري
و الهندسة المدنية