

**SAAD DAHLEB BLIDA'S UNIVERSITY**

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**THE IMPACT OF INTEROPERABILITY ON THE ROLE  
OF STRUCTURAL DESIGN ENGINEERS IN A BIM  
PROJECT.**

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## **ملخص :**

نمذجة معلومات البناء (BIM) هي عملية إنشاء وإدارة مبنى عبر دورة حياته ، وقد أحدثت تغييرًا كبيرًا للمهندسين المعماريين والمهندسين وصناعة البناء. الهدف من هذه العملية هو ضمان العمل التعاوني بين جميع أصحاب المصلحة في البناء. يعد التحليل الإنشائي جزءًا أساسيًا من دورة حياة المبنى ، ويتم إنشاء المعلومات المطلوبة في التحليل الهيكلي من النموذج المعماري. تختلف الأدوات المستخدمة من قبل المهندسين المعماريين والمهندسين الإنشائيين وقد تؤدي إعادة إدخال المعلومات إلى خطأ بشري لهذا قابلية التشغيل البيئي للبرنامج ضرورية.

في هذا المشروع ، سنقوم بمعالجة مشكلات التشغيل البيئي مع التركيز على مجال التحليل الهيكلي ، وسنقترح الحلول الممكنة.

## **Abstract:**

Building Information Modeling (BIM) is a process of creating and managing a building across its life cycle, it had brought great change for the Architects, engineers, and construction industry. The goal of this process is to guarantee collaborative work between all the construction stakeholders. Structural analysis is an essential part of the building's life cycle, the information needed in the structural analysis is generated from the architectural model. The tools used by architects and structural engineers are different and the re-input of information might lead to human error for that the interoperability of software's is necessary.

In this project, we will be addressing the interoperability issues with a focus on the structural analysis domain, and we will propose possible solutions.

## **Résumé :**

La modélisation des informations du bâtiment (BIM) est un processus de création et de gestion d'un bâtiment tout au long de son cycle de vie, il a apporté de grands changements pour les architectes, les ingénieurs et l'industrie de la construction. L'objectif de cette démarche est de garantir un travail collaboratif entre tous les acteurs de la construction. L'analyse structurelle est une partie essentielle du cycle de vie du bâtiment, les informations nécessaires à l'analyse structurelle sont générées à partir du modèle architectural. Les outils utilisés par les architectes et les ingénieurs structure sont différents et la ressaisie d'informations peut conduire à une erreur humaine c'est pour ça l'interopérabilité des logiciels est nécessaire.

Dans ce projet, nous aborderons les problèmes d'interopérabilité en nous concentrant sur le domaine de l'analyse structurelle, et nous proposerons des solutions possibles.

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*“Believe in yourself and all that you are. Know that there is something inside you that is greater than any obstacle.” Christian D. Larson*

## LIST OF TABLES

Table II.1: IFC versions

Table II.2: Exported file representation in different software.

Table II.3: Structural elements exportation results in different software's.

Table III.1: Evaluation of the loads reverting on the current floor hardy slab.

Table III.2: Evaluation of the loads reverting on the inaccessible terrasse hardy slab.

Table III.3: Evaluation of the loads reverting on the inaccessible terrasse flat slab.

Table III.4: Evaluation of the loads reverting on the current floor flat slab.

Table III.5: The results of loads descent on the column.

Table III.6: Verification of simple compression on the most solicited column.

Table III.7: Period and mass participation before shear walls disposition.

Table III.8: Period and mass participation after shear wall disposition.

Table III.9: Justification of interaction under vertical loads.

Table III.10: Justification of interaction under horizontal loads (X-X).

Table III.11: Justification of interaction under horizontal loads (Y-Y).

Table III.12: Penalty value.

Table III.13: Verification of the resulting seismic effort at the base of the structure.

Table III.14: Verification of the reduced normal effort.

Table III.15: Shifting verification in the sense (X-X).

Table III.16: Shifting verification in the sense (Y-Y).

Table III.17: The P-delta verification in the direction X-X.

Table III.18: the P-delta verification in the direction Y-Y.

Table III.19: The final dimensions of the structural elements.

Table III.20: The values of the load report.

Table III.21: The values of the isostatic moment in span.

Table III.22: The values of the moment at the supports.

Table III.23: The values of the maximum moment in span.

Table III.24: Values of transverse shear force.

Table III.25: Values of shear effort and moments in SLS.

Table III.26: Limit state of resistance to compression verification.

Table III.27: Acroterion solicitations.

Table III.28: Staircase dimensions.

Table III.29: The staircase solicitations.

Table III.30: Reinforcement of the staircase.

Table III.31: Stress verification in SLS.

Table III.32: Reinforcement results in ULS.

Table III.33: Stress verification in SLS.

Table III.34: Solicitations in different columns.

Table III.35: The longitudinal reinforcement of the columns.

Table III.36: The transversal reinforcements of the column.

Table III.37: Buckling verification.

Table III.38: Stress verification.

Table III.39: Shear stress verification.

Table III.40: Values of the nodal zone.

Table III.41: Primary beams reinforcements (25x40) cm<sup>2</sup>.

Table III.42: Secondary beams reinforcements (25x35) cm<sup>2</sup>.

Table III.43: Tangential stress verification.

Table III.44: Longitudinal reinforcements shear verification.

Table III.45: Concrete compression limit state verification.

Table III.46: Shear walls solicitations.

Table III.47: Solicitations and reinforcements of shear wall on all levels y-y.

Table III.49: Solicitations and reinforcements of shear wall on all levels x-x.

Table III.50: Calculation of strip foundation under shear walls.

## LISTE OF FIGURES

- Figure I.1: BIM 3D digital model.
- Figure I.2: Building information modeling process.
- Figure I.3: Time effort distribution curve by MacLeamy.
- Figure I.4: The development of BIM definition from 1975 to 2013.
- Figure I.5: Example of GLIDE Project in Demonstrates the Spiral Staircase.
- Figure I.6: A Masonry wall element progression from LOD 200 through 400.
- Figure I.7: The BIM maturity model by Mark Bew and Mervyn Richards.
- Figure I.8: BIM documentation.
- Figure II.1: Early design drawings.
- Figure II.2: Sketchpad.
- Figure II.3: 3D modeling in CAD.
- Figure II.4: BuildingSMART logo.
- Figure II.5: BIM and interoperability.
- Figure II.6: Physical and analytical model of a structure.
- Figure II.7: Analytical adjust.
- Figure II.8: Test model in Revit structure.
- Figure II.9: The model's different views in Revit.
- Figure II.10: Autodesk Revit export function.
- Figure II.11: IFC function.
- Figure II.12: IFC file import function in Sap2000 and Etabs.
- Figure II.13: RSA function in Autodesk Revit.
- Figure III.1: Ground and current floor plan.
- Figure III.2: The area of the weight reverting on the column B<sub>3</sub>.
- Figure III.3: Shear wall disposition.
- Figure III.4: Response spectrum.
- Figure III.5: Transversal cut on hardy slab.
- Figure III.6: Joists reinforcements in span and on supports.
- Figure III.7: Transversal cut on reinforced flat slab.
- Figure III.8: 3D view on reinforced flat slab.
- Figure III.9: Acroterion transversal cut.
- Figure III.10: Acroterion calculation model.

Figure III.11: The acroterion reinforcement schema.

Figure III.12: The static plan of the staircase.

Figure III.13: Reinforced staircase.

Figure III.14: Transversal cut on reinforced landing beam.

Figure III.15: Reinforced landing beam in 3D.

Figure III.16: Reinforced column (45x45) cm<sup>2</sup>.

Figure III.17: Reinforced column (40x40) cm<sup>2</sup>.

Figure III.18: Current floor primary beams reinforcement schema.

Figure III.19: Current floor reinforced primary beam in 3D.

Figure III.20: Current floor secondary beams reinforcement schema.

Figure III.21: Current floor reinforced secondary beam in 3D.

Figure III.22: Terrasse primary beams reinforcement schema.

Figure III.23: Terrasse reinforced primary beam in 3D.

Figure III.24: Terrasse secondary beams reinforcement schema.

Figure III.25: Terrasse reinforced secondary beam in 3D.

Figure III.26: Stress schema.

Figure III.27: Shear wall reinforcements schema.

Figure III.28: Reinforced shear wall in 3D.

Figure III.29: Reinforced isolated foundation in 3D.

Figure III.30: Reinforced strip foundation in 3D.



## LIST OF SYMBOLS

- $A$  (or  $A_s$  or  $A_l$ ): Area of the section of steel (longitudinal).
- $A_t$ : Sum of areas of the sections of transversal reinforcement.
- $B$ : Area of the concrete section.
- $E_s$ : Young steel module.
- $E_{ij}$ : Instant Young module in the age of days.
- $E_{vj}$ : Delayed Young module in the age of days.
- $F$ : Force or action in general.
- $I$ : Moment of inertia of the homogenized section compared to concrete (ELS)
- $M_{ser}$ : Bending moment of service calculation.
- $M_u$ : Bending moment of ultimate calculation.
- $N_{ser}$ : Normal effort of service calculation.
- $N_u$ : Normal effort of ultimate calculation.
- $P$ : Permanent action.
- $Q$ : Live action.
- $V_u$ : Shear effort of ultimate calculation.
- $a$ : Width of a column or a shear wall.
- $b$ : Width of a beam (table), of a column.
- $b_0$ : Width of the rib of a beam.
- $d$  (and  $d_0$ ): Position of the tense reinforcements (and compressed) compared to the most compressed fiber of the concrete section.
- $e$ : Eccentricity of the normal effort, width of a slab.
- $f_e$ : Steel elasticity limit.
- $f_{cj}$ : Characteristic resistance to the compression of concrete age of days.
- $f_{tj}$ : Characteristic resistance to the traction of concrete age of days.
- $g$ : Unitary permanent charge.
- $h$ : Height of a beam, a foundation.
- $h_0$ : Height of a beam heel.
- $h_1$ : Height of a hardy of a beam.
- $j$ : Number of days of concrete maturity.
- $L$ : A beam reach or a slab, height of a column.
- $L_f$ : Buckling length.
- $n$ : Equivalence coefficient steel-concrete.
- $q$ : Unitary permanent charge.

$S_t$ : Transversal reinforcement spacing.  
 $y_l$ : Depth of the neutral axe in SLS.  
 $y_u$ : Depth of the neutral axe in ULS.  
 $z$  (or  $z_b$ ): Bending torque lever arm.  
 $\alpha_u$ : Depth of the neutral axe dimensioned in ULS.  
 $\gamma_s$ : Partial security coefficient on steel (*gamma*).  
 $\gamma_b$ : Partial security coefficient on concrete.  
 $\varepsilon_{bcmax}$ : Maximum deformation of compressed concrete (*epsilon*).  
 $\varepsilon_{st}$ : Tense reinforcement deformation.  
 $\varepsilon_{sc}$ : Compressed reinforcement deformation.  
 $\eta$ : Cracking coefficient related to a reinforcement.  
 $\lambda$ : Mechanical slenderness of a compressed piece.  
 $\mu_{ser}$ : Ultimate reduced moment in SLS (*mu*).  
 $\mu_u$ : Ultimate reduced moment in ULS.  
 $\nu$ : Poisson coefficient (*nu*).  
 $\rho$ : Report of the steel section on the concrete section (*rho*).  
 $\sigma$ : Normal stress (*sigma*).  
 $\sigma_{bc}$ : Maximum stress of the compressed concrete.  
 $\sigma_{st}$ : The tense steel stress.  
 $\sigma_{sc}$ : Stress in the compressed steel.  
 $\tau$ : Tangential stress (*tau*).  
 $\tau_u$ : Conventional tangential stress.  
 $\tau_s$ : Bond stress.  
 $\varphi$ : Creep coefficient (*phi*).  
 $\phi_l$ : Longitudinal reinforcement diameter.  
 $\phi_t$ : Transversal reinforcement diameter.  
 $\psi_s$ : Sealing coefficient related to a reinforcement (*psi*).

# CONTENTS

<b>ABSTRACTS</b> .....	1
<b>THANKS</b> .....	2
<b>LIST OF TABLES</b> .....	3
<b>LIST OF FIGURES</b> .....	5
<b>LIST OF SYMBOLS</b> .....	7
<b>CONTENTS</b> .....	9
<b>GENERAL INTRODUCTION</b> .....	11
<b>CHAPTRE I: Definition and terminology</b> .....	12
I.1. The definition of BIM .....	12
I.2. The process of BIM .....	13
2.1. BIM in the design development phase .....	13
2.2. BIM in the construction phase .....	15
2.3. BIM in the operation phase .....	15
I.3. The development of BIM .....	15
I.4. Level of development .....	17
I.5. BIM maturity model .....	19
I.6. BIM software tools .....	20
6.1. Bentley systems .....	20
6.2. Revit .....	20
6.3. Tekla .....	20
6.4. NavisWorks .....	21
I.7. The BIM charter.....	21
<b>CHAPTER II: interoperability</b> .....	23
II.1. CAD tools .....	23
1.1. CAD tools in the AEC sector .....	23
1.2. History .....	23
1.3. 3D modeling in CAD .....	24
II.2. BuildingSMART (OpenBIM) .....	25
2.1. Interoperability .....	26
2.2. Industry Foundation Classes (IFC) .....	27
Versions of IFC .....	27

II.3. BIM interoperability with structural analysis software's .....	28
3.1. The physical model .....	28
3.2. The analytical model .....	28
3.3. Analytical objects .....	29
3.4. Working method .....	30
3.5. Conclusion .....	36
<b>CHAPTRE III: A new workflow (BIM process) .....</b>	<b>37</b>
III.1. The structure presentation .....	37
III.2. Evaluation of loads and overloads .....	38
III.3. Pre dimensioning of the structure elements .....	39
<i>a.</i> Hardy slab pre dimensioning .....	39
<i>b.</i> Flat slab pre dimensioning .....	39
<i>c.</i> Beams pre dimensioning .....	39
<i>d.</i> Columns pre dimensioning .....	39
<i>e.</i> Shear wall pre dimensioning .....	41
III.4. Seismic verification .....	42
<i>a.</i> Structure modeling .....	42
<i>b.</i> Spectral model method .....	42
<i>c.</i> Conclusion .....	48
III.5. Secondary elements calculation .....	48
<i>a.</i> Slabs .....	48
<i>b.</i> Acroterion .....	60
<i>c.</i> Staircase .....	64
<i>d.</i> The landing beam .....	68
III.6. Primary elements calculation .....	72
<i>a.</i> Columns .....	72
<i>b.</i> Beams .....	78
<i>c.</i> Shear walls .....	84
III.7. Infrastructure study .....	88
<b>GENERAL CONCLUSION .....</b>	<b>92</b>
<b>REFERENCES .....</b>	<b>93</b>
<b>APPENDICES .....</b>	<b>95</b>
<b>DICTIONARY .....</b>	<b>96</b>

# GENERAL INTRODUCTION

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The modern-day Architecture, Engineering, and construction (AEC) industry are in increasing demand for technology and well-trained professionals who are capable of implementing it. However, the use of digital information along the building life cycle may cause the loss of valuable information. Because the information is still handed in the form of drawings, either physical as printed plots or in a digital but limited format.

Building information modeling (BIM) is one of the promising developments in the AEC industry. The goal of this process is collaborative work between all construction stakeholders. With the help of a central database that feeds all parts of the project, and contains all the information necessary for construction from a digital model simulating reality.

In this thesis, we will be studying the interoperability of software for calculating concrete structures with CAD tools and the impact of data loss during the exchange of IFC models on the role of structural engineers in a BIM process.

This research is structured of three chapters:

**CHAPTER I:** In this chapter, we define the BIM technology and present an overview of its development and process. We also define BIM LOD's, maturity levels, and charter.

**CHAPTER II:** In the second chapter, we will explain interoperability and its exchange formats, define CAD tools. Then conclude the chapter with a study on the interoperability of software for calculating concrete structures.

**CHAPTER III:** In the third chapter, we will be drawing up a five-floor concrete structure based on a BIM model and entering the results on the platform.

# CHAPTRE I: Definition and terminology.

Building Information Modeling is witnessing a massive approach these days. With the global trends the AEC projects are getting more complex which leads Architects, Engineers and Contractors to search for the process that ends up with the fastest and best results.

## I.1. The definition of BIM

Building Information Modeling is a working method based on collaboration around a digital model. In a BIM process, each actor in construction or operation creates, informs and uses this model, and derives from it the information he needs for his job. In return, it feeds the model with new information to end up with an informed virtual object, representative of the construction, its geometric characteristics and behavioral properties.

A digital model or Building Information Model is a three-dimensional digital representation of the functional and physical characteristics of the structure. It is made up of objects and spaces identified and informed (nature, composition, physical and mechanical properties, behavior, performance, etc.). It describes the work during all or part of its life cycle: programming, design, construction, reception, operation, maintenance, and demolition. The digital model describing a building can be unique or made up of the sum of complementary models and/or business models.

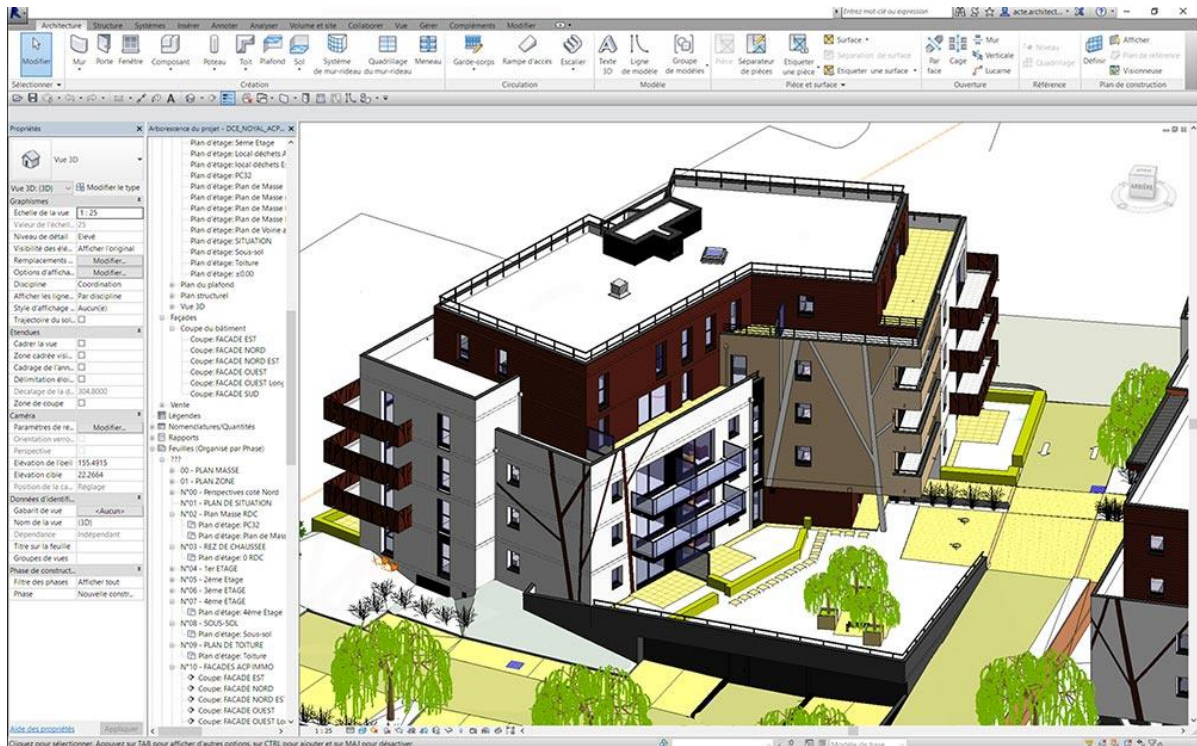


Figure I.1: BIM 3D digital model. [1]

## I.2. The process of BIM

BIM can be viewed as a virtual process that encompasses all aspects, disciplines, and systems of a facility within a single virtual model, allowing all team members (owners, architects, engineers, contractors, subcontractors, and suppliers) to collaborate more accurately and efficiently than traditional processes. As the model is being created, team members are constantly refining and adjusting their portions according to project specifications and design changes to ensure the model is as accurate as possible before the project physically breaks ground. [2]



*Figure I.2:* Building information modeling process. [1]

The foundations of BIM are laid on two pillars, communication and collaboration. The successful implementation of BIM requires early involvement of all project stakeholders. [2]

### 2.1. BIM in the design development phase

BIM provides a large number of advantages for the design and engineering process. Compared to conventional 2D processes, one of the most significant advantages of using BIM is that most of the technical drawings, such as horizontal and vertical sections, are derived directly from the model and are thus automatically consistent with each other. Clash detection between the different partial models makes it possible to identify and resolve conflicts between the design disciplines at an early stage. [3]

BIM also facilitates the integration of computations and simulations in a seamless way, as a lot of input information about the building's geometry and material parameters can be taken directly from the model. [3]

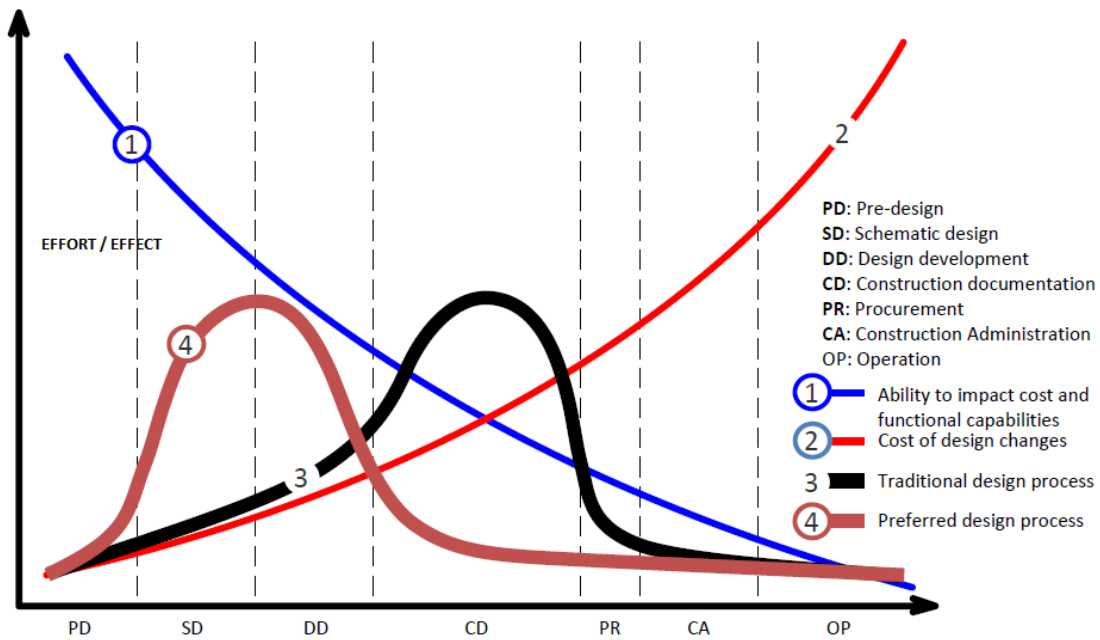


Figure I.3: Time effort distribution curve by MacLeamy [1]

In a BIM-based planning process, the ability to plan coordination requirements in detail and to employ computational analyses in the early design phases makes it possible to evaluate the impact of design decisions more comprehensively and to identify and resolve possible conflicts early on, significantly decreasing the effort required at later phases and improving the overall design quality. [3]

The Time/Effort curve shows that BIM implantation does not incur extra expenses at the design phase but the expenses will pay off at the construction phase later on. BIM implementation does not necessarily prolong the design stage. Instead, it can shorten the design stage by providing a useful platform for pondering design options, early involvement of all team members, and open information sharing. This encourages users to adopt BIM more confidently. The Time/Effort distribution curves are found a handy graphical tool in examining the cost/benefit patterns, and the strengths/weaknesses underlying a BIM implementation model. [4]

## 2.2. BIM in the construction phase

The application of BIM offers significant advantages not only for the design of a built facility, but also for preparing and executing its actual construction. Providing the digital building model as part of the tendering process makes it possible to determine the services required and costs for the contractors when preparing the bid and also facilitates precise billing at a later stage. By means of a 4D Building Information Model, which associates the individual building components with the scheduled construction times, the construction sequence can be validated, spatial collisions can be detected and the site logistics can be organized. A 5D model additionally integrates cost information and can be used to simulate the cost development over time. Finally, the invoicing of construction work, as well as issue management can also be supported using BIM methods. [3]

## 2.3. BIM in the operation phase

Further advantages of the BIM method result from using the digital building model across the comparatively long operation phase of a built facility. A critical prerequisite is the well-organized handover of BIM information from the design team to the owner, including all

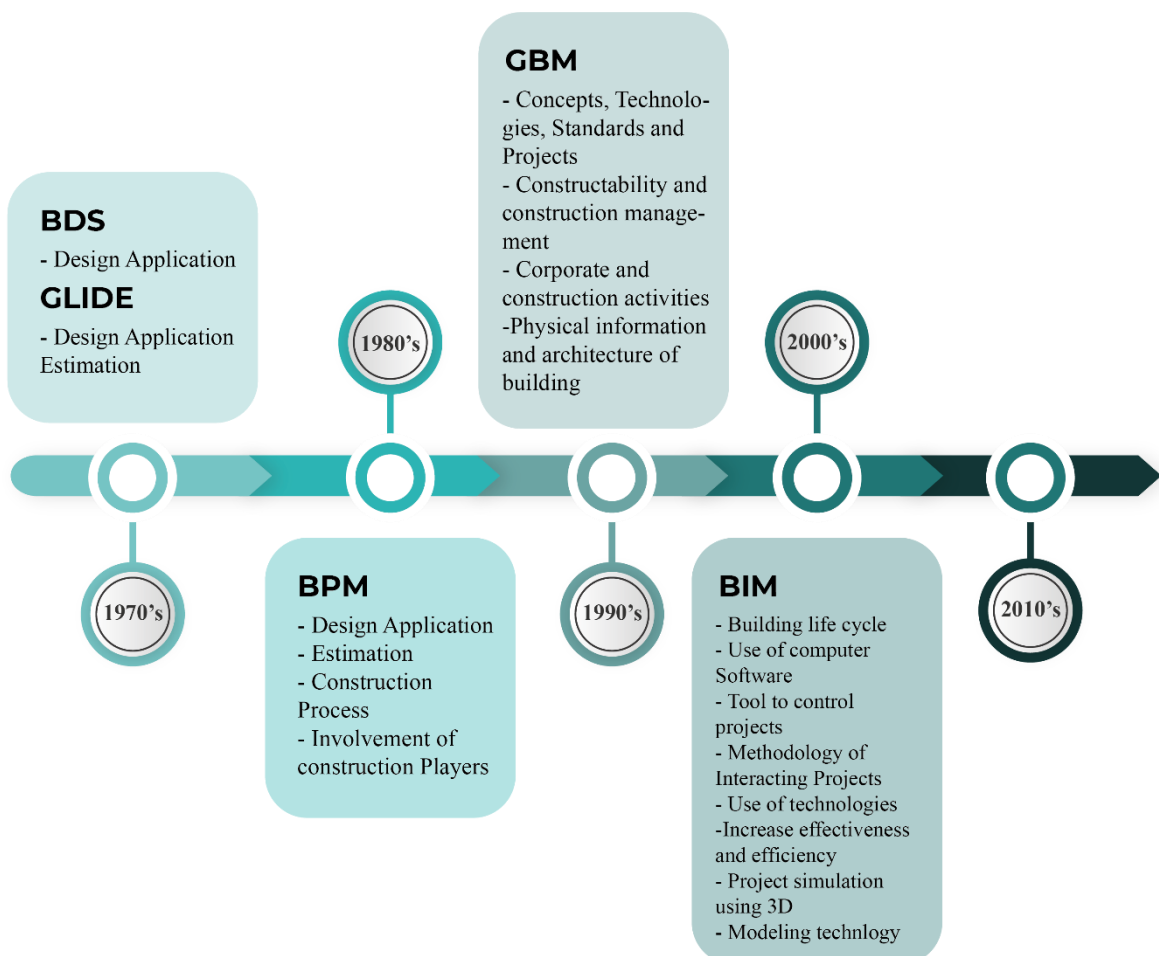


relevant information from the construction phase. If the owner receives high-value digital information instead of ‘dead’ drawings, he can feed them directly into his facility or asset management systems. In the case of buildings, this means that information about room sizes, HVAC, electricity and telecommunication is directly accessible and does not need to be entered manually. [3]

For the operation of a building, information about the installed devices including maintenance cycles and warranty conditions is particularly valuable. An important aspect is the constant upkeep of the digital building model; all changes in the real facility must be recorded in its digital twin. When larger renovations or modifications are required at a later date, the building model provides an excellent basis for the necessary design activities. When the built facility reaches the end of its life cycle and is going to be demolished, the digital twin provides detailed information about the materials used in its construction, in order to plan their environmentally sound recycling or disposal. [3]

### I.3. The development of BIM

BIM concept is originated from Professor Charles Eastman at the Georgia Tech School of Architecture in late 1970. Since its development, it has broadened up in wide perspectives. During the late 1970s, Eastman claimed that drawings for construction were inefficient due to its limitation to visualize the buildings as well as the drawings were not updated. Therefore, there are several organizations in the USA and Finland, which had developed a computer program using ICT in order to solve those problems. [5]

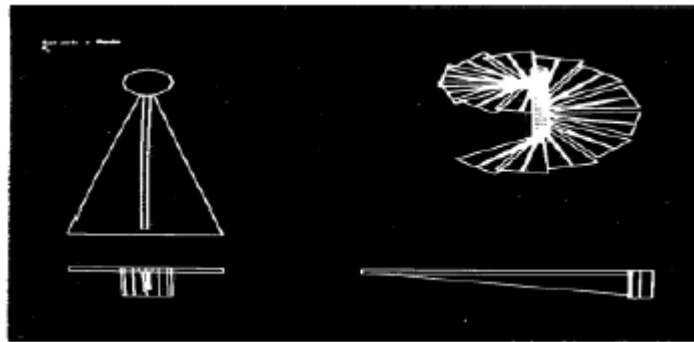


*Figure I.4:* The development of BIM definition from 1975 to 2013

The development of BIM definition has been expanded. In 1975, Professor Charles Eastman had introduced Building Description Systems (BDS) for easier coordination during design development. BDS is defined as database that capable for describing buildings at allowing design and construction. BDS has been used to construct model of complex physical systems that consists of specification of elements. [5]

However, BDS did not get wide popularity because not many architects had a chance to grip on it. Due to the limitation of technology in late 1970s, BDS had been limited to individual library and to potential design choices such as architectural, structural or energy aspect. Therefore, in 1977, Graphical Language for Interactive Design (GLIDE) had been introduced where it incorporated with many parts of BDS.

GLIDE has been expanded to cover certain elements of building and used as a tool in checking the accuracy of data cost estimating and evaluation of structural design. From the improvement of GLIDE, the 2D drawings produced are more consistent and accurate. [5]



*Figure 1.5:* Example of GLIDE Project in Demonstrates the Spiral Staircase. [5]

However, BDS and GLIDE are limited to the involvement of construction players at design stage only. In order to achieve more comprehensive improvement of the programs, construction players need to get involved and collaborate more in construction phase.

After the GLIDE was first introduced, it had been used until 1988. Then a new program called Building Product Model (BPM) had come up in 1989. BPM had covered design application, estimation, construction process and involvement of construction players. Contrast with BDS and GLIDE, BPM acted as a project library that consists of information of projects from planning to the completion of construction. It is a high-level computer interpretable communication for Computer Aided Design (CAD) in construction. As such, it can be used as a compilation and discussion of concepts, technologies, standards as well as projects.

However, the communication under BPM only focused on product information while the AEC industry requires the integration of information and knowledge that used for design and construction management. [5]

Then in 1995, Generic Building Model (GBM) had been introduced using the concept of BPM. GBM had been expanded to integrate information from current and future design that could be used throughout the life cycle in construction process. As the results, GBM had been able to improve projects information that enhanced incorporation of the construction activities. However, the construction industry had become more complex and challenging. It required the adoption of ICT widely for the purpose of better performance and expectation of projects. Therefore, Building Information Modeling (BIM) had been used to fulfill the AEC industry requirements. In 2000, BIM was defined as a structured model that representing building elements. The use of BIM had been expanded from the pre-construction phase to post

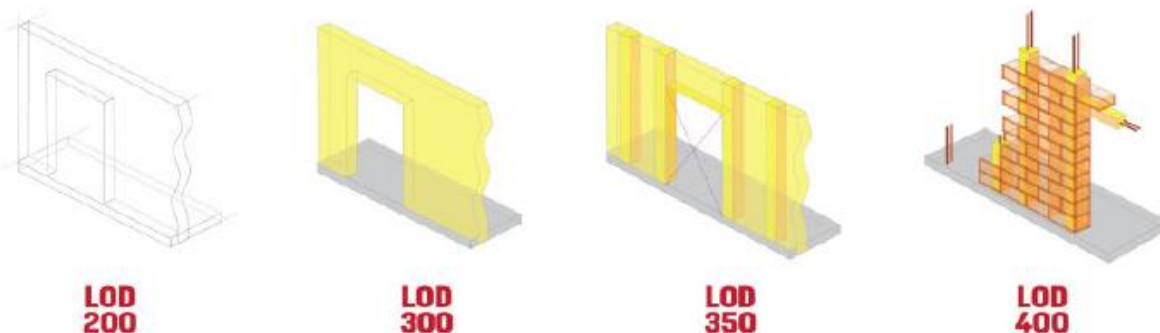
construction phase. Until 2005, it had been defined as a development and use of computer software to simulate the construction and operational of a facility. [5]

In 2006, BIM was defined as a new methodology to manage and increase the AEC performance in completing and managing the projects. In 2008, BIM was adapted as a project simulation that consisted of 3-Dimensional (3D) model of a project component. It was linked and integrated with the information required throughout the project phases. After 2008 until 2013, BIM had been enlarged as a technology revolution that helped to transform the way buildings were conceived, designed, constructed as well as operated. The adoption of BIM concept was said as paradigm shift for AEC industry that helped to achieve efficiency and effectiveness in construction projects. [5]

#### I.4. Level of Development

The Level of Detail or Development is the amount of information contained within an object making up the building model. As the design progresses, detail about the design of the individual elements will increase. [6]

The Level of Development of a BIM model increases as the project progresses, moving from a rough approximation to detail which is known to be accurate, suitable for construction and possibly building management. The American Institute of Architects called it the Level of Development rather than the Level of Detail as it refers to the decisiveness of the information not the amount of information though there may be some correlation between the two. The US-LOD is defined as how seriously you take the data associated with the model. [6]



*Figure I.6:* A Masonry wall element progression from LOD 200 through 400. [7]

**LOD 100:** LOD 100 elements are not geometric representations. Examples are information attached to other model elements or symbols showing the existence of a component but not its shape, size, or precise location. Any information derived from LOD 100 elements must be considered approximate. [8]

**LOD 200:** At this LOD, elements are generic placeholders. They may be recognizable as the components they represent, or they may be volumes for space reservation. Any information derived from LOD 200 elements must be considered approximate. [8]

**LOD 300:** The quantity, size, shape, location, and orientation of the element as designed can be measured directly from the model without referring to non-modeled information such as notes or dimension call-outs. The project origin is defined and the element is located accurately with respect to the project origin. [8]

**LOD 350:** Parts necessary for coordination of the element with nearby or attached elements are modeled. These parts will include such items as supports and connections. The quantity, size,

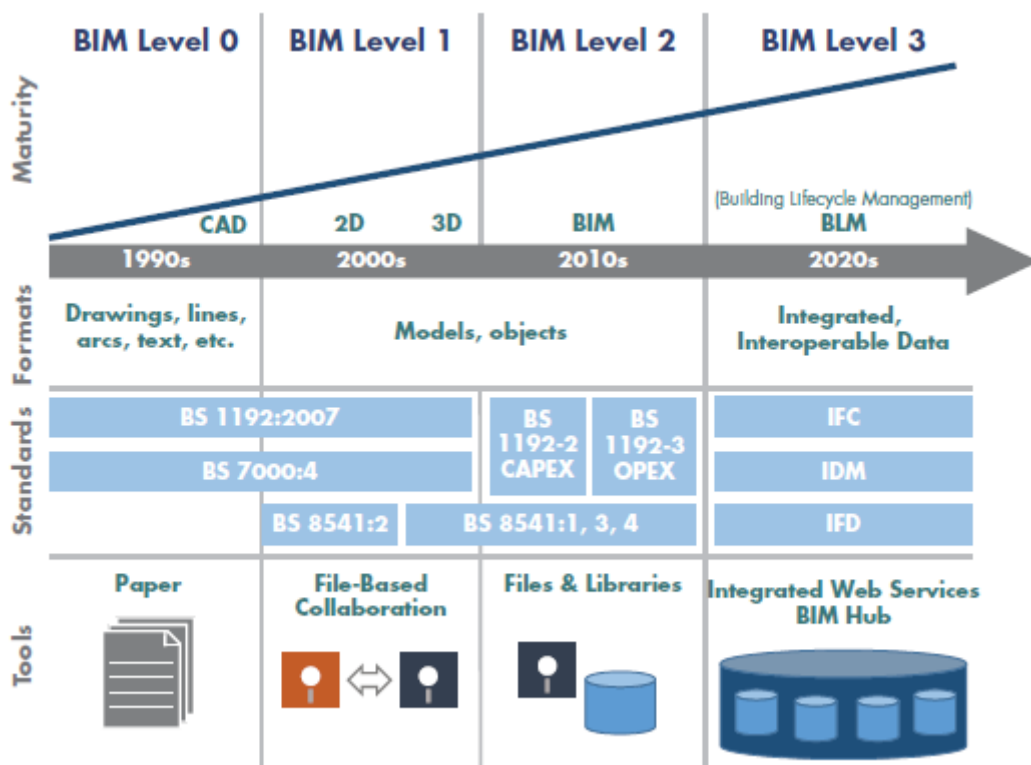
shape, location, and orientation of the element as designed can be measured directly from the model without referring to non-modeled information such as notes or dimension call-outs. [8]

**LOD 400:** An LOD 400 element is modeled at sufficient detail and accuracy for fabrication of the represented component. The quantity, size, shape, location, and orientation of the element as designed can be measured directly from the model without referring to non-modeled information such as notes or dimension call-outs. [8]

**LOD 500:** Since LOD 500 relates to field verification and is not an indication of progression to a higher level of model element geometry or non-graphic information, this Specification does not define or illustrate it. [8]

### I.5. BIM Maturity Model

Another way to characterize BIM is to define a progression of levels of maturity that expresses the degree of collaboration in the process as well as the levels of sophistication of use of the individual tools. In this view, BIM is seen as a series of distinct stages in a journey that began with computer-aided drawing and is taking the industry into the digital age. [9]



**Figure I.7:** The BIM maturity model by Mark Bew and Mervyn Richards. [9]

Since the UK Government BIM Task Group adopted the concept of “BIM Levels,” the four levels (Level 0 to Level 3) have become a widely adopted definition of the criteria for a project to be deemed BIM-compliant.

#### Level 0 BIM

This level is defined as unmanaged CAD. This is likely to be 2D, with information being shared by traditional paper drawings or in some instances, digitally via PDF, essentially separate sources of information covering basic asset information. The majority of the industry is already well ahead of this now. [9]

### **Level 1 BIM**

This is the level at which many companies are currently operating. This typically comprises a mixture of 3D CAD for concept work, and 2D for drafting of statutory approval documentation and Production Information. Models are not shared between project team members. [9]

### **Level 2 BIM**

This is distinguished by collaborative working all parties use their own 3D models, but they are not working on a single, shared model. The collaboration comes in the form of how the information is exchanged between different parties and is the crucial aspect of this level. Design information is shared through a common file format, which enables any organization to combine that data with their own in order to make a federated BIM model, and to carry out interrogative checks on it. [9]

### **Level 3 BIM**

This level represents full collaboration between all disciplines by means of using a single, shared project model that is held in a centralized repository (normally an object database in cloud storage). All parties can access and modify that same model, and the benefit is that it removes the final layer of risk for conflicting information. This is known as “Open BIM.” [9]

## **I.6. BIM software tools**

While 3D modeling is one of BIM pillars, it is not just replacing the 3D modeling like AutoCAD. BIM also includes maintenance data, cost tracking, project scheduling, and more. Down below is a list of some of the well-known BIM software each with their own advantages and disadvantages, but all aim at the same goal which is unification and standardization.

### **6.1. Bentley systems**

Bentley Systems offers a wide range of related products for architecture, engineering, infrastructure, and construction. The Bentley AECOSim is an evolutionary descendant of Triforma, an earlier product. Bentley is a major player in the civil engineering, infrastructure, and plant marketplace.



Bentley AECOSim platform applications are file-based systems, meaning that all actions are immediately written to a file and result in lower loads on memory. The system scales well.

In addition to its base design modeling tools, Bentley has a large array of additional systems (approximately 40 applications), many of which were acquired in support of its civil engineering products. [9]

### **6.2. Revit**

Revit is a well-known and popular BIM platform, introduced by Autodesk in 2002 after Autodesk acquired the Revit software from a start-up company. Revit provides an easy-to-use interface with drag-over hints for each operation and smart cursor. Its menus are well organized according to workflow, and its operator menus gray out unavailable actions within the current system context. Its drawing generation support is very good; its drawing production is strongly associative, so that drawing releases are easily managed.



Revit is a strong platform, especially because of its range of supporting applications. Revit has the largest set of associated applications. [9]

### 6.3. Tekla



Tekla Corp is a Finnish company founded in 1966 with offices worldwide. In 2012, Trimble purchased Tekla.

Tekla has multiple divisions: Building and Construction, Infrastructure, and Energy. Its initial construction product was Xsteel, which

was introduced in the mid-1990s and grew to be a widely used steel detailing application throughout the world. It is largely file-based and scales well. It supports multiple users working on the same project model on a server.

In the early 2000s, Tekla added timber and precast concrete design and fabrication-level detailing for structural and architectural precast. In 2004 the expanded software product was renamed Tekla Structures to reflect its expanded support, including for steel, precast concrete, timber, reinforced concrete, and for structural engineering. Recently, it has added Construction Management capabilities and a structural design application. [9]

### 6.4. NavisWorks



NavisWorks (known for a while as JetStream) is a 3D design review package for Microsoft Windows.

Used primarily in construction industries to complement 3D

design packages (such as Autodesk Revit, AutoCAD, and MicroStation), Navisworks allows users to open and combine 3D models; navigate around them in real-time, and review the model using a set of tools including comments, redlining, viewpoint, and measurements. A selection of plug-ins enhances the package adding interference detection, 4D time simulation, photorealistic rendering and PDF-like publishing.

The software was originally created by Sheffield, UK based developer NavisWorks. Autodesk purchased NavisWorks on June 1, 2007.

## 1.7. The BIM charter

BIM Documentation is fundamental to BIM and is needed at various stages with the project lifecycle. BIM-specific documents provide the project or the owner/operator's information that ensures all of the project team are aligned. This helps the team understand how the flow and management of data is produced and shared throughout the project lifecycle. Projects with coordinated documentation have seen results that are more efficient, this is due to following a clear and concise set of documents produced at the beginning of the project.

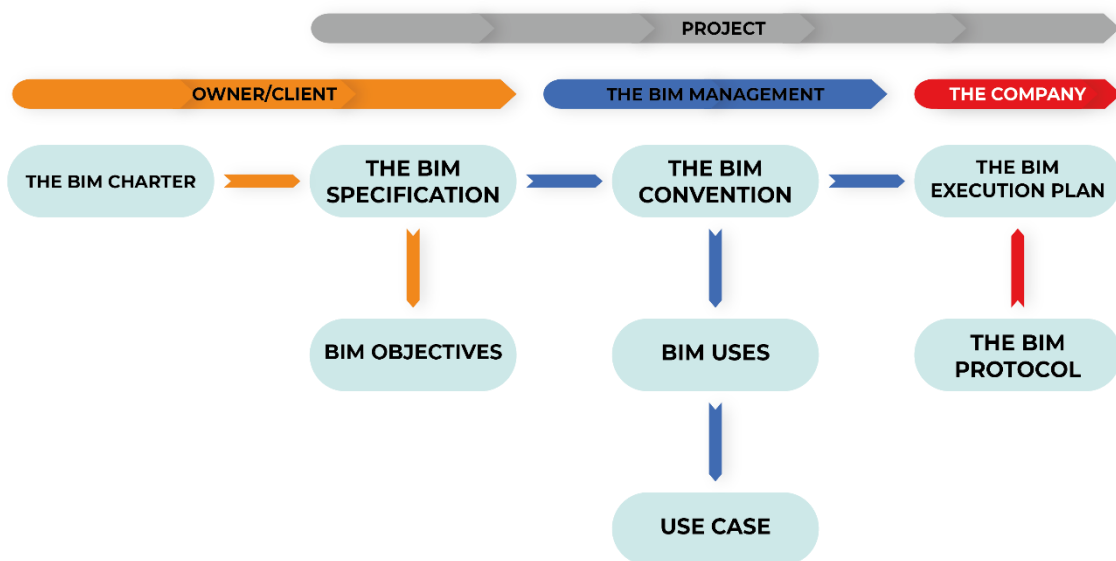
The client who wishes to use BIM must define its charter. This document reflects the objectives, expected performance and requirements of the client in the BIM process and is the matrix of BIM specifications. The BIM charter will be attached to the contacts of various construction workers. [10]

**The BIM specifications:** They constitute the BIM components of the construction program by specifying for the project the requirements and objectives of successive speakers and including those that are included in the BIM charter. [10]

**The BIM convention:** A document describing the organizational and graphic representation methods of a specific project as well as the processes, models and uses. . It defines the level of detail and information desired from the model for each stage of the project. The BIM Convention is developed by the BIM Manager at any stage of the work life cycle, but ideally as early as possible. It is submitted to each of the parties at the start of the BIM project, preferably at the start of the phase. The BIM convention is a contractual document.

**The BIM protocol:** A complementary document to the BIM Convention. It brings together a set of rules and procedures to be followed, which define the main axes of the BIM process within each entity. This is a complement to the general conditions (defined in the charter and the agreement) allowing to add additional rights and obligations for the client and the contracting parties in order to facilitate collaborative work, while preserving intellectual property rights and differentiating responsibilities between the parties involved. The BIM protocol can serve as a basis for the development of a BIM Convention.

**The BIM execution plan:** BIM Execution Plan (BEP) is a document shared and agreed by all parties in the project team. It specifies how they will work together through the BIM process to deliver the requirements of the Employment Information Requests. BIM Execution Plans typically detail team roles and responsibilities, deliverables and time scales associated with them, approval procedures and logistics, formats and conventions for file sharing. [6]



*Figure I.8: BIM documentation.*

### I.8. Conclusion

BIM has changed the way buildings are constructed and the way they function. This technology supports all phases of the design, allowing better coordination and faster results. The virtual models contain precise information that is needed to support the construction and give a smooth workflow between all the buildings stakeholders. In addition, the amount of details BIM provides at a lower cost makes it the most efficient solution for the AEC industry.



# CHAPTER II: Interoperability.

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No single computer application can support all of the tasks associated with building design and production. Interoperability depicts the need to pass data between applications, allowing multiple types of experts and applications to contribute to the work at hand [11]. In this chapter we will be discussing the interoperability of BIM for structural analysis software and studying the best way to achieve it for a structural engineer.

## II.1. CAD tools

It was only 30 years ago that nearly all design drawings were produced by hand with pen and paper. Small changes in the design led to erasing and redrawing, whereas major changes often meant starting drawings again from scratch. An old adage commonly heard in architectural practices reflected this practice: “never draw more in the morning than you can erase in the afternoon”. [12]

CAD fundamentally changes this process and has enabled more complex designs.



*Figure II.1:* Early design drawings. [1]

### 1.1. CAD tools in the AEC sector

Computer-aided design (CAD) is the use of technology to aid the design and, particularly, the drafting of a part product, including entire buildings. It is both a visual and symbol-based method of communication, following standard conventions to a specific technical field, such as architecture or engineering. [12]

CAD software assists designers and engineers in a wide variety of automobiles, roads, aircrafts and all types of buildings, through to digital cameras, mobile phones, clothes and computers. It has become an essential tool for modern design, and has helped to lower product development costs and to shorten the design process. [12]

### 1.2. History

The first recorded graphic system was developed by the Massachusetts institute of technology’s (MIT) Lincoln laboratory in the mid-1950s for the United States air force’s air defense systems. Ivan Sutherland extended this technology for his Ph.D. thesis at MIT in 1960 to develop a program called sketchpad, which is considered to be the first step towards CAD. It was the first recorded tool that enabled the designer to interact with the computer graphically using a light pen to draw on the computer’s monitor. [12]





**Figure II.2:** Sketchpad. [1]

Due to the high cost of early computers, large companies were the earliest commercial users of CAD. First generation CAD software systems were typically 2D drafting applications which were primarily intended to automate repetitive drafting chores, but which also enabled scaling and rotation of design elements.

In 1962, early work produced by professor Charles Eastman at Carnegie-Mellon university, the Building Description System, a library of several hundred thousand architectural elements which can be assembled and drawn on screen to form a complete 3D design concept, was established. Personal computers (PCs) first appeared in the early 1980s, which made CAD more widespread. Auto-desk founded in 1982, launched the first CAD software for PCs “AutoCAD release 1”. Bentley systems developed and released ‘MicroStation’. In 1984, Micro-Control systems developed the first 3D wire-frame CAD software named ‘CADKEY’. [12]

In 1985, and with Apple releasing the first Macintosh 128 in 1984, Diehl Graphsoft developed ‘MiniCad’, which rapidly became the best-selling CAD software for the Mac computer. In 1997, Revit Technology Corporation revolutionizes building design with Revit, the world’s first parametric building modeler developed for the AEC industry. Later in 2008, Autodesk launched Revit Series 2007; one of the most popular BIM based CAD applications. In 2010, GoBIM launched the first BIM application for Apple iPhone and iPad. [12]

### **1.3. 3D modeling in CAD**

CAD uses both 2D drawings and 3D models. 3D CAD is the manufacturing standard since the 1990s; this technology has brought many benefits to the industry, in particular by enabling rapid execution and efficiency of very complex design work. The widespread adoption of 3D CAD has largely been driven by the need of businesses to stay competitive. Indeed, 3D CAD allows a much faster delivery of products. Thus, any company that does not use 3D CAD would inevitably fall behind on the other. Therefore, 3D CAD has been used more and more in the design of buildings. [13]

Since a detailed 3D CAD model mirrors the completed project in the computer, it affords a project team the opportunity to practice or rehearse the construction of a unique artifact virtually before building it in reality. [13]



**Figure II.3:** 3D modeling in CAD. [1]

It is more commonly understood that a design documented with a 3D CAD model will most likely have fewer errors and coordination issues because the construction of the model by multiple designer's forces and allows them to reconcile inconsistencies. Evaluation of a design 3D is also faster than with 2D drawings because reviewers can more quickly understand the scope and status of the design. [13]

## **II.2. BuildingSMART (openBIM)**

BuildingSMART international is an organization that aims to improve the exchange of information between software's that are used in the AEC industry.



**Figure II.4:** BuildingSMART logo. [1]

In the late 1994, Autodesk initiated an industry consortium to advise the company on the development of a set of C++ classes that could support integrated application development. Twelve U.S companies joined the consortium. Initially called the Industry Alliance for Interoperability, the Alliance opened membership to all interested parties in September 1995 and changed its name in 1997 to the international alliance for interoperability (IAI). [9]

The new alliance was reconstituted as a nonprofit industry-led international organization with the goal of publishing the Industry Foundation Classes (IFC) as a neutral AEC product data model responding to the building lifecycle. [9]

In 2005, it was felt that the IAI name was too long, confusing, and complex for people to understand. At a meeting in Norway of the IAI Executive Committee, IAI was renamed BuildingSMART, reflecting its ultimate goal. [9]

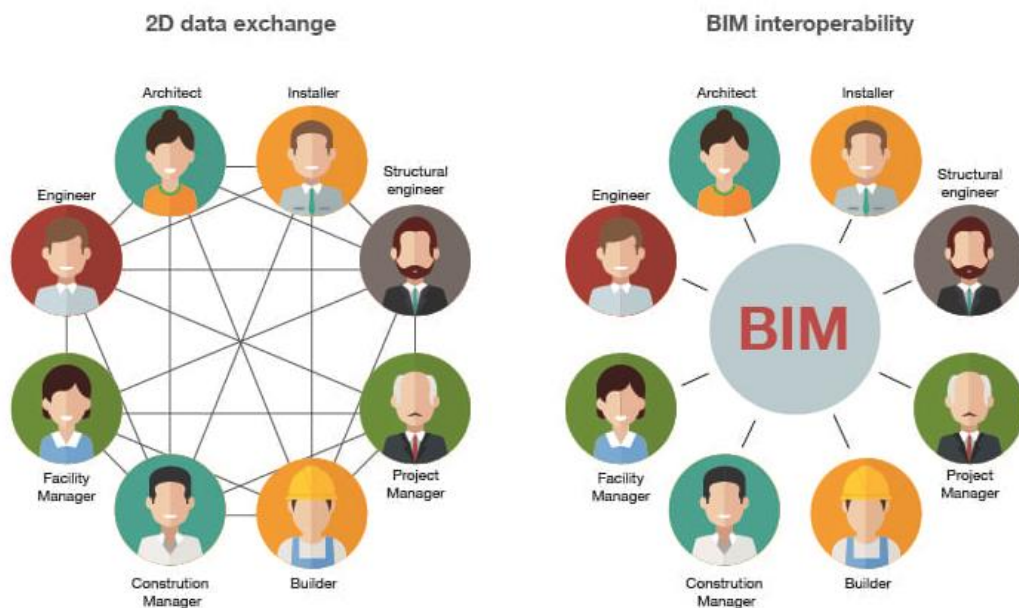
As of 2017, building smart had 17 chapters in 22 countries. The BuildingSMART members meet twice every year to develop or update international standards and to share and document best practices for BIM. [9]

OpenBIM extends the benefits of BIM by improving the accessibility, usability, management and sustainability of digital data in the built asset industry. At its core, openBIM is a collaborative process that is vendor-neutral. OpenBIM processes can be defined as sharable project information that supports seamless collaboration for all project participants. OpenBIM facilitates interoperability to benefit projects and assets throughout their lifecycle [14].

## 2.1. Interoperability

Interoperability is the ability of two or more systems to exchange information. It is one of the pillars of BIM because the information contained in a BIM model needs to be exchanged to be useful.

All stakeholders working in a given project phase, use computer applications which consume and/or supply information processed by different software employed by other collaborators on that phase. Each pair of communicating applications must be able to access (insert, extract, update or modify) a subset of the information created by the other. Likewise, BIM information must flow along the building lifecycle, being dealt by a full range of professionals with their software. Interoperability is key to preventing re-creation or re-input of data and to enable efficient use of information [15].



**Figure II.5:** BIM and interoperability. [1]

IFC (Industry Foundation Classes), a developing standard by buildingSMART International, is often cited as the prominent means by which BIM interoperability can be achieved. Several software applications have been certified to be IFC-compliant but actual use shows that flawed exchanges are too common yet. [15]

## 2.2. Industry foundation classes (IFC)

IFC specifies how information is to be exchanged. It is one of the very few public and internationally recognized standards (ISO/PAS 16739:2005) for exchange of information in the AEC domain. Simply put, it is a specification of the things (physical items or abstract ideas) used in building construction so that they can be represented in a model and the relationships (grouping, association, connection, etc.) that exist between them. [15]



### Versions of IFC

There were six principal releases of the IFC since the first version in 1996: IFC1.5.1, IFC2.0, IFC2x (the x is short for ‘extension’), IFC2x2 (the second extension of IFC2), IFC2x3 (the third extension of IFC2), and IFC4 (formally known as IFC2x4) in 2013. [16]

IFC4 is an advanced schema of IFC2x3, IFC4 is extended and overcomes certain limitations of IFC2x3. The IFC2x3 Certification is based on one Model View Definition, the so-called Coordination View 2.0. The original intend of this MVD was to support the Exchange Requirements for the coordination of BIM-models from architects, structural and building service engineers during the design phase.[17]

Basically, the idea was to use IFC as a reference model for planning coordination. However, during the years more and more came into this MVD, which was for the purpose of design transfer and they finally ended up in a release 2.0 of the CV. [17]

One of the goals for IFC4-implementation and certification is to make the MVDs more specific to their intended purpose and not to mix too many differing requirements into one MVD. With this, the certification also shall become more rigorous, which means, an application can only pass, if it supports the whole MVD without any exceptions. In order to achieve this, the former CV2.0 from IFC2x3 became split into two MVDs for IFC4:

- The Reference View, for viewing and coordination purposes and referencing domain models to each other (e.g., for clash detection).
- The Design Transfer View for purposes of exchanging IFC-models to be imported and used further on in other applications for design and evaluation tasks. [17]

**Table II.1:** IFC versions.

version	year	state
<b>IFC 2x3</b>	February 2006	
<b>IFC XML 2x3</b>	June 2007	
<b>IFC 2x3 TC1</b>	July 2007	Recommended
<b>IFC4</b>	Mars 2013	Graphisoft Archicad 23 certified import / export since September 2019
<b>IFC4 Add1</b>	August 2015	Graphisoft Archicad 23 certified import / export since September 2019
<b>IFC4 Add2</b>	July 2016	Graphisoft Archicad 23 certified import / export since September 2019

### II.3. BIM interoperability with Structural analysis software's

Interoperability is the core of BIM information exchange in the AEC domain. In this thesis we try to find the best way to achieve interoperability for a structural engineer. A lot of companies have invested in the interoperability of software, Autodesk Revit is the market's leader in this segment. Revit provides direct links with many of the structural analysis platform. In addition, it supports exporting with IFC formats.

In this study we will be designing a simple room with Autodesk Revit 2018, the Revit structure model combines a physical representation of the model associated with an analytical representation. This model is used for structural design, and derives structural analysis applications.

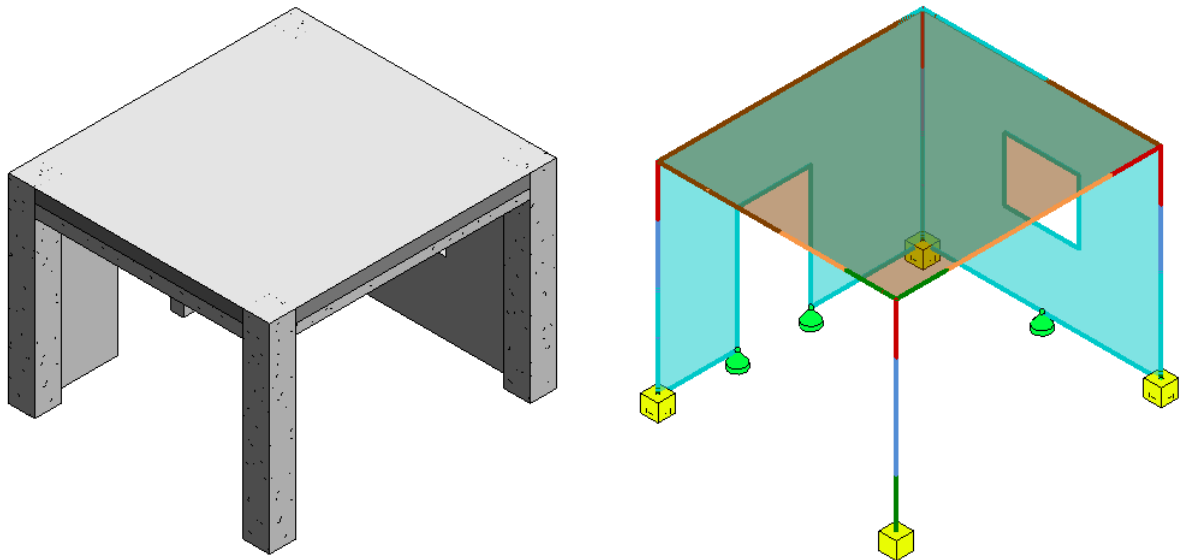
#### 3.1. The physical model

This is the model that most closely resembles the three-dimensional structure to be made. This is the one we rely on to check the graphic consistency of the structure. It represents the formwork of the structure (walls, columns, beams, slabs, etc.).

#### 3.2. The analytical model

It is the "wireframe representation of the physical model of a structure", composed of analytical elements, material properties, loads, and support conditions.

**This calculation model is created automatically as the physical model is developed.**



*Figure II.6:* Physical and analytical model of a structure.

Using a building information model, both the physical and the analytical representations of the structure remain interconnected, facilitating the process of transferring information and keeping the integrity of the model.

### 3.3. Analytical objects

**Columns:** formwork model in black and calculation model in blue of beam type.



**Beams:** formwork model in black and calculation model in red, orange, and green of type beam



**Shear walls:** formwork model in black and calculation model in cyan of type shell.



**Slabs:** formwork model confused with the calculation model in brown of type shell.



We can also modify our analytical model by using the function analytical adjust.



**Figure II.7:** Analytical adjust.

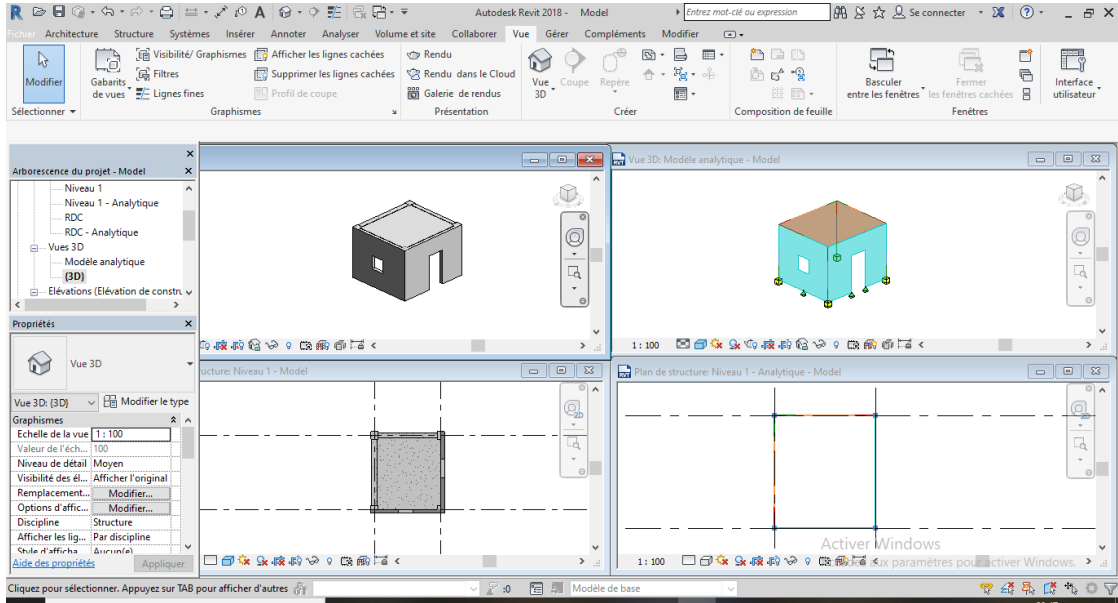
The physical representation denotes the physical layout of the structure in the building (beams, columns, walls, footings, etc.) it also derives the construction documentation. As the physical representation develops, the analytical representation is created automatically, containing the necessary data needed for third-party analysis applications. The analytical representation is an abstract (usually simplified) 3D digital model used for structural analysis. The engineer adds specific loads, material properties, and so forth. And then runs the analysis.

If the engineer chooses to, the analysis program can then return information that dynamically updates the building model and therefore the documentation as well. This capability eliminates much of the redundant work done by structural engineers to model and analyze single or multiple material building frames (steel, concrete, masonry, wood) using many different applications. [18]

### 3.4. Working Method

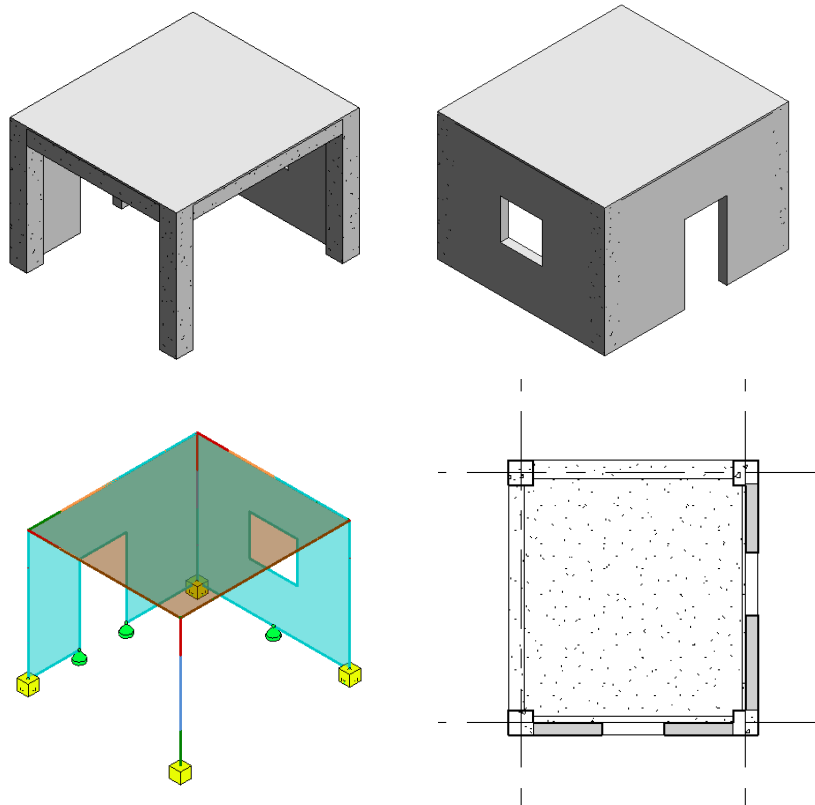
First, a structural model for a small room size 4m x 4m is developed in Autodesk Revit, the model contains all the information from material properties, section properties, degree of freedom, and load description.





**Figure II.8:** Test model in Revit structure.

Different views of the model are shown below.



**Figure II.9:** The model's different views in Revit.

Once the modeling part is finished, we will be testing the interoperability of BIM for structural analysis. We conducted simple experiments using different structural analysis software's as described below:

Sap2000 is a civil engineer software designed by Computers and Structures Inc. it is used for designing and analyzing structural systems. It has some advanced analyzing systems especially for 3D complex structures.



Etabs is an extended 3D buildings analyzing system, it is based on spatial FEA and focuses on spatial features of the structure such as different earthquake zones for slabs.



Autodesk Robot Structural Analysis is an integrated graphic program for modeling, analyzing and designing various types of structures. It can create structures, carry out calculations, and verify results. It also can create documentation for the designed and calculated structure.



In this study we will be using Sap2000 V18, Etabs 2018, and Autodesk RSA 2018. We will be exporting the model using IFC file format and importing it to both Sap2000 V18 and Etabs 2018.

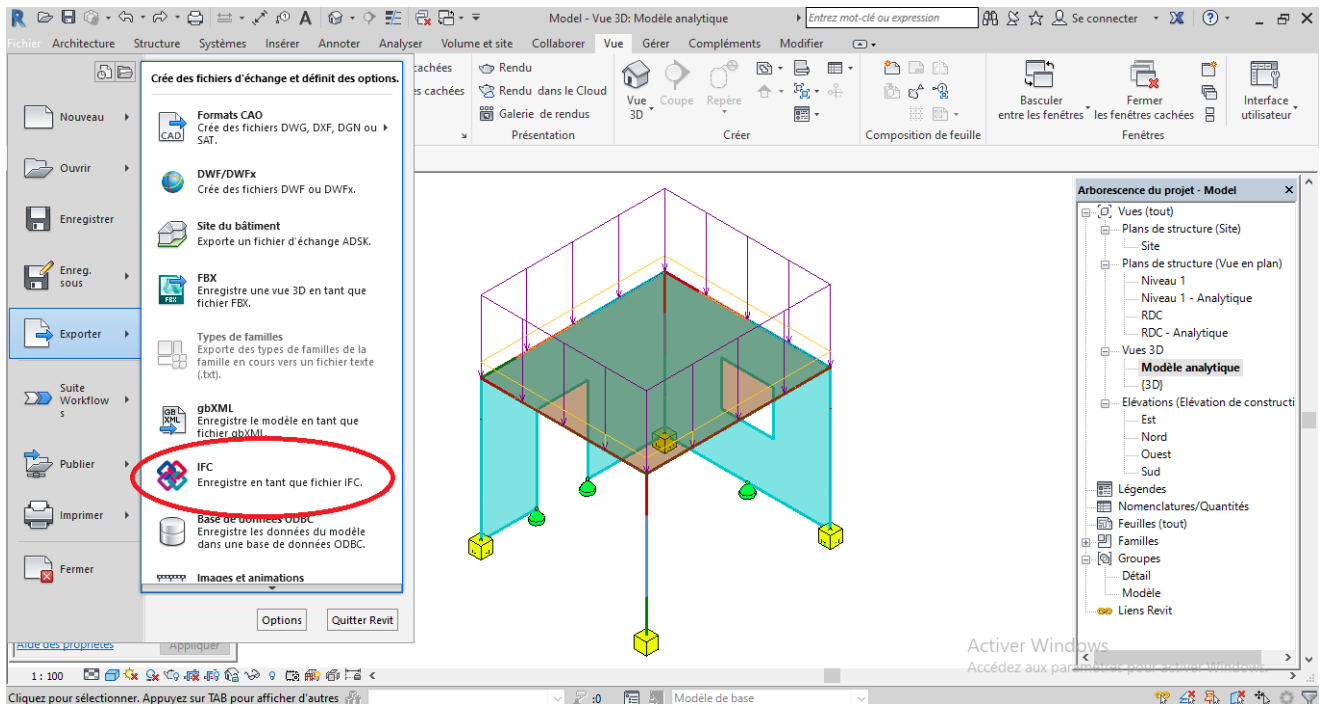


Figure II.10: Autodesk Revit export function.

We will be using IFC4 design transfer view, because it is the most compatible with structural design transfer.



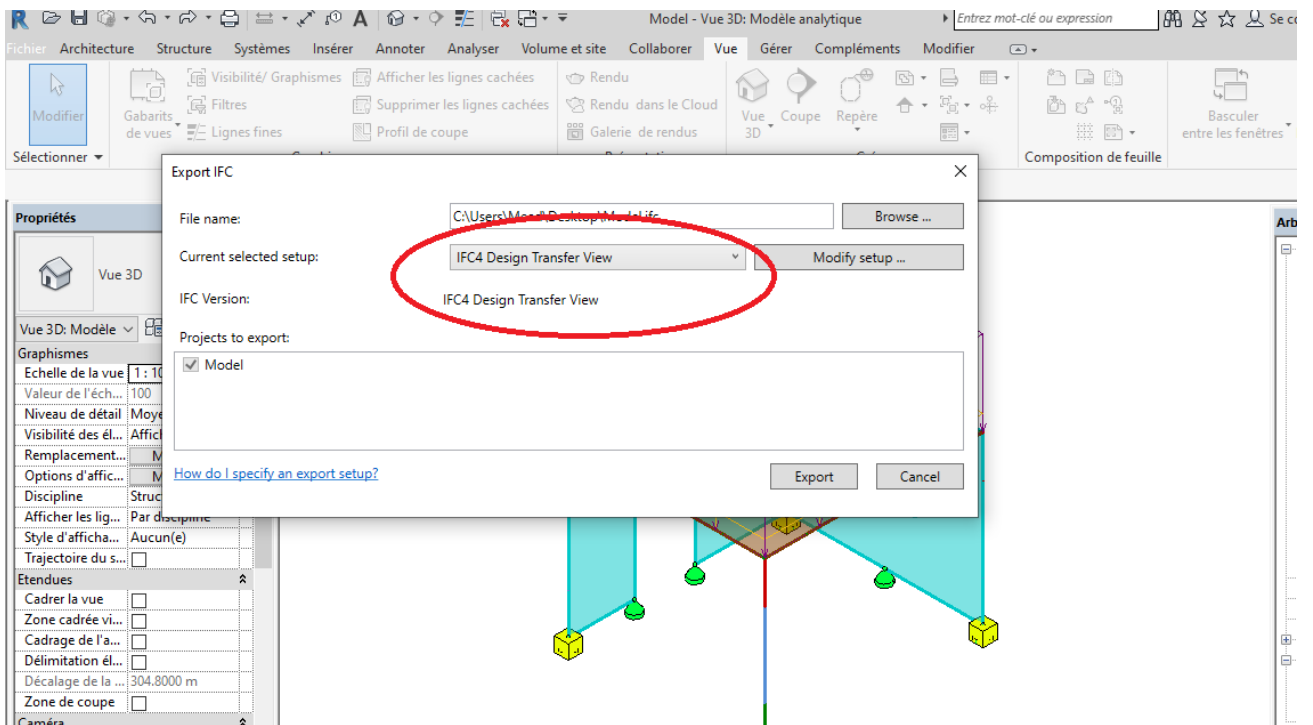


Figure II.11: IFC function.

Once the model is successfully exported, we will import the IFC file in Sap2000 V18 and Etabs 2018, and we will compare the results.

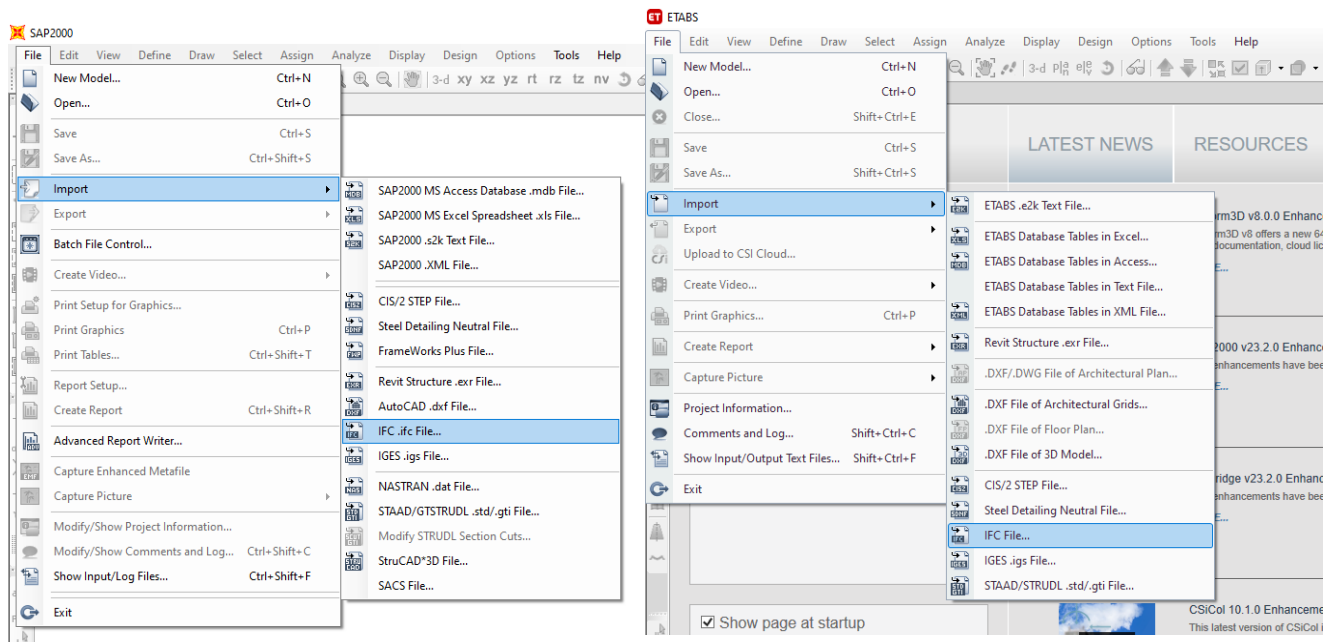
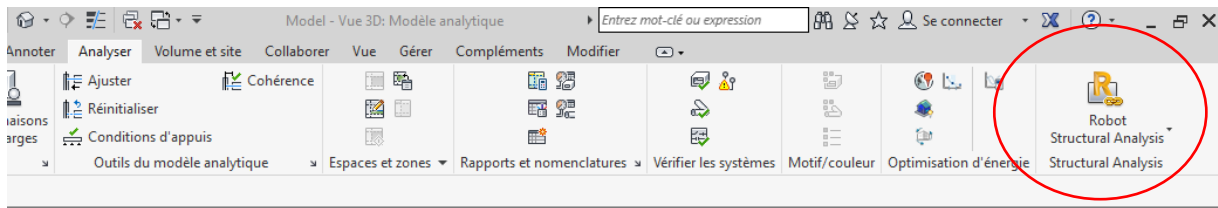


Figure II.12: IFC file import function in Sap2000 and Etabs.

Now using Revit links, we will transfer the model to RSA.



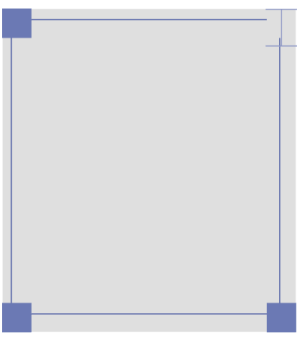
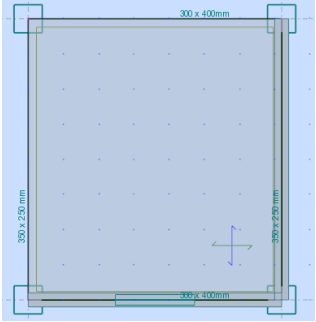
**Figure II.13:** RSA function in Autodesk Revit.

During this import/export process, a few problems occurred that caused unsuccessful import/export results. For example, when IFC files were created in the *Autodesk Revit Structure* and imported into *Etabs 2018* and *SAP2000 V18*, material properties were wrong and loads information could not be loaded, boundary conditions such as a pin on certain point/node/element were also not loaded. In addition, we have noticed some changes in the model information that could cause errors in analyzing the structure.

The results are shown in the tables below:

**Table II.2:** Exported file representation in different software.

	Exported IFC file result in Etabs 2018	Exported IFC file result in SAP2000 V18	File extension results in RSA 2018
3D view			

2D view		no view shown	
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**Table II.3:** Structural elements exportation results in different software's.

Structural analysis software	Structural elements	Observation
<b>Sap2000 V18</b>	Columns	Badly exported some elements lose their properties
	Beams	Badly exported elements sections deformed
	Shear walls	Exported
	Slabs	Exported
	Nodes and restrains	Badly exported
	Loads	Not exported
	Material properties	Badly exported
<b>Etabs 2018</b>	Columns	Badly exported some elements lose their properties
	Beams	Badly exported elements sections deformed
	Shear walls	Exported
	Slabs	Exported
	Nodes and restrains	Badly exported
	Loads	Not exported
	Material properties	Badly exported
<b>Robot Structural analysis</b>	Columns	Exported
	Beams	Exported
	Shear walls	Exported
	Slabs	Exported
	Nodes and restrains	Exported
	Loads	Exported
	Material properties	Exported
	Openings	Exported

From the software point of view, different software's have different information coverage and different usage in the AEC industry. How to import an IFC file and export it for further analysis successfully is another important consideration for interoperability. When an IFC model is exported from one software and imported into another, certain information may miss or become untraceable. Redefining information manually is time consuming and human error-prone. But without a full interoperability this manual input cannot be avoided. In the structural analysis domain, for example, when a shell element is being analyzed, the following information are needed but may not be successfully transferred using IFC models: new classes of load setting, combined material definition, different forces and moments this requires direct support for IFC files and their processing in structural analysis software applications.

Even with direct IFC file support, missing information can still be a big problem in the importation/exportation of IFC files, especially those that are undetected. To address this, developers have to manually check potential information loss before file exchange, or semi-automatically check it in order to avoid/reduce unknown information missing during the file exchange. Human-induced errors also need to be considered in such checking. The sizes and complexities of IFC models affect the importation/exportation process as well. Large models developed in more powerful platforms may contain information that is not directly interpretable in smaller platforms. One way to address this problem is to follow strict information requirements and MVDs to ensure the consistency of models across different platforms.

On the contrary, the link between Autodesk Revit structure and Autodesk Robot makes the transfer operation easier, and assures a smoother workflow. Every structural element was exported perfectly and we didn't notice any information loss during this process. This helps reduce the time needed to recreate or update analysis models, and prevent manual errors that are due to the re-insert of different properties.

### **3.5. Conclusion**

Interoperable data exchange is a very important element in the structural engineering domain, because it ensures collaborative work between all the construction stakeholders, and it reduces errors resulted to manual re-insert of different properties. Based on our review, research gaps were identified with IFC interoperability for structural analysis, and it needs better information coverage. For a civil engineer the best way to achieve interoperability and reduce lost data files is to use links to export models in the BIM method. Therefore, this will facilitate information flow between different parties in the AEC industry with a central model. Resulting in a better information flow and less interoperability problems.

## CHAPTRE III: A new workflow (BIM process).

In this chapter, we applied the BIM process with the Autodesk Revit software to build a concrete structure model including its geometry and sections. Then convert it to Autodesk Robot structural Analysis and obtain structural behavior and explore the security of our structure.

### III.1. The structure presentation

In this project we will be studying a Ground+4 building for residential use with an inaccessible terrasse. This project is a current building with medium importance, its total height is less then 48m, which leads us to classify it according to the RPA99/ version 2003 “art 3.2” in the usage group 2.

- Site geotechnic input:

Ground admissible stress:  $\overline{\sigma_{sol}} = 2,1 \text{ bar}$

Minimum foundation anchoring:  $D = 1,8\text{m}$

- Building geometric characteristics:

The structure presented has a rectangular form. In which the dimensions are:

$L_x = 25,25\text{m}$ .       $L_y = 10,7\text{m}$ .

Total height:  $h_t = 15,30\text{m}$ .

The ground floor height:  $h_{Ground} = 3,06\text{m}$ .

The current floor height:  $h_{floor} = 3,06\text{m}$ .

$f_{c28} = 25 \text{ MPa}$

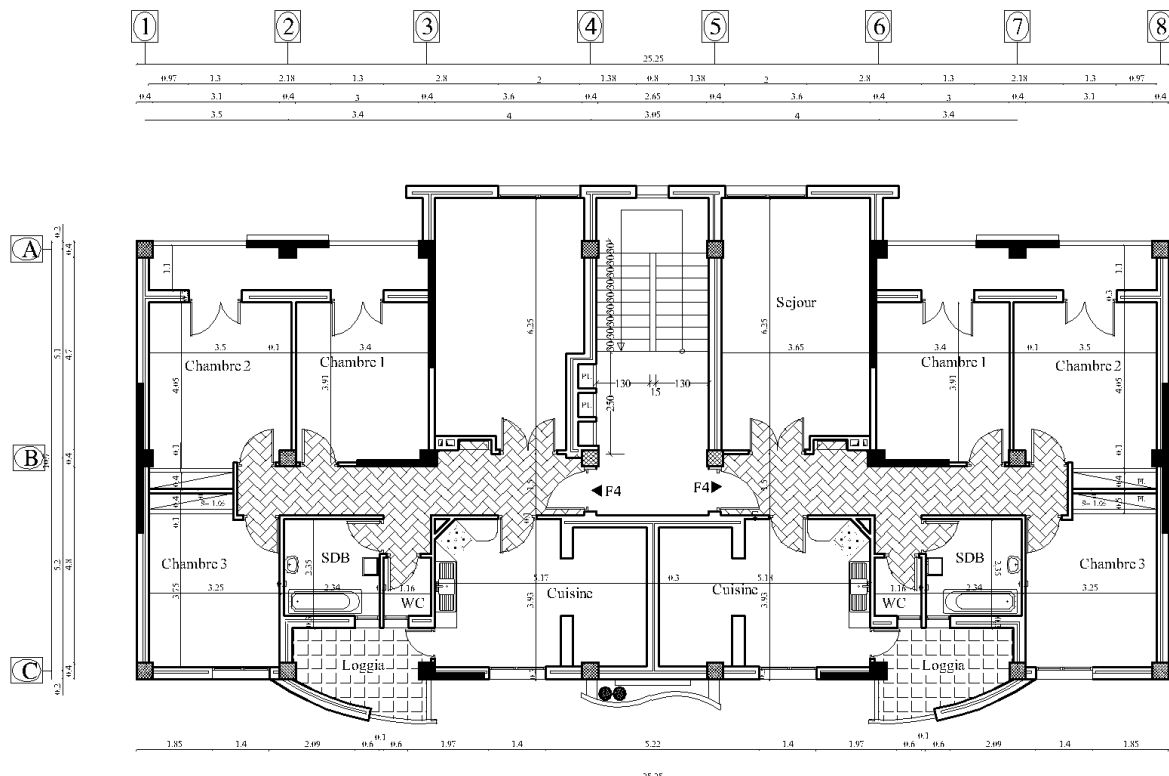


Figure III.1: Ground and current floor plan.

### III.2. Evaluation of loads and overloads

**Table III.1:** Evaluation of the loads reverting on the current floor hardy slab.

N°	LAYERS	unit weight (KN/m <sup>3</sup> )	Depth (m)	Weight (KN/m <sup>2</sup> )
1	Floor tile	22	0,02	0,44
2	Laying mortar	22	0,02	0,44
3	Sand layer	18	0,03	0,54
4	Hardy slab	14	0,16+0,04	2,8
5	Partition wall	09	0,1	0,9
6	Plaster coating	05	0,02	0,10
<b>DEAD LOAD G (KN/M<sup>2</sup>)</b>				5,22
<b>IMPOSED LOAD Q (KN/M<sup>2</sup>)</b>				1,5

**Table III.2:** Evaluation of the loads reverting on the inaccessible terrasse hardy slab.

N°	LAYERS	Unit weight (KN/m <sup>3</sup> )	Depth (m)	Weight (KN/m <sup>2</sup> )
1	Gravel protection	20	0.05	1,00
2	Multilayer waterproofing	6	0.02	0,12
3	Slope form	22	0.05	1,1
4	Hardy slab	/	0.16+0.04	2,80
5	thermic isolation	04	0.04	0,16
6	Plaster coating	10	0.02	0,2
7	Polyan sheet	/	/	0,01
<b>DEAD LOAD G</b>				5,39
<b>IMPOSED LOAD Q</b>				1,00

**Table III.3:** Evaluation of the loads reverting on the inaccessible terrasse flat slab.

N°	LAYERS	Unit weight (KN/m <sup>3</sup> )	Depth (m)	Weight (KN/m <sup>2</sup> )
1	Flat slab	25	0.15	3.75
2	Floor tile	22	0.02	0.44
3	Laying mortar	10	0.02	0.20
4	Sand layer	18	0.01	0.18
5	Hollow brick partition	09	0.1	0.9
6	Plaster coating	10	0.02	0.2
7	Cement plaster	10	2x0.02	2x0.2
<b>DEAD LOAD G</b>				6
<b>IMPOSED LOAD Q</b>				1,5

**Table III.4:** Evaluation of the loads reverting on the current floor flat slab.

N°	LAYERS	Unit weight (KN/m <sup>3</sup> )	Depth (m)	Weight (KN/m <sup>2</sup> )
1	Flat slab	25	0.15	3.75
2	Floor tile	22	0.02	0.44
3	Laying mortar	10	0.02	0.20
4	Sand layer	18	0.01	0.18
5	Hollow brick partition	09	0.1	0.9
6	Plaster coating	10	0.02	0.2
7	Cement plaster	10	2x0.02	2x0.2
<b>DEAD LOAD G</b>				6
<b>IMPOSED LOAD Q</b>				3,5

### III.3. Pre dimensioning of the structure elements

#### a. Hardy slab pre dimensioning

$$\text{We have: } \frac{L}{25} \leq h \leq \frac{L}{20}$$

$$L = 3,60m$$

$$\frac{360}{25} \leq h \leq \frac{360}{20} \rightarrow 14,4 \leq h \leq 18$$

$$\text{We take: } \mathbf{h = 20cm (16 + 4 cm)}$$

#### b. Flat slab pre dimensioning

$$\text{We have: } e \geq \frac{L}{10}$$

$$L = 1,35m$$

$$e \geq \frac{135}{10} \rightarrow e \geq 13,5cm$$

$$\text{We take: } \mathbf{e = 15cm.}$$

#### c. Beams pre dimensioning

*Primary beams:*

$$\text{We have: } \frac{L}{15} \leq h \leq \frac{L}{10}$$

$$L = 4,8m$$

$$\frac{480}{15} \leq h \leq \frac{480}{10} \rightarrow 32cm \leq h \leq 48cm$$

$$\text{We take: } \mathbf{h = 40cm}$$

$$0,4h \leq b \leq 0,7h \rightarrow 18 \leq b \leq 31,5$$

$$\text{We take: } \mathbf{b = 25cm}$$

We adopted for primary beams a section of  $(40 \times 25)cm^2$

*Secondary beams:*

$$\text{We have: } \frac{L}{15} \leq h \leq \frac{L}{10}$$

$$L = 3,6m$$

$$\frac{360}{15} \leq h \leq \frac{360}{10} \rightarrow 24cm \leq h \leq 36cm$$

$$\text{We take: } \mathbf{h = 35cm}$$

$$0,4h \leq b \leq 0,7h \rightarrow 14 \leq b \leq 24,5$$

$$\text{We take: } \mathbf{b = 25cm}$$

We adopted for secondary beams a section of  $(35 \times 25)cm^2$

According to the Algerian seismic regulations:

$$h \geq 30cm , \quad b \geq 20cm , \quad h/b \leq 4$$

conditions are verified.

#### d. Columns pre dimensioning

The columns that have the most influence area: B<sub>3</sub> and B<sub>6</sub>.

*Slabs weight:*

$$S_1 = 1,8 \times 2,4 = 4,32m^2$$

$$S_2 = 1,8 \times 2,35 = 4,23m^2$$

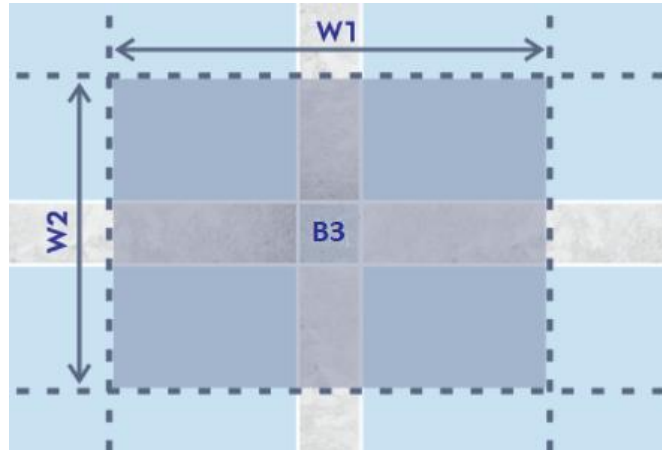
$$S_3 = 1,5 \times 2,35 = 3,53m^2$$

$$S_4 = 1,5 \times 2,4 = 3,6m^2$$

$$S_t = 15,68m^2$$

$$W_{terrace} = 15,68 \times 5,36 = 84,04 KN$$

$$W_{floor} = 15,68 \times 5,18 = 81,22 KN$$



**Figure III.2:** The area of the weight reverting on the column  $B_3$

*Beams weight:*

$$G_p = 0,40 \times 0,25 \times 25 \times (2,35 + 2,4) = 11,875KN$$

$$G_s = 0,35 \times 0,25 \times 25 \times (1,5 + 1,8) = 7,22KN$$

$$G_{beam} = 19,10KN$$

*Imposed loads:*

$$Q_0 = 15,68 \times 1 = 15,68 KN$$

$$Q = 15,68 \times 1,5 = 23,52 KN$$

*Column's weight:*

$$h = 45cm, b = 45cm$$

$$W_c = 0,45 \times 0,45 \times 25 \times 3,06 = 15,49KN$$

*Loading reduction:*

$$1^{st} \text{ floor: } Q_t = Q_0 = 15,68 KN$$

$$2^{nd} \text{ floor: } Q_t = Q_0 + Q = 39,2 KN$$

$$3^{rd} \text{ floor: } Q_t = Q_0 + 1,9 Q = 60,37 KN$$

$$4^{th} \text{ floor: } Q_t = Q_0 + 2,7 Q = 79,18 KN$$

$$\text{Ground: } Q_t = Q_0 + 3,4 Q = 95,65 KN$$



**Table III.5:** The results of loads descent on the column.

<b>FLOOR</b>	<b>PERMANENTE LOAD (KN)</b>				<b>IMPOSED LOAD (KN)</b>		<b>NORMAL EFFORT (KN)</b>	
	$G_{floor}$	$G_{beam}$	$G_{column}$	$G_{total}$	$G_{cum}$	$Q_{floor}$	$Q_{cum}$	$N_u = 1,35G_c + 1,5Q_c$
<b>4<sup>TH</sup></b>	84,04	21,47	15,49	121	121	15.68	15.68	<b>186,87</b>
<b>FLOOR 3<sup>RD</sup></b>	81,22	21,47	15,49	118,18	239,18	23.52	39.2	<b>381,69</b>
<b>FLOOR 2<sup>ND</sup></b>	81,22	21,47	15,49	118,18	357,36	23.52	60.37	<b>572,99</b>
<b>FLOOR 1<sup>ST</sup></b>	81,22	21,47	15,49	118,18	475,54	23.52	79.18	<b>760,75</b>
<b>FLOOR GROUND</b>	81,22	21,47	15,49	118,18	593,72	23.52	95.65	<b>944,99</b>

We need to verify that:  $S \geq \frac{N}{\sigma_{bc}}$

$$\sigma_{bc} = 15 \text{ Mpa}$$

**Table III.6:** Verification of simple compression on the most solicited column.

<b>Floor</b>	<b>Section (m<sup>2</sup>)</b>	$\frac{N}{\sigma_{bc}}$ (m <sup>2</sup> )	<b>observation</b>
<b>4<sup>th</sup> floor</b>	0,16	0,012	<b>Verified</b>
<b>3<sup>rd</sup> floor</b>	0,16	0,025	<b>Verified</b>
<b>2<sup>nd</sup> floor</b>	0,16	0,038	<b>Verified</b>
<b>1<sup>st</sup> floor</b>	0,2025	0,051	<b>Verified</b>
<b>Ground</b>	0,2025	0,062	<b>Verified</b>

The section (45x45) cm<sup>2</sup> is verified for the whole building. we will take the section (45x45) cm<sup>2</sup> for the ground floor and the 1<sup>st</sup> floor. And the section (40x40) cm<sup>2</sup> for the rest of the floors.

*Buckling verification:*

$$\lambda = \frac{L_f}{i} \leq 50$$

$L_f$ : Columns buckling length.

$\lambda$ : Column's slenderness.

$$i = \sqrt{\frac{I}{B}}$$

$$L_f = 0,7 \times 2,66 = 1,862m$$

$$I = \frac{b^4}{12} = 21,3 \times 10^{-4} m^4$$

$$B = b^2 = 0,16 m^2.$$

$$i = 0,115m$$

$$\lambda = \frac{L_f}{i} = \frac{1,862}{0,115} = 16,19 \leq 50 \quad \text{Condition verified.}$$

e. *Shear wall pre dimensioning*

$$a \geq \frac{h_e}{20}$$

$$a \geq \frac{2.66}{20} = 0.133m$$

We take:  $a = 15\text{cm}$

$a$ : The depth of the shear wall.

$$L \geq 4a = 4 \times 15 = 60\text{cm}.$$

$L = 300\text{cm}$ . **Condition verified**

### III.4. Seismic verification

#### a. Structure modeling

To test the BIM method, we created our structure model in Autodesk Revit providing all information necessary in this calculation (geometry, sections, materials, loads ...), once the modeling part is done, we will convert our project to Autodesk Robot Structural Analysis. This software will allow us to automatically determine the dynamic characteristics of our structure.

#### b. Spectral model method

It is the most used method for structure analyzing, in this method we need to verify that:

$$V_{dyn} \leq 0,8V_{st}$$

**The digital period verification:**

$$T = C_T \times h_N$$

$$C_T = 0,05 \quad h_N = 15,30\text{m}$$

$$T = 0.387\text{s}$$

$$T_{exp} = 1.3 \times T \quad \rightarrow \quad T_{exp} = 0.503\text{s}$$

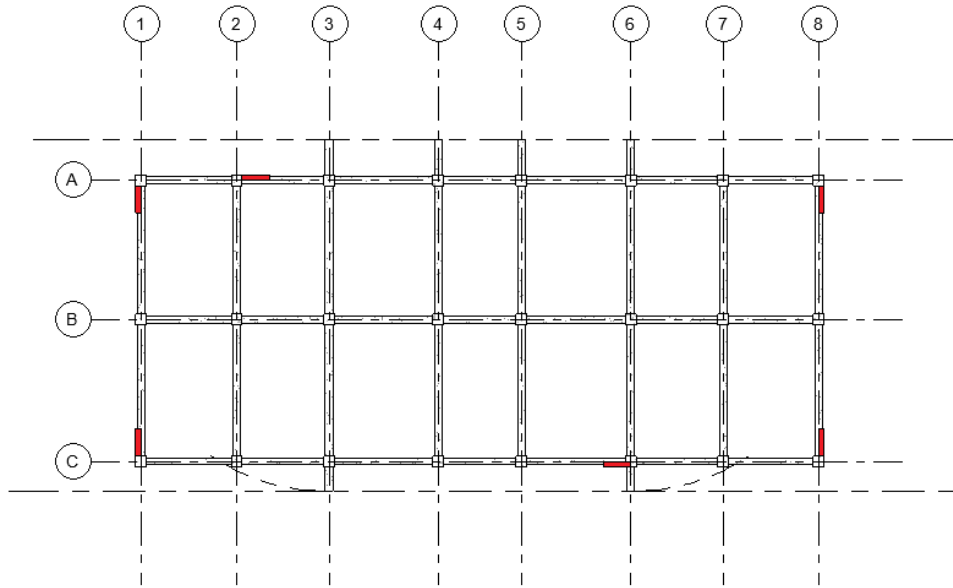
**Table III.7:** Period and mass participation before shear walls disposition.

MODE	FREQUENCY	PERIOD	MASSES CUMULATED UX	MASSES CUMULATED UY	MODAL MASS UX	MODAL MASS UY
1	1,43	0,70	0,00	79,53	0,00	79,53
2	1,48	0,68	26,46	79,53	26,46	0,00
3	1,62	0,62	80,37	79,53	53,91	0,00
4	4,43	0,23	80,40	90,11	0,03	10,58
5	4,73	0,21	87,05	90,12	6,65	0,01
6	4,95	0,20	91,06	90,13	4,01	0,02

$$T_{digital} \leq T_{exp}$$

$$T_{digital} = 0.7\text{s} > T_{exp} = 0.503\text{s} \quad \text{Condition not verified.}$$

Since the RPA99/ version 2003 requires to use shear walls for a construction that has total height more than 8m in zone III, we opted for this disposition:



**Figure III.3:** Shear wall disposition.

**Table III.8:** Period and mass participation after shear wall disposition.

MODE	FREQUENCY	PERIOD	MASSES CUMULATED UX	MASSES CUMULATED UY	MODAL MASS UX	MODAL MASS UY
1	1,70	0,59	0,02	73,67	0,02	73,67
2	1,77	0,56	73,95	73,69	73,93	0,02
3	2,07	0,48	76,06	73,69	2,11	0,00
4	5,64	0,18	88,22	73,90	12,15	0,20
5	5,72	0,17	88,39	87,20	0,17	13,31
6	7,47	0,13	88,39	87,20	0,00	0,00
7	10,96	0,09	93,90	87,20	5,51	0,00
8	11,96	0,08	93,90	92,94	0,00	5,73

$$T_{exp} = 0.503s$$

**The digital period verification**

$$T_{digital} \leq T_{exp}$$

$$T_{digital} = 0.59s > T_{exp} = 0.503s$$

Condition not verified.

**Interaction justification**

**Vertical loads (ELS)**

Shear walls must support at the most 20% of the vertical loads.

**Table III.9:** Justification of interaction under vertical loads.

LEVEL	VERTICAL LOADS (KN)			PERCENTAGE (%)		OBSERVATION
	structure	Shear walls	Total	structure	Shear walls	
GROUND	12221,29	1482,82	13704,11	89,18	10,82	Verified
LEVEL 1	9762,02	976,51	10738,53	90,91	9,09	Verified
LEVEL 2	7158,37	795,96	7954,33	89,99	10,01	Verified
LEVEL 3	4717,05	497,50	5214,55	90,46	9,54	Verified
LEVEL 4	2292,62	179,34	2471,96	92,75	7,25	Verified

So, the interaction under vertical loads is verified.

**Horizontal loads ( $E_x$ ,  $E_y$ )**

Shear walls must support at the most 75% of horizontal loads:

*In the direction (X-X):*

**Table III.10:** Justification of interaction under horizontal loads (X-X).

LEVEL	HORIZONTAL LOADS (KN)			PERCENTAGE (%)		OBSERVATION
	structure	Shear walls	Total	structure	Shear walls	
GROUND	1147,67	613	1760,67	65,18	34,82	Verified
LEVEL 1	1187,72	471,51	1659,23	71,58	28,42	Verified
LEVEL 2	1022,98	407,91	1430,89	71,49	28,51	Verified
LEVEL 3	828,69	247,15	1075,84	77,03	22,97	Verified
LEVEL 4	636,31	-28,63	607,68	104,71	-4,71	verified

*In the direction (Y-Y):*

**Table III.11:** Justification of interaction under horizontal loads (Y-Y).

LEVEL	HORIZONTAL LOADS (KN)			PERCENTAGE (%)		OBSERVATION
	Structure	Shear walls	Total	structure	Shear walls	
GROUND	800,93	892,19	1693,12	47,30	52,70	Verified
LEVEL 1	812,4	788,52	1600,92	50,75	49,25	Verified
LEVEL 2	703,48	686,06	1389,54	50,63	49,37	Verified
LEVEL 3	573,41	483,78	1057,19	54,24	45,76	Verified
LEVEL 4	537,92	70,84	608,76	88,36	11,64	Verified

So, the interaction under vertical loads is verified.

**Estimation of the seismic effort at the base of the structure**

$$V = \frac{A \times D_{x,y} \times Q}{R} \times W$$

Zone acceleration coefficient A:

Usage Groupe 2  
 Seismic zone III } **A = 0.25**

Dynamic amplification factor D:

$$\xi = 7,00\%$$

$$\eta = \sqrt{7/(2 + \xi)} \geq 0.7$$

$$\eta = 0,88$$

Site type: S3  $T_2 = 0.50s$

$$D = \begin{cases} 2,5\eta & 0 \leq T \leq T_2 \\ 2,5\eta \left(T_2/T\right)^{\frac{2}{3}} & T_2 \leq T \leq 3.0s \\ 2,5\eta \left(T_2/3.0\right)^{\frac{2}{3}} \left(3.0/T\right)^{\frac{5}{3}} & T \geq 3.0s \end{cases}$$

$$D_{x,y} = 2,205$$

Quality factor Q:

$$Q = 1 + \sum_1^6 P_q$$

**Table III.12:** Penalty value.

N°	Q	OBSERVATION		PENALTY
		x-x	y-y	
1	Minimum conditions on bracing lines	Verified	Not verified	0.05
2	Plan redundancy	Not verified	Verified	0.05
3	Regularity in plan	Verified	Verified	0.05
4	Regularity in elevation	Verified	Verified	0.05
5	Materiel quality control	Not verified	Not verified	0.05
6	Execution control	Not verified	Not verified	0.1

$$Q_{x,y} = 1.20$$

Behavior coefficient R:

$$R = 4,00$$

The structure weight:

$$W_i = W_{Gi} + \beta W_{Qi}$$

$\beta$ : weighting coefficient, in our case  $\beta = 0,2$

$$W = 12049,13 \text{ KN}$$

The total seismic effort at the base of the structure is:

$$V_{x,y} = 1992,44 \text{ KN}$$

Now we verify that:  $V_{dynamic} \geq 0,8 V_{static}$

**Table III.13:** Verification of the resulting seismic effort at the base of the structure.

	$V_{dyn}$	$0,8 V_{st}$	OBSERVATION
X-X	1779,46	1593,95	Verified
Y-Y	1918,26	1593,95	Verified

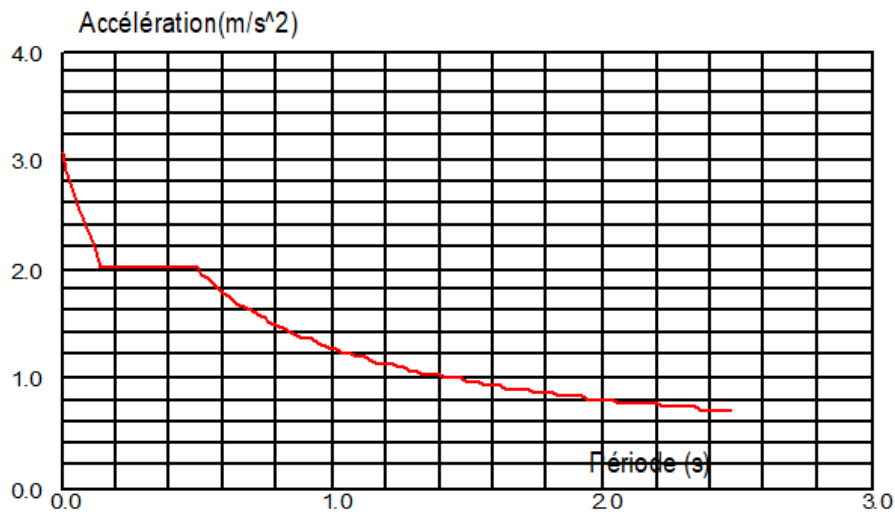


Figure III.4: Response spectrum.

**Reduced normal effort verification**

To verify the reduced normal effort, we need to verify the following formula:

$$\gamma = \frac{N}{f_{c28} \times B} \leq 0,3$$

Table III.14: Verification of the reduced normal effort.

LEVEL	COLUMN SECTION			N (KN)	COMBINATIO N	γ	OBSERVATION
	b (cm)	h (cm)	section (cm <sup>2</sup> )				
GROUND	45	45	2025	1498,08	G + Q + E <sub>x</sub>	0,296	Verified
LEVEL 1	45	45	2025	1246,01	G + Q + E <sub>x</sub>	0,246	Verified
LEVEL 2	40	40	1600	840,91	G + Q + E <sub>x</sub>	0,210	Verified
LEVEL 3	40	40	1600	449,42	G + Q + E <sub>x</sub>	0,112	Verified
LEVEL 4	40	40	1600	160,37	G + Q + E <sub>x</sub>	0,040	Verified

The reduced normal effort is verified for all the floors.

**Shifting verification**

$$\Delta_k = \delta_k - \delta_{k-1}$$

It is necessary that:  $\Delta_k \leq 1\%$  level height.

$\delta_k$ : the horizontal shifting at every floor.

**Table III.15:** Shifting verification in the sense (X-X).

**IN THE DIRECTION X-X**

LEVEL	$\delta_k$ (cm)	$\delta_{k-1}$ (cm)	$\Delta_k$ (cm)	$h_k$ (cm)	$\Delta_k/h_k$	OBSERVATION
<b>GROUND</b>	1,051	0	1,05	306,0	0,343	<i>Verified</i>
<b>LEVEL 1</b>	3,064	1,051	2,01	306,0	0,658	<i>Verified</i>
<b>LEVEL 2</b>	5,085	3,064	2,02	306,0	0,660	<i>Verified</i>
<b>LEVEL 3</b>	6,744	5,085	1,66	306,0	0,542	<i>Verified</i>
<b>LEVEL 4</b>	7,922	6,744	1,18	306,0	0,385	<i>Verified</i>

**Table III.16:** Shifting verification in the sense (Y-Y).

**IN THE DIRECTION Y-Y**

LEVEL	$\delta_k$ (cm)	$\delta_{k-1}$ (cm)	$\Delta_k$ (cm)	$h_k$ (cm)	$\Delta_k/h_k$	OBSERVATION
<b>GROUND</b>	0,977	0	0,98	306,0	0,319	<i>Verified</i>
<b>LEVEL 1</b>	3,006	0,977	2,03	306,0	0,663	<i>Verified</i>
<b>LEVEL 2</b>	5,208	3,006	2,20	306,0	0,720	<i>Verified</i>
<b>LEVEL 3</b>	7,143	5,208	1,94	306,0	0,632	<i>Verified</i>
<b>LEVEL 4</b>	8,722	7,143	1,58	306,0	0,516	<i>Verified</i>

So, the condition is verified.

**P-delta verification**

We need to verify that:  $\theta_k = \frac{P_k \times \Delta_k}{V_k \times h_k} \leq 0,1$

$P_k$ : the total weight of the structure and overloads applied on top of the level.

$V_k$ : shear effort applied on the level.

$\Delta_k$ : the shifting related to the level.

$h_k$ : the level height.

*In the direction X-X:*

**Table III.17:** The P-delta verification in the direction X-X

LEVEL	$h_k$ (CM)	$P_k$ (KN)	$\Delta_k$ (CM)	$V_k$ (KN)	$\theta_k$	OBSERVATION
<b>GROUND</b>	306,0	12049,1	1,05	1760,67	0,024	<i>Verified</i>
<b>LEVEL 1</b>	306,0	9441,9	2,01	1659,23	0,037	<i>Verified</i>
<b>LEVEL 2</b>	306,0	7015,4	2,02	1430,88	0,032	<i>Verified</i>
<b>LEVEL 3</b>	306,0	4633,3	1,66	1075,84	0,023	<i>Verified</i>
<b>LEVEL 4</b>	306,0	2247,7	1,18	607,69	0,014	<i>Verified</i>

**Table III.18:** The P-delta verification in the direction Y-Y

<b>LEVEL</b>	<b><math>h_k</math> (CM)</b>	<b><math>P_k</math> (KN)</b>	<b><math>\Delta_k</math> (CM)</b>	<b><math>V_k</math> (KN)</b>	<b><math>\theta_k</math></b>	<b>OBSERVATION</b>
<b>GROUND</b>	306,0	12049,1	0,98	1693,12	0,023	<b>Verified</b>
<b>LEVEL 1</b>	306,0	9441,9	2,03	1600,92	0,039	<b>Verified</b>
<b>LEVEL 2</b>	306,0	7015,4	2,20	1389,55	0,036	<b>Verified</b>
<b>LEVEL 3</b>	306,0	4633,3	1,94	1057,19	0,028	<b>Verified</b>
<b>LEVEL 4</b>	306,0	2247,7	1,58	608,76	0,019	<b>Verified</b>

c. Conclusion

After several research on the shear walls disposition and on the increase on the sections of the structural elements, and with balancing the resistance standards and the economy standards, we succeeded to satisfy the conditions that are required by the RPA99/version 2003. Which allows us to keep our model and proceed on the calculation of the structure elements.

The final dimensions of the structure are shown by the table below:

**Table III.19:** The final dimensions of the structural elements.

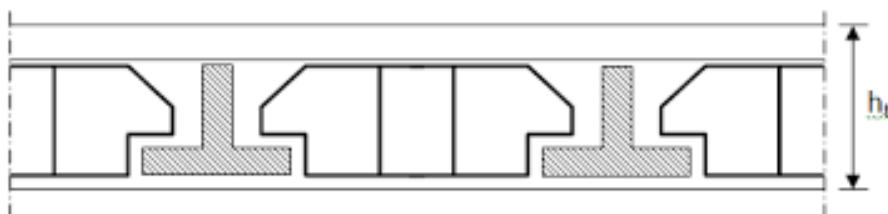
<b>LEVEL</b>	<b>GROUND</b>	<b>LEVEL 1</b>	<b>LEVEL 2</b>	<b>LEVEL 3</b>	<b>LEVEL 4</b>
<b>COLUMN (CM<sup>2</sup>)</b>	45 × 45	45 × 45	40 × 40	40 × 40	40 × 40
<b>SHEAR WALL (CM)</b>			15		
<b>PRIMARY BEAM (CM<sup>2</sup>)</b>			40 × 25		
<b>SECONDARY BEAM (CM<sup>2</sup>)</b>			35 × 25		

**III.5. Secondary elements calculation**

a. Slabs

For any structure, slabs are a horizontal element that supports imposed loads (furniture, vehicles, people ... etc.), and permanent loads (partitions, coatings ...). these loads will be transmitted to beams, columns, and shear walls. In the end all these efforts are transmitted to foundations.

**Hardy slabs:**



**Figure III.5:** Transversal cut on hardy slab.

This slab will consist of a hollow core (without a role only for filling, it has no function of resistance), of joists (these are the resistant elements of slabs), of a compression table (reinforced concrete slab its height varies from 4 to 6cm) and a mesh reinforcement.



We adopted for our hardy slab a height of 20cm:

$$\text{With } \left\{ \begin{array}{l} h_{hc} = 16\text{cm} : \text{height of the hollow core.} \\ h_{ct} = 4\text{cm} : \text{height of the compression table.} \end{array} \right.$$

Compression table:

The compression table is armed with a mesh reinforcement type TLE500, with a yield strength of  $f_e=500\text{MPa}$ , the meshes dimensions should not go past the following values, given by the BAEL 91:

20cm for reinforcements that are perpendicular on the joists.

30cm for reinforcements that are parallel on the joists.

*Reinforcements that are perpendicular on the joists:*

$$A_{\perp} = \frac{4L}{f_e}$$

L: distance between joists axes (L=55cm).

$$A_{\perp} = \frac{4 \times 55}{500} = 0.44\text{cm}^2$$

$$A_{\perp} = \mathbf{6HA5 = 1,17\text{cm}^2/ml} \quad \text{with } e = \mathbf{15\text{cm}}$$

*Reinforcements that are parallel on the joists:*

$$A_{\parallel} = \frac{A_{\perp}}{2} = \frac{1.17}{2} = 0.58\text{cm}^2$$

$$A_{\parallel} = \mathbf{6HA5 = 1,17\text{cm}^2/ml} \quad \text{with } e = \mathbf{15\text{cm}}$$

**Conclusion:** we will adopt for the reinforcement of the compression table a mesh reinforcement (TLE500) of dimension (6x6x150x150) mm<sup>2</sup>.

Joists calculation:

joists are considered uniformly loaded and will be calculated in two steps:

- *Before casting the compression table:*

The joist is considered as simply supported on its two ends. It has to support its propre weight, and the hollow core weight that is 0.95KN/m<sup>2</sup>, and the imposed overload.

- Propre weight:  $G_1 = (0,04 \times 0,12) \times 25 = 0,12 \text{ KN/ml}$
- Hollow core weight:  $G_2 = 0,92 \times 0,65 = 0,62 \text{ KN/ml}$
- Imposed overload:  $Q = 1 \text{ KN/ml}$

**Reinforcement in the ultimate limit state (ULS):**

the calculation will be made for the most unfavorable span:

$$L_{max} = 4 \text{ m}$$

**Load combination:**

$$q_u = 1,35 G + 1.5Q$$

$$\text{With } G = G_1 + G_2$$

$$q_u = 1,35 \times (0,12 + 0,62) + 1,5 \times 1 = 2,5 \text{ KN/ml}$$

**Max moment in span:**

$$M_u = \frac{q_u \times l^2}{8} = \frac{2,5 \times 4^2}{8} = 5 \text{ KN.m}$$

**Max transverse shear force:**

$$T = \frac{q_u \times l}{2} = \frac{2,5 \times 4}{2} = 5 \text{ KN}$$

**Calculation of reinforcement:**

Reinforcement cover:  $c = 2 \text{ cm}$

Useful height  $d = h - c = 2 \text{ cm}$

$$\mu_u = \frac{M_u}{bd^2f_{bc}} = \frac{5}{0,12 \times 0,02^2 \times 14,16 \times 10^3} = 7,35$$

$\mu_u > \mu_1 = 0.392 \rightarrow$  the section is double armed.

**Conclusion:** the joist height does not allow us to have two layers of reinforcements. Therefore, it is necessary to provide a low flow to help the joist to support the loads.

- *After casting the compression table:*

The joist will be calculated as a beam in section T based on several supports. The loads and overloads are considered evenly distributed over all the joists.

**The width of the compression table:**

the width of the hollow core to be taken into account for each dimension of a rib, is limited by the most restrictive of the following values:

- $b_1 \leq \frac{l-b_0}{2} = \frac{65-12}{2} = 26,5 \text{ cm}$
- $b_1 \leq \frac{l_1}{10} = \frac{400}{10} = 40 \text{ cm}$
- $b_1 \leq \frac{2}{3} \times \frac{l_1}{2} = \frac{2}{3} \times \frac{400}{2} = 133,33 \text{ cm}$

With:  $l$ : distance between joists axes.

$l_1$ : span length.

$b$ : width of the compression table to be taken into account in the calculation.

$b_0$ : rib width ( $b_0 = 12 \text{ cm}$ ).

$h_0$ : thickness of the compression table ( $h_0 = 4 \text{ cm}$ )

We take:  $b_1 = 26,5 \text{ cm}$ ,  $b = 2 \times b_1 + b_0 = 2 \times 26,5 + 12 = 65 \text{ cm}$

**Verification of the conditions for applying the inclusive method:**

Application domain: this method is applied to slabs with moderated overloads, the imposed overloads are at most equal to two times the permanent load or  $5 \text{ KN/m}^2$ . meaning:

$$\begin{aligned} \text{Current floor slab: } G &= 5,18 \text{ KN/m}^2 ; & G &= 5,18 \times 0,65 = 3,37 \text{ KN/ml} \\ Q &= 1,5 \text{ KN/m}^2 ; & Q &= 5 \times 0,65 = 3,25 \text{ KN/ml} \end{aligned}$$

Combination of action:

$$\text{ULS: } Q_u = 1,35G + 1,5Q = 9,42 \text{ KN/ml}$$

$$\text{SLS: } Q_s = G + Q = 6,56 \text{ KN/ml}$$

Method choice:

$$1) \quad \begin{cases} Q = 5 \text{ KN/ml} < 2G = 6,74 \text{ KN/ml} \\ Q \leq 5 \text{ KN} \end{cases}$$

**Condition verified.**

2) The moments of inertia of the transversal sections are the same in different continuous spans. **Condition verified.**

3) The successive clear spans are in a ratio included between 0,8 and 1,25:

$$0,8 \leq \frac{l_i}{l_{i+1}} \leq 1,25$$

$$\frac{350}{340} = 1,02; \quad \frac{340}{400} = 0,85; \quad \frac{305}{400} = 0,80; \quad \frac{400}{340} = 1,17; \quad \frac{340}{350} = 0,97$$

**Condition verified.**

4) The cracking is considered non-detrimental. **Condition verified.**

Since all the conditions are verified, we conclude that the inclusive method is relevant.

Exposition of the method:

the report  $\alpha$  of the imposed overloads on the sum of the permanent loads and imposed loads in unweighted values  $\alpha = \frac{Q}{Q+G}$ , varies from 0 to 2/3 for a slab of a moderated imposed overloads.

In effect for  $Q = 0 \rightarrow \alpha = 0$  and for  $Q = 2G \rightarrow \alpha = 2/3$

$M_0$ : the maximum value of the bending moment in the span between supports.

$$M_0 = \frac{q \times l^2}{8}$$

With:  $l$ : the length of the span between supports.

$q$ : evenly distributed load.

$M_w$  and  $M_e$ : are the values of the moments on the supports from the left and from the right respectively.

$M_t$ : maximum moment in the span, taken into account in the calculations of the considered span

The values of  $M_e$ ,  $M_w$  and  $M_t$  should verify the following conditions:

$$M_t \geq -\frac{M_e + M_w}{2} + \max[1,05M_0; (1 + 0,3\alpha)M_0]$$

$$M_t \geq \frac{1+0,3\alpha}{2} M_0 \quad \text{in the case of intermediate span.}$$

$$M_t \geq \frac{1,2+0,3\alpha}{2} M_0 \quad \text{in the case of edge span.}$$

The absolute value of every moment on an intermediate support should be at least equal to:

$0,6M_0$  in the case of a beam of two spans.

$0,5M_0$  for the neighbor edge supports in the case of a beam of more than two spans.

$0,4M_0$  for the other intermediate supports in the case of a beam of more than 3 spans.

$0,3M_0$  for the edge supports that are half restrained.

In our case we have a beam on 8 supports as represented in the following diagram:

Application:

- Loads combination:

$$G = 5,18 \times 0,65 = 3,37 \text{KN/ml}$$

$$Q = 1,5 \times 0,65 = 0,975 \text{KN/ml}$$

$$\text{ULS: } q_u = 1,35G + 1,5Q = 6,01 \text{KN/ml}$$

$$\text{SLS: } q_s = G + Q = 4,34 \text{KN/ml}$$

Calculation of the load report:

$$\alpha = \frac{0,975}{0,975 + 3,31} = 0,227 < 0,67$$

We will be needing for our calculation the following values:

**Table III.20:** The values of the load report.

	INTERMEDIATE SPAN	EDGE SPAN
$\alpha$	$1 + 0,3\alpha$	$1,2 + 0,3\alpha/2$
<b>0,227</b>	1,068	0,634

Calculation of isostatic moment:

in span:  $M_0 = \frac{q \times l^2}{8}$

**Table III.21:** The values of the isostatic moment in span.

SPAN	1-2	2-3	3-4	4-5	5-6	6-7	7-8
<b>L (M)</b>	3,5	3,4	4	3,05	4	3,4	3,5
<b>M<sub>0</sub> (KN.m)</b>	9,20	8,68	12,02	6,99	12,02	8,68	9,20

At the supports:  $M_{support} = \beta M_{0max}$

**Table III.22:** The values of the moment at the supports.

SUPPORT	1	2	3	4	5	6	7	8
<b>INCLUSIVE COEFFICIENT</b>	0,3	0,5	0,4	0,4	0,4	0,4	0,5	0,3
<b>M<sub>support</sub> (KN.m)</b>	2,76	4,6	4,81	4,81	4,81	4,81	4,6	2,76

Edge span 1-2:

$$M_t \geq -\frac{M_e + M_w}{2} + \max[1,05M_0; (1 + 0,3\alpha)M_0]$$

$$1,05M_0 = 9,66KN.m \quad ; \quad (1 + 0,3\alpha)M_0 = 9,82KN.m$$

$$M_t \geq -\frac{2,76 + 4,6}{2} + 9,82 = 6,14 KN.m$$

$$M_t \geq \frac{1,2 + 0,3\alpha}{2} M_0$$

$$M_t \geq 0,634 \times 9,20 = 5,83$$

$$M_t \geq 6,14 KN.m$$

**Table III.23:** The values of the maximum moment in span.

SPAN	1-2	2-3	3-4	4-5	5-6	6-7	7-8
<b>M<sub>t</sub> (KN.m)</b>	6,14	4,63	8,02	3,73	8,02	4,63	6,14

Calculation of transverse shear force:

we have:  $V_i = \pm q \times \frac{l}{2} - \frac{M_w - M_e}{l}$

$$\text{span 1-2: } \begin{cases} V_1 = 6,01 \times \frac{3,5}{2} - \frac{2,76-4,6}{3,5} = 11,04 \text{ KN} \\ V_2 = -6,01 \times \frac{3,5}{2} - \frac{2,76-4,6}{3,5} = -9,99 \text{ KN} \end{cases}$$

**Table III.24:** Values of transverse shear force.

SPAN	1-2	2-3	3-4	4-5	5-6	6-7	7-8
$V_w$	11,04	10,28	12,02	9,16	12,02	10,16	9,99
$V_e$	-9,99	-10,16	-12,02	-9,16	-12,02	-10,28	-11,04

Reinforcement in ULS:

- *Longitudinal reinforcement:*

The reinforcement will be done considering the maximum moment:

In span:  $M_u = 8,02 \text{ KN.m}$

On supports:  $M_u = 4,81 \text{ KN.m}$

**In span:**

If  $M_{t \max} > M_{table} \rightarrow$  the neutral axe is in the rib.

If  $M_{t \max} > M_{table} \rightarrow$  the neutral axe is in the compression table.

Balance moment by the compression table:

$$M_{table} = b \cdot h_0 \cdot \left(d - \frac{h_0}{2}\right) \cdot f_{bc}$$

$$M_0 = 0,65 \times 0,04 \times \left(0,18 - \frac{0,04}{2}\right) \times 14,16 \times 10^3 = 58,91 \text{ KN.m}$$

$$M_0 = 58,91 \text{ KN.m} > M_{t \max} = 8,02 \text{ KN.m}$$

So, the neutral axe is in the compression table.

The calculation will be made as if for a rectangular section b x h (65x20) cm.

$$\mu = \frac{M_u}{b \times d^2 \times f_{bu}} = \frac{8,02 \times 10^3}{65 \times 18^2 \times 14,16} = 0,0269$$

$$\mu = 0,0269 < \mu_1 = 0,371 \rightarrow \text{tensioned reinforcements only}$$

$$\alpha = 1,25(1 - \sqrt{1 - 2\mu_u}) = 0,034$$

$$Z = d(1 - 0,4\alpha) = 17,76 \text{ cm}$$

$$A_t = \frac{M_u}{Z \times \sigma_{st}} = \frac{8,02 \times 10^3}{17,76 \times 435} = 1,038 \text{ cm}^2$$

$$\text{We take: } A_t = \mathbf{3HA10} = \mathbf{2,36 \text{ cm}^2}$$

**On the supports:**

The compression table is at the tense part of the section, so we will be studying a rectangular section of (12x20) cm<sup>2</sup>.

$M_u = 4,81 \text{ KN.m}$

$$\mu = \frac{M_u}{b_0 \times d^2 \times f_{bc}} = \frac{4,81 \times 10^3}{12 \times 18^2 \times 14,16} = 0,087$$

$$\mu = 0,087 < \mu_1 = 0,371 \rightarrow \text{tensioned reinforcements only}$$

$$\alpha = 1,25(1 - \sqrt{1 - 2\mu_u}) = 0,113$$

$$Z = d(1 - 0,4\alpha) = 17,18 \text{ cm}$$

$$A_t = \frac{M_u}{Z \times \sigma_{st}} = \frac{4,81 \times 10^3}{17,18 \times 435} = 0,64 \text{ cm}^2$$

$$\text{We take: } A_a = \mathbf{3HA10} = \mathbf{2,36 \text{ cm}^2}$$

- *transverse reinforcements:*  
the minimal diameter of the transverse reinforcements should not surpass one of the following values:

$$\varphi_t \leq \min \left\{ \frac{h}{35}; \varphi_l; \frac{b_0}{10} \right\}$$

With:  $h$ : the total height of the beam.

$b_0$ : the width of the rib.

$\varphi_l$ : the minimal diameter for the longitudinal reinforcement.

$$\text{So: } \varphi_t \leq \min \left\{ \frac{200}{35}; 10; \frac{120}{10} \right\} = \min\{0.57; 10; 12\} = 0,57\text{cm}$$

We take:  $\varphi_t = \mathbf{6mm}$

The spacing between the transverse reinforcements is given by the code (BAEL 91 modified 99/art: A5.1.22).

$$S_t \leq \min(0,9d; 40\text{cm})$$

$$S_t \leq 16,2\text{cm}$$

We take:  $S_t = \mathbf{15cm}$

- *Anchoring of reinforcements (sealing length):*

(BAEL 91 modified 99/art: A5.1.22).

It represents the length of the steel anchored in the concrete so that the tensile stress or the compressive stress requested in the bar could be mobilized.

$$L_s = \frac{\varnothing \times f_e}{4 \times \tau_s} = \frac{1,00 \times 500}{4 \times 2,83} = 44,16\text{cm}$$

The code of BAEL 91 (A.6.1.21) admits that the anchoring of a straight bar ending by a normal hook is assured once the anchored span measured without the hook " $L_c$ " is at least equal to " $0,4L_s$ " for the steel H.A.

$$\text{So: } L_c = 0,4L_s = 0,4 \times 44,16$$

$$L_c = 17,66\text{cm}$$

verifications in ULS:

**condition of non-fragility:** (BAEL91/ A4.2.1).

$$A_{min} = \frac{0,23b.d.f_{t28}}{f_e}$$

**In span:**

$$A_{min} = \frac{0,23 \times 12 \times 18 \times 2,1}{500} = 0,207\text{cm}^2$$

$$A_t = 2,36\text{cm}^2 > A_{min} = 0,207\text{cm}^2 \quad \mathbf{\text{Condition verified}}$$

**On the supports:**

$$A_{min} = \frac{0,23 \times 12 \times 18 \times 2,1}{500} = 0,207\text{cm}^2$$

$$A_t = 1,57\text{cm}^2 > A_{min} = 0,207\text{cm}^2 \quad \mathbf{\text{Condition verified}}$$

**Verification of the transverse shear force: (BAEL91/ A.5.1)**

$$\tau_u^{max} = \frac{T_u^{max}}{b_0 d} \leq \bar{\tau}_u$$

With:  $T_u = 12,02 \text{ KN}$

$$\tau_u^{max} = \frac{12,02 \times 10}{12 \times 18} = 0,56 \text{ MPA}$$

$$\bar{\tau}_u = \min \left\{ \frac{0,2 f_{c28}}{\gamma_b}; 5 \text{ MPA} \right\} = \min \{ 3,33 \text{ MPA}; 5 \text{ MPA} \}$$

$\tau_u < \bar{\tau}_u \rightarrow$  **condition verified**

**Bond stress verification:**

We need to verify that:

$$\tau_{se} = \frac{T_u^{max}}{0,9 d \sum U_i} \leq \bar{\tau}_{se}$$

With:  $\sum U_i$ : the sum of useful perimeters of bars.

$$\tau_{se} = \frac{12,02 \times 10^3}{0,9 \times 180 \times 94,2} = 0,78 \text{ MPA}$$

$$\bar{\tau}_{se} = \Psi_s \cdot f_{t28} = 1,5 \times 2,1 = 3,15 \text{ MPA}$$

$\tau_{se} = 0,78 \text{ MPA} \leq \bar{\tau}_{se} = 3,15 \text{ MPA}$  **Condition verified.**

**The influence of the shear force on concrete:**

(BAEL91 modified 99/Art: A5.3.313)

$$T_u < 0,4 \frac{f_{c28}}{\gamma_b} 0,9 d b_0$$

$$T_u < 0,4 \times \frac{25}{1,5} \times 0,9 \times 18 \times 12 = 129,6 \text{ KN}$$

Edge supports:

$$T_u^{max} = 11,04 < 129,6 \text{ KN} \rightarrow \text{condition verified.}$$

Intermediate supports:

$$T_u^{max} = 12,02 < 129,6 \text{ KN} \rightarrow \text{condition verified.}$$

**The influence of the shear force on the reinforcements:**

we need to verify that:

$$A \geq \frac{\gamma_s}{f_e} \left( T_u^{max} + \frac{M_{max}}{0,9 d} \right)$$

Edge support:

$$A = 1,57 \geq \frac{1,15}{500 \times 10^{-1}} \left( 11,04 - \frac{6,14}{0,9 \times 0,18} \right) = -0,62 \text{ cm}^2$$

Condition verified

Intermediate supports:

$$A = 1,57 \geq \frac{1,15}{500 \times 10^{-1}} \left( 12,02 - \frac{8,02}{0,9 \times 0,18} \right) = -0,86 \text{ cm}^2$$

**Condition verified.**

Calculation in SLS:

**Bending moment and shear force in SLS:**

When the load is the same on different spans the BAEL (A.6.5.1) specifies that the multiplication of the calculation results in ULS by the coefficient  $q_s/q_u$  gives us the values of the internal efforts of the calculation in SLS. The values of the internal efforts are represented in the following table.

$$\frac{q_s}{q_u} = \frac{4,34}{6,01} = 0,72$$

**Table III.25:** Values of shear effort and moments in SLS.

SPAN	1-2	2-3	3-4	4-5	5-6	6-7	7-8
$M_w$	1,98	3,22	3,37	3,37	3,37	3,37	3,22
$M_e$	3,22	3,37	3,37	3,37	3,37	3,22	1,98
$V_w$	7,72	7,2	8,41	6,41	8,41	7,11	6,99
$V_e$	-6,99	-7,11	-8,41	-6,41	-8,41	-7,2	-7,72
$M_t$	4,2	3,24	5,61	2,61	5,61	3,24	4,2

Verification in SLS:

**Limit state of crack openings:** (BAEL91 modified 99/ A.4.5.3)

we have a little damaging cracking so no verification is necessary.

**Limit state of resistance to compression:** (BAEL91 modified99/ A.4.5.2)

We need to verify that:

$$\sigma_{bc} = \frac{M_{ser}}{I} y \leq \bar{\sigma}_{bc} = 0,6 \times f_{c28} = 15MPa$$

With:  $H = \frac{b \times h_0}{2} - 15A(d - h_0)$

$$\frac{b_0}{2} \times y^2 + [(b - b_0) \times h_0 + 15A] \times y - [(b - b_0) \times \frac{h_0^2}{2} + 15A \cdot d] = 0$$

$$I = \frac{b \times y^3}{3} - \frac{(b - b_0)}{3} \times (y - h_0)^3 + 15A \times (d - y)^2$$

**Table III.26:** Limit state of resistance to compression verification.

	$H (cm^3)$	$y (cm)$	$I (cm^4)$	$\sigma_{bc} (MPa)$	$\bar{\sigma}_{bc} (MPa)$
<b>IN SPAN</b>	-167,98	3,92	8323,05	2,64	15
<b>AT THE SUPPORTS</b>	-471,6	7,75	13804,71	4,35	15

So, the condition is verified.

**Limit state of deformation:** (BAEL91 modified99/ Art.6.5.2)

The arrow developed in the joist has to stay smaller than the eligible arrow so as not to affect the appearance and use of the construction.

**Calculation of the arrow:**

According to BAEL91, it is possible to justify the arrow if the following conditions are verified:

$$\frac{h}{L} \geq \frac{1}{22,5}$$



$$\frac{h}{L} \geq \frac{M_t}{15M_0}$$

$$\frac{A_s}{b_0 d} \leq \frac{3,6}{f_e}$$

With:  $h$ : the total height of the section.

$L$ : the maximum span length.

$M_t$ : maximum bending moment.

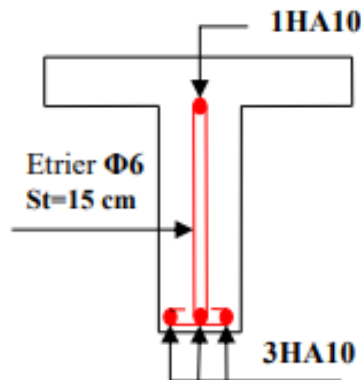
$b_0$ : the rib width.

$$\frac{h}{L} = \frac{20}{400} = 0,05 \geq \frac{1}{22,5} = 0,044 \rightarrow \text{condition verified.}$$

$$\frac{h}{L} = 0,05 \geq \frac{M_t}{15M_0} = \frac{5,61}{15 \times 8,7} = 0,04 \rightarrow \text{condition verified.}$$

$$\frac{A_s}{b_0 d} = \frac{1,57}{18 \times 12} = 0,007 \leq \frac{3,6}{500} = 0,0072 \rightarrow \text{condition verified.}$$

All the conditions are verified, therefor the calculation of the arrow is not necessary.



**Figure III.6:** Joists reinforcements in span and on supports.

#### **Flat slab (balcony):**

The balcony we are studying is a flat slab balcony resting on an edge beam. The calculation will be done for a strip 1m wide.

The flat slab's thickness is already calculated:  $e_p = 15\text{cm}$

#### Loads and overloads:

Permanent load:  $G = 6,43 \text{ KN/m}^2$

Imposed overload:  $Q = 3,5 \text{ KN/m}^2$

Balcony rail:  $G_2 = 2,52 \text{ KN/ml}$

#### Combinations:

- **for ULS:**

$$q_u = (1,35G + 1,5Q) \times 1\text{m} = 13,93 \text{ KN/ml}$$

$$p_u = 1,35G_2 \times 1 = 3,4 \text{ KN}$$

- **for SLS:**

$$q_s = (G + Q) \times 1\text{m} = 9,93 \text{ KN.ml}$$

$$p_s = G_2 \times 1 = 2,52 \text{ KN/ml}$$

Moments and transverse shear force:

**ULS:**

$$M_u = \frac{q_u \times l^2}{2} + p_u \times l = \frac{13,93 \times 1,35}{2} + 3,4 \times 1,35$$

$$M_u = \mathbf{13,99 \text{ KN.m}}$$

$$V_u = q_u \times l + p_u = 13,93 \times 1,35 + 3,5$$

$$V_u = \mathbf{22,30 \text{ KN}}$$

**SLS:**

$$M_s = \frac{q_s \times l^2}{2} + p_s \times l = \frac{9,93 \times 1,35}{2} + 2,52 \times 1,35$$

$$M_s = \mathbf{10,10 \text{ KN.m}}$$

$$V_s = q_s \times l + p_s = 9,93 \times 1,35 + 2,52$$

$$V_s = \mathbf{13,4 \text{ KN}}$$

Reinforcement:

The reinforcement will be calculated in simple bending.

$c \geq 2\text{cm}$  we take  $c = 2\text{cm}$  which will give us  $d = e - c = 15 - 2 = 13\text{cm}$

$$\mu_{bu} = \frac{13,99 \times 10^{-3}}{1 \times 0,13^2 \times 14,16} = 0,058 \leq \mu_R = 0,371 \rightarrow A' = 0$$

$$\alpha = 1,25(1 - \sqrt{1 - 2\mu_u}) = 0,075$$

$$z = d(1 - 0,4\alpha) = 12,6\text{cm}$$

$$A_t = \frac{13,99 \times 10^3}{12,6 \times 435} = 2,55\text{cm}^2$$

We take  $A = \mathbf{4HA12} = \mathbf{4,52\text{cm}^2}$

$$S_t = \frac{100}{5} = 20\text{cm}$$

Distribution reinforcement:

$$A_r = \frac{A_s}{4} = \frac{4,52}{4} = 1,13\text{cm}^2$$

We take  $A = \mathbf{4HA8} = \mathbf{2,01\text{cm}^2}$

$$S_t = \frac{100}{5} = 20\text{cm}$$

Verifications in ULS:

**Condition of non-fragility:**

$$A_{min} = 0,23b \times d \times \frac{f_{t28}}{f_e} = \frac{0,23 \times 100 \times 13 \times 2,1}{500} = 1,25\text{cm}^2$$

$$A_t = 4,52\text{cm}^2 > A_{min} = 1,25\text{cm}^2 \rightarrow \text{condition verified.}$$

**Checking shear effort:**

$$\tau_u = \frac{V_u}{b \times d} \leq \bar{\tau}_u$$

$$\tau_u = \frac{22,3 \times 10^3}{1000 \times 130} = 0,171 \text{ MPa}$$

$$\bar{\tau}_u = \min \left\{ \frac{0,2f_{c28}}{\gamma_b}; 5\text{MPa} \right\} = 3,33\text{MPa}$$

$$\tau_u = 0,171 \text{ MPa} < \bar{\tau}_u = 3,33 \text{ MPa} \rightarrow \text{condition verified.}$$

The transversal reinforcements are not necessary.

**checking the adhesion of the bars:**

$$\tau_{se} = \frac{V_u}{0,9 \times d \times \sum U_i} \leq \bar{\tau}_{se}$$

$$\sum U_i = n \times \pi \times \Phi = 4 \times 3,14 \times 12 = 150,72 \text{ mm}$$

$$\bar{\tau}_{se} = \psi \times f_{t28} = 1,5 \times 2,1 = 3,14 \text{ MPa}$$

$$\tau_{se} = \frac{22,3 \times 10^3}{0,9 \times 13 \times 150,72} = 1,26 \text{ MPa}$$

$$\tau_{se} = 1,14 \text{ MPa} < \bar{\tau}_{se} = 3,15 \text{ MPa} \rightarrow \text{condition verified.}$$

Verification in SLS:

$$\sigma_{bc} = \frac{M_{ser}}{I} y \leq \sigma_{bc}^{adm} = 0,6 f_{c28}$$

$$\frac{b}{3} y^2 + 15Ay - 15Ad = 0$$

$$33,33y^2 + 67,8y - 881,4 = 0$$

$$\mathbf{y = 4,22 \text{ cm}}$$

Moment of inertia:

$$I = \frac{b}{3} y^3 + 15A(d - y)^2$$

$$I = 7731,39 \text{ cm}^4$$

$$\sigma_{bc} = \frac{10,10 \times 10^3}{7731,39} \times 4,22 = 5,51$$

$$\sigma_{bc}^{adm} = 0,6 f_{c28} = 15 \text{ MPa}$$

$$\sigma_{bc} = 5,51 \leq \sigma_{bc}^{adm} = 15 \text{ MPa} \rightarrow \text{condition verified.}$$

**steel stress verification:**

$$\sigma_{st} = \frac{15 \times M_{ser}}{I} \times (d - y) \leq \sigma_{st}^{adm} = \min \left[ \frac{2}{3} f_e; \left( 110 \sqrt{\eta \times f_{tj}} \right) \right]$$

With  $\eta = 1,6$  (HA) and  $f_{t28} = 2,1 \text{ MPa}$

$$\sigma_{st}^{adm} = 333,33 \text{ MPa}$$

$$\sigma_{st} = \frac{15 \times 10,10 \times 10^3}{7731,39} \times (13 - 4,22) = 172,05 \text{ MPa}$$

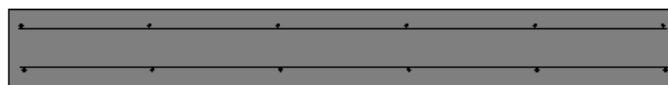
$$\sigma_{st} = 172,05 \text{ MPa} \leq \sigma_{st}^{adm} = 333,33 \text{ MPa} \rightarrow \text{condition verified.}$$

**Arrow verification:**

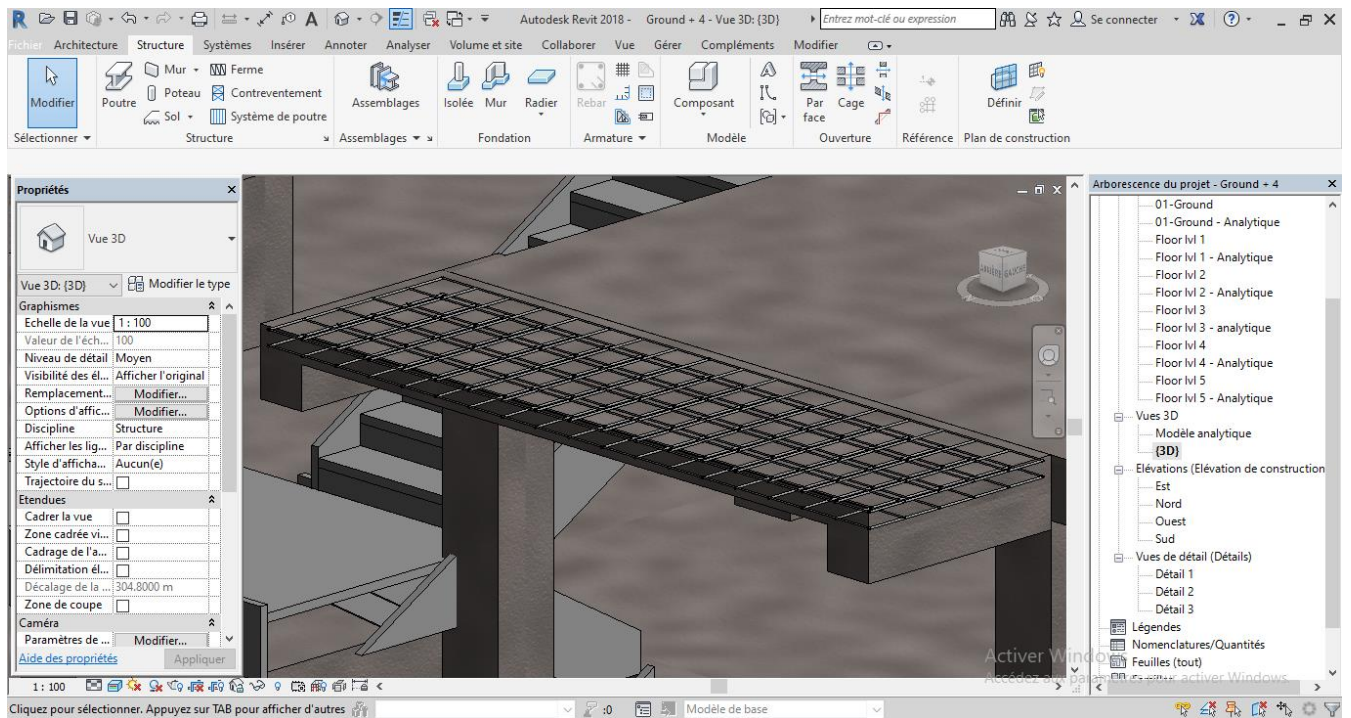
$$\frac{h}{L} \geq \frac{1}{22,5} \rightarrow \frac{15}{135} = 0,111 \geq \frac{1}{22,5} = 0,044$$

$$\frac{A_s}{bd} \leq \frac{3,6}{f_e} \rightarrow \frac{4,52}{1000 \times 13} = 0,0003 \leq \frac{3,6}{500} = 0,0072$$

**Condition verified.**



**Figure III.7:** Transversal cut on reinforced flat slab.

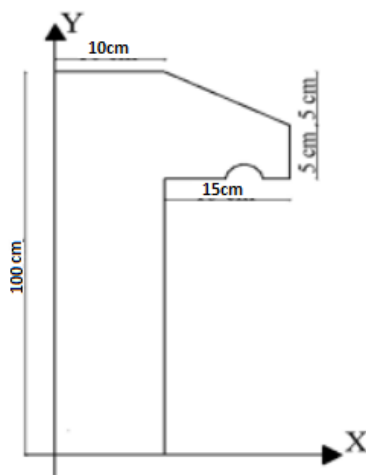


**Figure III.8:** 3D view on reinforced flat slab.

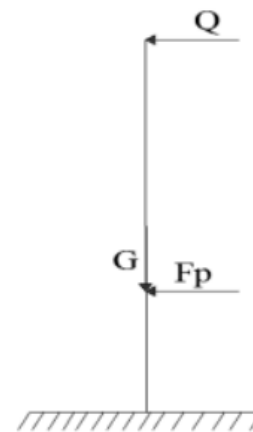
**b. Acroterion**

The acroterion is a security element at the terrace, it forms a wall against any fall. It is considered as a built-in console, subjected to its propre weight ( $G$ ), an horizontal overload due to the handrail ( $Q$ ) and a seismic force ( $F_p$ ).

The calculation will be made in compound bending at the embedding section for a strip of one linear meter. The dimensions of the acroterion are given below.



**Figure III.9:** Acroterion transversal cut.



**Figure III.10:** Acroterion calculation model.

**Loads and overloads evaluation:**

*Propre weight:*

$$G_0 = 25 \times S_{acr} = 25 \times 0,06125 = 1,53 \text{ KN}$$

$$G_{coating} = 18 \times 0,03 = 0,54 \text{ KN}$$

From which the total load is:  $G = W_p = 1,53 + 0,54 = 2,07 \text{ KN}$

*Imposed overload:*

$$Q = 1 \text{ KN}$$

*Seismic force:*

The seismic force is calculated by the following formula:

$$F_p = 4 \times A \times C_p \times W_p \quad \text{RPA99 (Art 6.2.3)}$$

With:  $\begin{cases} A = 0,25 \text{ (zone III): zone acceleration coefficient.} \\ C_p = 0,8 \text{ horizontal force factor.} \\ W_p = 2,07 \text{ KN : the acroterion propre weight.} \end{cases}$

$$F_p = 1,66 \text{ KN}$$

*Center of gravity:*

$$x_G = \frac{\sum A_i \cdot x_i}{\sum A_i} = 7,14 \text{ cm}$$

$$y_G = \frac{\sum A_i \cdot y_i}{\sum A_i} = 28,47 \text{ cm}$$

The acroterion is subjected to:

$$N_G = 2,07 \text{ KN}$$

$$M_G = 0 \text{ KN.m}$$

$$N_Q = 0 \text{ KN}$$

$$M_Q = Q \times h = 0,5 \text{ KN.m}$$

$$N_{Fp} = 0 \text{ KN}$$

$$M_{Fp} = F_p \times y_G = 0,46 \text{ KN.m}$$

**Table III.27:** Acroterion solicitations.

COMBINATION SOLICITATIONS	ALS	ULS	SLS
	$G + Q + E$	$1,35G + 1,5Q$	$G + Q$
$N \text{ (KN)}$	2,07	2,79	2,07
$M \text{ (KN.m)}$	0,96	0,75	0,5

**Acroterion study in ULS:**

$$\text{Eccentricity: } e_0 = \frac{M_u}{N_u} = \frac{0,75}{2,79} = 0,2688 \text{ m}$$

$$\frac{h}{6} = 0,083 \text{ m}$$

the center of pressure is outside the central core, which means that the section is partially compressed, moreover N is a compressive force so the reinforcement is done by assimilation to simple bending subjected to a moment  $M_u = N_u \times e$ .

According to [2] (Art 4.4), the section is subjected to a normal compressive effort, it has to justify to the ultimate limit state of form stability (buckling).

So, the risk of buckling leads us to replace ( $e_0$ ) by ( $e$ ) such as:

$$e = e_0 + e_a + e_2$$

With:

$e_a$ : additional eccentricity reflecting the initial geometric imperfections.

$e_2$ : eccentricity due to second order effects, related to the deformation of the structure.

$$e_a = \max(2cm; h/250) = 2cm$$

$$e_2 = \frac{3 \times l_f^2 \times (2 + \emptyset \times \alpha)}{10^4 \times h_0}$$

$\emptyset$ : final deformation report due to the creep at instantaneous deformation under the load considered.

$$\alpha = \frac{M_G}{M_G + M_Q} = \frac{0}{0 + 0,5} = 0$$

$$l_f = 2 \times h = 1m : \text{buckling length.}$$

$$e_2 = 0,006m$$

$$e = 0,2688 + 0,02 + 0,006 = 0,2948m$$

The calculation solicitations become:

$$N_u = 2,79 KN.$$

$$M_u = 2,79 \times 0,2948 = 0,82 KN.m$$

### ***The acroterion reinforcement:***

Calculation in ULS:

$\frac{h}{6} < e_0 \rightarrow$  the section is partially compressed and  $e_0$  is out of the section, so the calculation is done by assimilation to simple bending with  $M_f$ :

$$M_f = M_u + N_u \left( d - \frac{h}{2} \right) = 0,9037 KN.m$$

$$\mu_{bu} = \frac{M_f}{f_{bu} \times b \times d^2} = \frac{0,9037 \times 10^{-3}}{14,16 \times 1 \times 0,08^2} = 0,0997 < \mu_R = 0,371 \rightarrow A' = 0$$

$$\alpha = 1,25 \left[ 1 - \sqrt{1 - 2\mu_{bu}} \right] = 0,131$$

$$z = d(1 - 0,4\alpha) = 0,07m$$

$$A_1 = \frac{M_f}{z \times \sigma_s} = 0,29cm^2$$

Back to the compressed bending:

$$A = A_1 - \frac{N_u}{\sigma_s} = 0,22 cm^2/ml$$

### ***Acroterion study in ALS:***

Eccentricity calculation:

$$e_0 = \frac{M_{ALS}}{N_{ALS}} = \frac{0,96}{2,07} = 0,4637m$$

$$\frac{h}{6} = 0,083m$$

$$e_0 > h/6$$

The center of pression is outside of the central core.

$$e = e_0 + e_a + e_2$$

$$e = 0,4897m$$

$$N_u = 2,07KN$$

$$M_u = 2,07 \times 0,4897 = 1,01 KN.m$$

**The acroterion reinforcement:**

Calculation in ALS:

$\frac{h}{6} < e_0 \rightarrow$  the section is partially compressed and  $e_0$  is out of the section, so the calculation is done by assimilation to simple bending with  $M_f$ :

$$M_f = M_u + N_u \left( d - \frac{h}{2} \right) = 1,072 \text{ KN.m}$$

$$\mu_{bu} = \frac{M_f}{f_{bu} \times b \times d^2} = \frac{1,072 \times 10^{-3}}{18,48 \times 1 \times 0,08^2} = 0,0906 < \mu_R = 0,371 \rightarrow A' = 0$$

$$\alpha = 1,25 \left[ 1 - \sqrt{1 - 2\mu_{bu}} \right] = 0,11$$

$$z = d(1 - 0,4\alpha) = 0,076 \text{ m}$$

$$A_1 = \frac{M_f}{z \times \sigma_s} = 0,32 \text{ cm}^2$$

$$A = A_1 - \frac{N_u}{\sigma_s} = 0,31 \text{ cm}^2/\text{ml}$$

**Observation:** the reinforcement is done in ALS because it is the most unfavorable situation.

**Condition of non-fragility:**

$$A_{min} = 0,23 \times b \times d \times \frac{f_{t28}}{f_e} = 0,7728 \text{ cm}^2 > A$$

So, we will reinforce with  $A_{min}$  and we chose:  $A_s = 5HA8 = 2,51 \text{ cm}^2/\text{ml}$

**Distribution reinforcement:**

$$A_r = \frac{A}{4} = 0,5025 \text{ cm}^2/\text{ml} \rightarrow \text{we chose: } A_r = 4HA6 = 1,13 \text{ cm}^2/\text{ml}$$

**Spacing calculation:**

$$\text{Primary sense: } S_t \leq \frac{100}{5} = 20 \text{ cm}$$

$$\text{Secondary sense: } S_t \leq \frac{100}{4} = 25 \text{ cm}$$

**Shear check:**

We need to verify that:  $\tau_u < \bar{\tau}$

$$V_u = Q + F_p = 1 + 1,66 = 2,66 \text{ KN}$$

$$\tau_u = \frac{V_u}{b \times d} = \frac{2,66 \times 10^{-3}}{1 \times 0,08} = 0,03325 \text{ MPa}$$

$\tau_u < \bar{\tau}$  **Condition verified.**

Verification in SLS:

**Stress verification:**

$$\bar{\sigma} = \min \left( \frac{2}{3} \times f_e; 110 \sqrt{\eta \times f_{t28}} \right) = 333,33 \text{ MPa}$$

$$\sigma_{bc} = \frac{N_{ser}}{\mu_t} \times y; \sigma_{st} = 15 \times \frac{N_{ser} \times (d - y)}{\mu_t}$$

$$y = y_c + C$$

$$C = e_G - \frac{h}{2} = \frac{M_{ser}}{N_{ser}} - \frac{h}{2} = 0,242 - 0,05 = 0,1915 \text{ m (with } C < 0 \text{ and } y_c > 0)$$

$$y_c^3 + p y_c + q = 0 \dots \dots \dots (1)$$

$$p = -3C^2 - \frac{90A'}{b} (c - d') + \frac{90A}{b} (d - c) = -0,1053 \text{ m}^2$$

$$q = -2C^3 - \frac{90A'}{b}(c - d')^2 - \frac{90A}{b}(d - c)^2 = 0,0128m^3$$

The solution of the equation (1) depends on  $\Delta = 4p^3 + 27q^2 = -2,39 \times 10^{-4} < 0$   
 So, there are 3 solutions, we keep the one that is convenient to the following interval:

$$-C \leq y_c \leq h - C \rightarrow 0,192 \leq y_c \leq 0,292$$

$$y_{c1} = a \cos(\varnothing/3) = 0,2115$$

$$y_{c2} = a \cos(\varnothing/3 + 120^\circ) = -0,3737$$

$$y_{c3} = a \cos(\varnothing/3 + 240^\circ) = 0,1622$$

$$\text{With: } a = 2 \sqrt{-\frac{p}{3}} = 0,3748$$

$$\varnothing = \cos^{-1} \left( \frac{3q}{2p} \sqrt{-\frac{3}{p}} \right) = 166,71^\circ$$

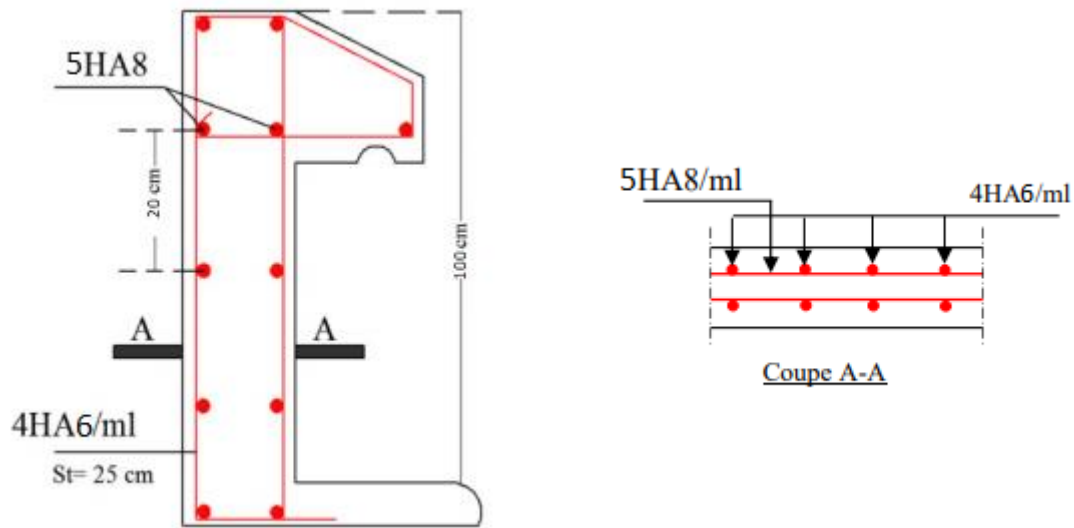
So, we take  $y_c = 0,2115 \rightarrow y = 0,01995m$

$$\mu_t = \frac{by^2}{2} - A(d - y) = 4,823 \times 10^{-5}$$

$$\sigma_{bc} = \frac{2,07 \times 10^{-3}}{4,823 \times 10^{-5}} \times 0,01995 = 0,8562 \text{ MPa} < \overline{\sigma}_{bc}$$

$$\sigma_{st} = 15 \times \frac{2,07 \times 10^{-3}}{4,823 \times 10^{-5}} \times (0,08 - 0,01995) = 32,22 \text{ MPa} < \overline{\sigma}_{st} = 333,33 \text{ MPa}$$

**Conditions verified.**



**Figure III.11:** The acroterion reinforcement schema.

c. Staircase

the staircase is a construction made up of a regular series of steps, allowing access to a floor, to go from one level to another by going up and down.

Different types of staircases exist, in this project we have a straight two-flight staircase.

H: the landing height.

$L_0$ : projected length of flight.

$L_v$ : length of flight.

$L_p$ : length of landing.

The dimensioning of the staircase comes to determining the following:



**The height (h) and width (g) of the steps:**

- The height of the riser (h) varies between 14 and 18cm.
- The tread (g) varies between 25 and 32cm.

For a comfortable staircase, we will use the formula BLONDEL verifying the coherence between the height of the stairs and the tread.

$$60cm \leq 2h + g \leq 64cm \dots (1)$$

The number of risers is given by:  $n = H/h$ .

With fixing the height of the risers h at 17cm, we will have the following riser number:

$$n = \frac{153}{17} = 9 \text{ risers.}$$

The number of stairs is  $(n - 1) = 8 \text{ stairs.}$

$$g = \frac{L_0}{n - 1} = \frac{240}{8} = 30cm$$

Which give us by the formula (1) a tread  $g=30cm$ , so the formula BLONDEL is verified.

**Connection angle:**

$$\alpha = \tan^{-1}\left(\frac{h}{g}\right) = \tan^{-1}\left(\frac{0,17}{0,3}\right) = 29,54^\circ$$

**Bench thickness:**

It is determined by satisfying the following two conditions:

$$\frac{L}{30} \leq e \leq \frac{L}{20} \rightarrow 14,96 \text{ cm} \leq e \leq 22,45cm$$

$$e \geq 11cm$$

We take:  $e = 17cm$

**Table III.28:** Staircase dimensions.

<b>H(m)</b>	<b>h(cm)</b>	<b>n</b>	<b>g(cm)</b>	<b>L<sub>0</sub>(m)</b>	<b>L<sub>p1</sub>(m)</b>	<b>L<sub>p2</sub>(m)</b>	<b>L<sub>v</sub>(m)</b>	<b>L(m)</b>	<b>α</b>	<b>e(cm)</b>
<b>1,53</b>	17	9	30	2,40	2,6	1,65	2,84	4,49	29,54°	17

**Solicitation's calculation:**

$$\text{Flight: } G_v = 7,96 \text{ KN/m}^2$$

$$Q_v = 2,5 \text{ KN/m}^2$$

$$\text{Landing: } G_p = 5,9 \text{ KN/m}^2$$

$$Q_p = 2,5 \text{ KN/m}^2$$

**Calculation of the loads that comes back on the staircase:**

$$\text{ULS: } q_{uv} = (1,35G_v + 1,5Q_v) \times 1 = 14,49 \text{ KN/m.}$$

$$q_{up} = (1,35G_p + 1,5Q_p) \times 1 = 11,17 \text{ KN/m.}$$

$$\text{SLS: } q_{sv} = (G_v + Q_v) \times 1 = 10,46 \text{ KN/m.}$$

$$q_{sp} = (G_p + Q_p) \times 1 = 8,4 \text{ KN/m.}$$

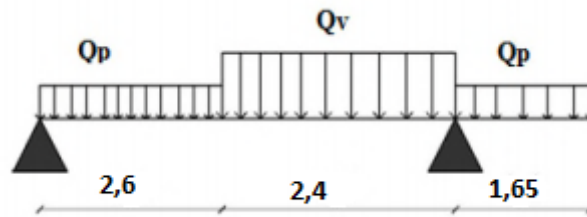


Figure III.12: The static plan of the staircase.

**Supports reactions:**

$$\sum F/x = 0 \rightarrow R_B + R_A = q_v \cdot 2,4 + q_p \cdot (2,6 + 1,65)$$

$$\sum M/A = 0 \rightarrow R_B = \begin{cases} ULS: 46,41 \text{ KN} \\ SLS: 34,37 \text{ KN} \end{cases}$$

$$\sum M/B = 0 \rightarrow R_A = \begin{cases} ULS: 35,83 \text{ KN} \\ SLS: 26,43 \text{ KN} \end{cases}$$

The beam we are studying is isostatic, so the calculation will be made by the method of sections.

The results are shown in the table below:

Table III.29: The staircase solicitations.

SUPPORT	REACTIONS (KN)		V (KN)	M (KN.M)		V <sup>max</sup> (KN)	M <sub>0max</sub>	
	ULS	SLS		ULS	SLS		ULS	SLS
A	35,83	26,43	35,83	0	0	35,83	57,47	41,33
B	46,41	34,37	-27,98	-16,62	16			

Because the supports are partially embedded, we have to correct the moments obtained:

$$M_{tu} = 0,85M_{u0} = 48,84 \text{ KN.m}$$

$$M_{au} = -0,5 M_{u0} = -28,73 \text{ KN.m}$$

**Reinforcement:**

The reinforcement is made in simple bending with  $M_t^{max}$  for a section  $(b \times h) = (1ml \times e)$ , the same thing for the reinforcement in the supports with  $M_a^{max}$ .

The results are shown in the table below:

Table III.30: Reinforcement of the staircase.

POSITION	$M_u$ (KN.m)	$\mu_u$	$\alpha$	Z (m)	$A^{calculé}$ (CM <sup>2</sup> /ML)	$A^{min}$ (CM <sup>2</sup> /ML)	$A^{adopted}$ (CM <sup>2</sup> /ML)	$s_t$ (CM)
SPAN	48,84	0,153	0,208	13,75	8,16	1,81	5HA16 = 10,06	20
SUPPORT	-28,73	0,09	0,118	14,46	4,56	1,81	5HA12 = 5,66	20

**Distribution reinforcements:**

We have distributed loads  $\rightarrow A_{distribution} = \frac{A_{principal}}{4}$

In span:  $A_{dis} = 2,51 \text{ cm}^2/\text{ml}$ , we take:  $A_{dis} = 5HA8 = 2,51 \text{ cm}^2/\text{ml}$

In supports:  $A_{dis} = 1,41 \text{ cm}^2/\text{ml}$ , we take:  $A_{dis} = 5HA8 = 2,51 \text{ cm}^2/\text{ml}$

**Verification of the shear effort:**

$$V^{max} = 35,83 \text{ KN} \rightarrow \tau_u = \frac{V^{max}}{1 \times d} = 0,239 \text{ MPa} \leq \bar{\tau}_u = 3,33 \text{ MPa}$$

So, we don't need to put transversal reinforcements.

**Calculation in SLS:**

As the cracking's are little harmful, we will only verify the stress in the concrete ( $\sigma_{bc}$ ):

$$\text{We need to verify that: } \sigma_{bc} = \frac{M_s}{I} \times y < \bar{\sigma}_{bc} = 0,6f_{c28}$$

By applying the method of sections:

$$M_s^{max} = 41,33 \text{ KN.m}$$

$$M_{ts} = 0,85M_s^{max} = 35,13 \text{ KN.m}$$

$$M_{as} = -0,5 M_s^{max} = -20,67 \text{ KN.m}$$

The stress verifications are presented in the table below:

**Table III.31: Stress verification in SLS.**

POSITION	$M_{ser}$ (KN.m)	Y (cm)	I (cm <sup>4</sup> )	$\sigma_{bc}$ (MPa)	$\bar{\sigma}_{bc}$ (MPa)	$\sigma_{bc} \leq \bar{\sigma}_{bc}$ (MPa)
IN SPAN	35,13	6,28	19729,14	11,18	15	Verified
IN SUPPORT	-20,67	5,03	14710,61	7,07	15	verified

**Arrow verification:**

The arrow verification is necessary if one of the following conditions is not verified:

$$\frac{h}{L} = \frac{17}{665} = 0,025 \geq \frac{1}{16} = 0,062 \rightarrow \text{condition not verified.}$$

$$\frac{h}{L} = 0,025 \leq \frac{M_t}{10M_0} = \frac{35,13}{10 \times 41,33} = 0,084 \rightarrow \text{condition verified.}$$

$$\frac{A_s}{bd} = \frac{10,06}{100 \times 15} = 0,0067 \leq \frac{4,2}{500} = 0,0084 \rightarrow \text{condition verified.}$$

The first condition is not satisfied, so the arrow verification is necessary.

The eligible arrow for a beam bigger than 5 meters is defined by the BAEL91 as

$$\text{following: } f = \frac{5q_s t^4}{384.E_v.I_{fv}} \leq \bar{f} = \frac{l}{500}$$

$$q_{max} = 10,46 \text{ KN/ml}$$

$$E_v = 3700 \sqrt{f_{c28}}$$

**Position of the neutral axis:**

$$V_1 = \frac{S_{/xx}}{B_0} = \frac{\frac{bh^2}{2} + 15A_t d}{bh + 15A_t} = \frac{14450 + (15 \times 10,06 \times 15)}{1700 + (15 \times 10,06)} = 9,02 \text{ cm}$$

$$V_2 = h - V_1 = 17 - 9,02 = 7,98 \text{ cm}$$

$$I_0 = (V_1^3 + V_2^3) \frac{b}{3} + 15A_t (V_2 - c)^2$$

$$I_0 = (9,02^3 + 7,98^3) \frac{100}{3} + 15 \times 10,06 \times (7,98 - 2)^2$$

$$I_0 = 46797,59 \text{ cm}^4$$

$$f = \frac{5 \times 10,46 \times 6,65 \times 10^3}{384 \times 18500 \times 10^6 \times 46797,59 \times 10^{-8}} = 0,0001 \text{ cm}$$

$$\bar{f} = \frac{665}{500} = 1,33 \text{ cm}$$

$$f = 0,0001 \text{ cm} < \bar{f} = 1,33 \text{ cm} \rightarrow \text{condition verified.}$$

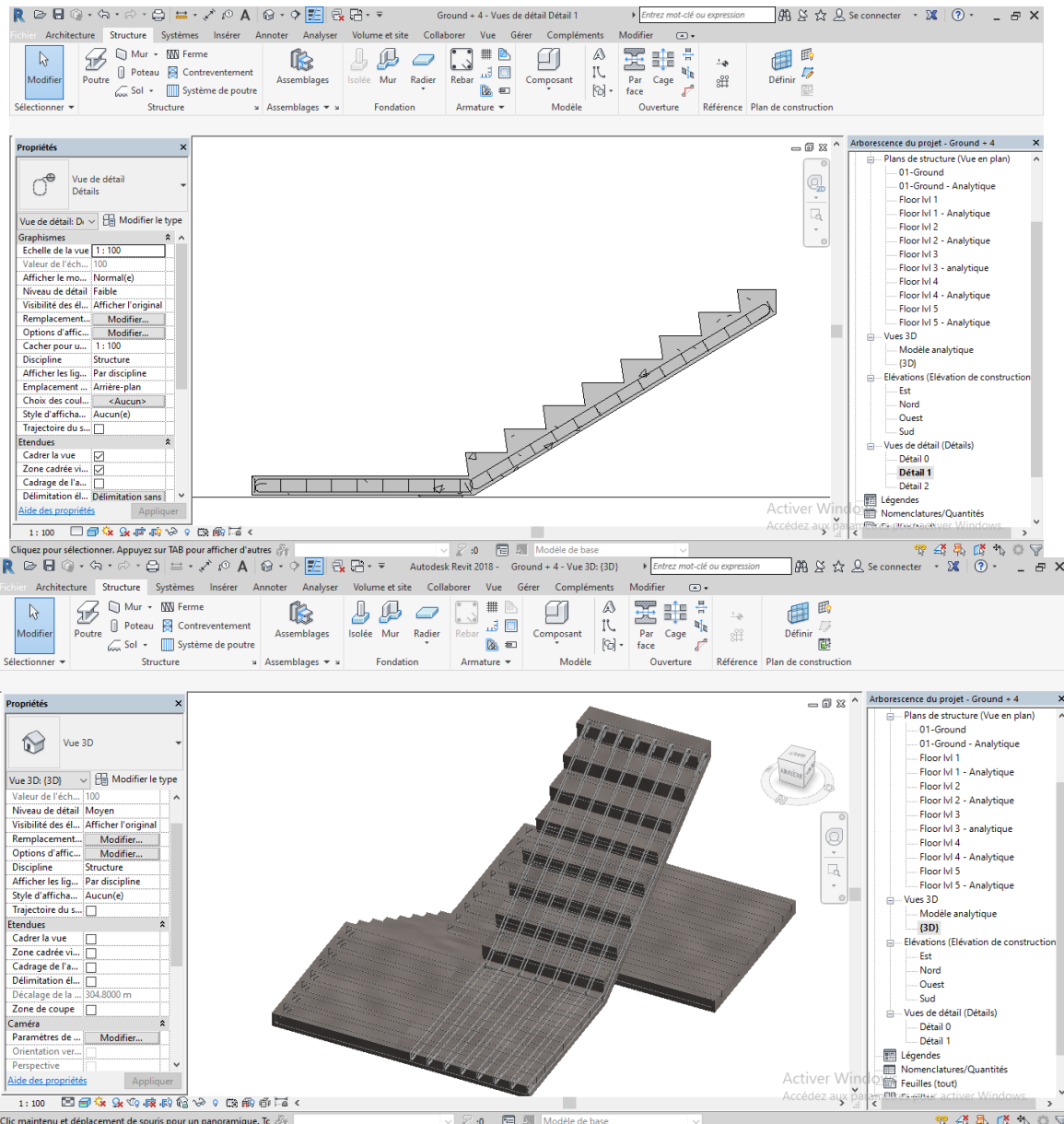


Figure III.13: Reinforced staircase.

d. The landing beam

**Dimensioning:**

After the condition of the arrow defined by the BAEL91:

$$\frac{L}{15} \leq h \leq \frac{L}{10} \rightarrow 20,33 \text{ cm} \leq h \leq 30,5 \text{ cm}$$

**RPA99/2003 rules:**

$$h \geq 30\text{cm}$$

$$b \geq 20\text{cm} \quad \text{so, we take: } b=h=30\text{cm}$$

$$h/b \geq 4$$

**loads definition:**

the landing beam is subjected to:

$$\text{its propre weight: } G_0 = 25 \times 0,30 \times 0,30 = 2,25 \text{ KN/m}$$

transmitted load from the flight:

$$\text{ULS : } R_{Bu} = 46,41 \text{ KN/m}$$

$$\text{SLS : } R_{Bs} = 34,37 \text{ KN/m}$$

Torsional moment:  $M_{tor} = M_B \times l/2$  : provoked by the bending of the flight.

**Calculation is simple bending:**

$$\text{ULS: } q_u = 1,35 G_0 + R_{Bu} = 49,44 \text{ KN/m}$$

$$\text{SLS : } q_s = G_0 + R_{Bs} = 36,62 \text{ KN/m}$$

**Moments :**

$$M_{0u} = \frac{q_u l^2}{8} = 57,49 \text{ KN.m}$$

$$M_{tu} = 0,85 M_0 = 48,87 \text{ KN.m}$$

$$M_{au} = -0,5 M_0 = -28,75 \text{ KN.m}$$

$$\text{Shear effort: } V_u = q_u \frac{l}{2} = 87,67 \text{ KN}$$

Reinforcements in ULS:

The results of the reinforcement are shown in the table below:

**Table III.32:** Reinforcement results in ULS.

POSITIO N	M (KN.m)	PIVOT	$\mu_{bu}$	$\alpha$	Z (M)	$A_{FS}^{cal}$ (cm <sup>2</sup> )	$A^{min}$ (cm <sup>2</sup> )
SPAN	48,87	A	0,147	0,199	0,257	4,37	1,01
SUPPORT	-25,75		0,077	0,100	0,268	2,3	1,01

**Shear stress in simple bending:**

$$\tau_u = \frac{V^{max}}{b \times d} = \frac{87,67 \times 10^{-3}}{0,30 \times 0,28} = 1,04 \text{ MPa}$$

**Transversal reinforcements:**

We fix  $S_t = 15\text{cm}$  and we calculate  $A_{trans}$ :

$$\text{a) } A_{trans} \geq \frac{0,4 \times b \times S_t}{f_e} \rightarrow A_{trans} \geq 0,36 \text{ cm}^2$$

$$\text{b) } A_{trans} \geq \frac{b \times S_t (\tau_u - 0,3 f_{t28})}{0,9 f_e} \rightarrow A_{trans} \geq 0,41 \text{ cm}^2$$

$$A_{trans} = \max(0,41\text{cm}^2; 0,36\text{cm}^2), \text{ we take } A_{trans} = 0,41\text{cm}^2.$$

Torsional calculation:

**Torsional moment:**

$$M_{tor} = -M_B \times \frac{l}{2} = -16,62 \times \frac{3,05}{2} = -25,34 \text{ KN.m}$$

With:  $M_B$ : moment at support (B) obtained in the calculation of the staircase.  
 The real section ( $b \times h$ ) is replaced by a hollow section equivalent in thickness ( $e = \emptyset/6$ ), because researches have showed that the core of a full section doesn't play any role is the torsional limit state.

$\emptyset = \min(b, h)$ : a cercle diameter could be included in the section ( $b \times h$ ).

$$e = \frac{h}{6} = 5\text{cm} \rightarrow \Omega = (b - e) \times (h - e) = 625\text{cm}^2$$

$U = 2(b + h) = 120\text{cm}$ : perimeter of the landing beam section.

**Longitudinal reinforcements:**

$$A_t^{tor} = \frac{M_{tor} \times U}{2\Omega\sigma_{st}} = \frac{25,34 \times 10^{-3} \times 1,2}{2 \times 0,0625 \times 435} = 5,59\text{cm}^2$$

**Transversal reinforcements:**

$$\text{We fix } S_t = 15\text{cm} \rightarrow A_t^{tor} = \frac{M_{tor} \times S_t}{2\Omega\sigma_{st}} = 0,69\text{cm}^2$$

**Shear stress:**

$$\tau_{tor} = \frac{M_{tor}}{2\Omega e} = 2,89\text{MPa}$$

We need to verify that:  $\tau_{total} \leq \bar{\tau}$

$$\text{With: } \tau_{total} = \sqrt{\tau_u^2 + \tau_{tor}^2} = \sqrt{1,04^2 + 2,89^2} = 3,07\text{ MPa}$$

$$\bar{\tau} = \min\left(\frac{0,2f_{cj}}{\gamma_b}; 5\text{MPa}\right) = 3,33\text{ MPa}$$

$\tau_{total} \leq \bar{\tau}$  **condition verified.**

There is no shear risk.

**Global reinforcement:**

$$\text{In span: } A_s = A_{span} + \frac{A_t^{tor}}{2} = 4,37 + \frac{5,59}{2} = 7,16\text{ cm}^2$$

We take: **4HA14 + 2HA12 = 8,42cm<sup>2</sup>**

$$\text{In supports: } A_s = A_{supp} + \frac{A_t^{tor}}{2} = 2,3 + \frac{5,59}{2} = 5,09\text{ cm}^2$$

We take: **3HA14 + 1HA12 = 5,75cm<sup>2</sup>**

**Transversal reinforcements:**

$$A_{trans} = A_{trans}^{S,B} + A_{trans}^{tor} = 0,41 + 0,69 = 1,10\text{ cm}^2$$

We take  $4\emptyset 8 = 2,01\text{ cm}^2$  (a frame + a stirrup).

Verification in SLS:

**Stress verification:**

$$q_s = 36,62\text{ KN/m}$$

**Moments :**

$$M_{0s} = \frac{q_s l^2}{8} = 42,58\text{ KN.m}$$

$$M_{ts} = 0,85 M_0 = 36,19\text{ KN.m}$$

$$M_{as} = -0,5 M_0 = -18,09\text{ KN.m}$$

The stress verification is presented in the table below:

Table III.33: Stress verification in SLS.

POSITION	$M_{ser}(KN.m)$	$Y (cm)$	$I (cm^4)$	$\sigma_{bc} (MPa)$	$\overline{\sigma}_{bc} (MPa)$	$\sigma_{bc} \leq \overline{\sigma}_{bc}$ (MPa)
IN SPAN	36,19	13,52	51194,63	9,55	15	Verified
AT SUPPORTS	-18,09	11,81	39079,64	5,46	15	verified

**Arrow verification:**

The arrow is verified if the following conditions are verified:

$$h \geq \max\left(\frac{1}{16}; \frac{M_t}{10M_0}\right) \times l \rightarrow h = 30 \geq 25,92 \quad \text{condition verified.}$$

$$A \leq \frac{4,2.b.d}{f_e} = 8,42 \text{ cm}^2 < 8,82 \text{ cm}^2 \quad \text{condition verified.}$$

$$L = 3,05 \text{ m} < 8 \text{ m} \quad \text{condition verified.}$$

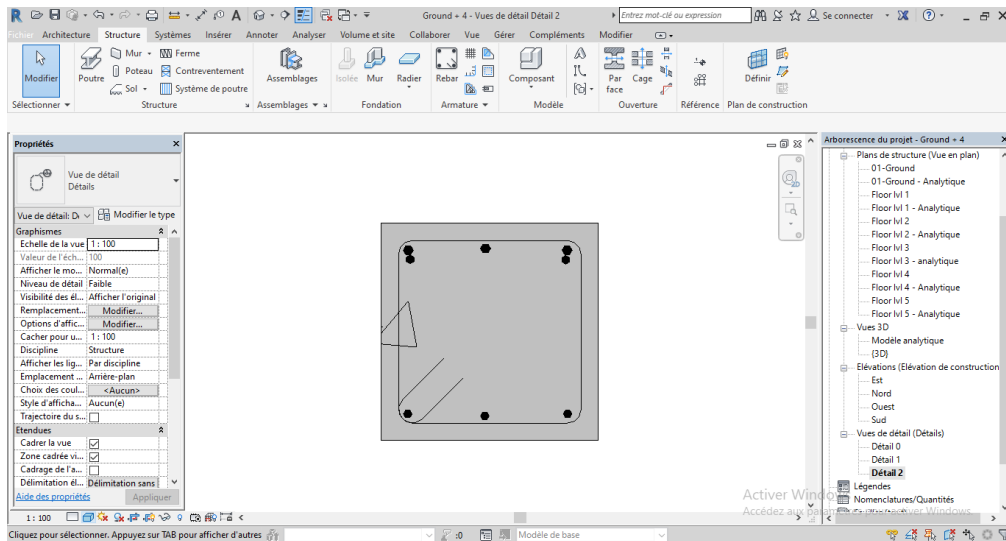


Figure III.14: Transversal cut on reinforced landing beam.

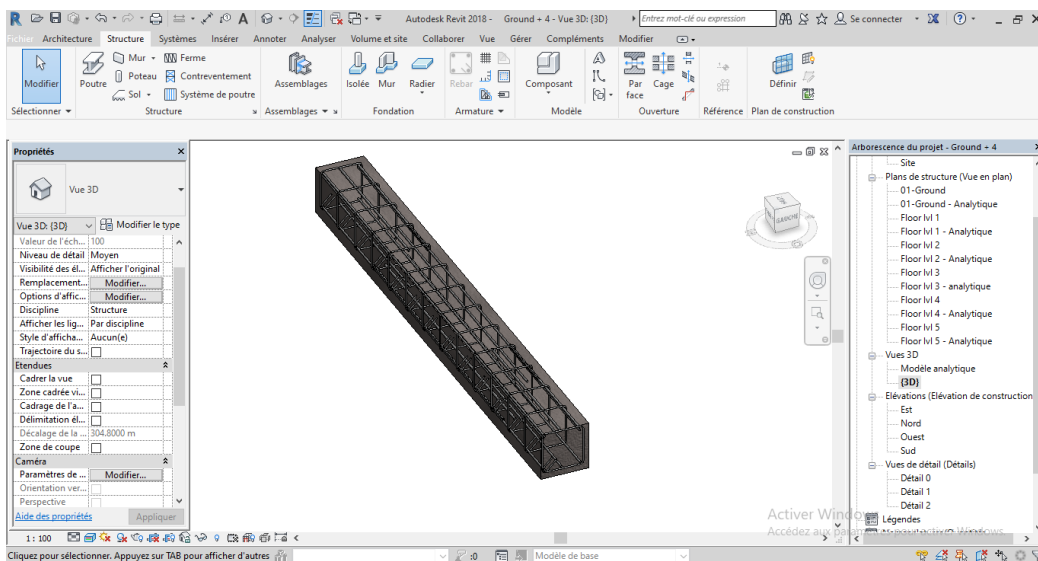


Figure III.15: Reinforced landing beam in 3D.

### III.6. Primary elements calculation

The columns and shear walls are subjected to normal efforts, shear efforts and bending moments. Therefore, they will be calculated in compound bending.

The beams are subjected to bending moments and shear efforts. Therefore, they will be calculated in simple bending.

#### a. Columns

The columns are calculated in compound bending under the most unfavorable action (normal effort (N) bending moment (M)) resulting from the combinations given as following:

$$\begin{cases} 1,35G + 1,5Q \dots \dots \dots ULS \\ G + Q \dots \dots \dots SLS \\ G + Q \pm E \dots \dots \dots Accidental \\ 0,8G \pm E \dots \dots \dots Accidental \end{cases}$$

The reinforcement adopted will be the maximum between the following solicitations:

$$\begin{aligned} N^{max} \rightarrow M^{corresponding} &\rightarrow A_1 \\ M^{max} \rightarrow N^{corresponding} &\rightarrow A_2 \\ N^{min} \rightarrow M^{corresponding} &\rightarrow A_3 \end{aligned} \quad A = \max(A_1; A_2; A_3)$$

#### Solicitation:

The resulting solicitation of calculation of the most unfavorable combination are derived from Autodesk Robot Structural Analysis 2018, the results are shown in the table below:

Table III.34: Solicitations in different columns.

LEVEL	$N_{max} \rightarrow M_{corr}$		$M_{max} \rightarrow N_{corr}$		$N_{min} \rightarrow M_{corr}$	
	N (KN)	M (KN.m)	M (KN.m)	N (KN)	N (KN)	M (KN.m)
GROUND	1104,66	1,66	149,26	772,62	1,35	72,11

#### Reinforcement calculation:

- Little damaging cracking.
- Calculation in compound bending.
- Calculation according to BAEL91 mod 99.

#### Example of calculation:

We will show an example of calculation for the columns of section (45x45) cm<sup>2</sup>, and the rest of the results of other sections reinforcement will be given in a summary table.

$$N_{max} = 1104,66 \text{ KN} \rightarrow M_{max}^{corr} = 1,66 \text{ KN.m}$$

$$M_{max} = 149,26 \rightarrow N^{corr} = 772,62 \text{ KN.m}$$

$$N_{min} = -278,94 \text{ KN} \rightarrow M_{max}^{corr} = 99,9 \text{ KN.m}$$

$$b = 45 \text{ cm} ; h = 45 \text{ cm} ; d = 43 \text{ cm}$$

$$\text{Accidental situation: } \gamma_b = 1,15 \text{ and } \gamma_s = 1$$

- Calculation under  $N_{max}$  and  $M_{corr}$ :

$$e_G = \frac{M}{N} = \frac{1,66}{1104,66} = 0,0015 \text{ m} < \frac{h}{2} = \frac{0,45}{2} = 0,225 \text{ m}$$

The pressure center is in the interior of the section between the reinforcements AA'.

So, the calculation is made as following:

We need to verify the following condition:

$$(a) \leq (b) \dots \dots \dots (I)$$

$$a = (0,337h - 0,81d') \times b \times h \times f_{bu}$$



$$b = N_u \times (d - d') - M_{UA}$$

$$M_{ua} = M + N \times (d - h/2) = 1,66 + 1104,66 \times (0,43 - 0,45/2) = 228,12 \text{ KN.m}$$

$$a = 0,5 > b = 0,45$$

(I) is not verified.

So, the section is partially compressed.

The calculation method is done by assimilation to the simple bending:

$$\mu_{bu} = \frac{M_{ua}}{f_{bu} \times b \times d^2} = \frac{228,12 \times 10^{-3}}{18,48 \times 0,45 \times 0,43^2} = 0,148$$

$$\mu_{bu} < 0,358 \rightarrow \text{pivot } A ; A = 0 ; f_{st} = \frac{f_e}{\gamma_s} = 500 \text{ MPa}$$

$$\alpha = 1,25[1 - \sqrt{1 - 2\mu_{bu}}] = 0,20$$

$$z = d(1 - 0,4\alpha) = 0,39 \text{ m}$$

$$A_1 = \frac{M_{ua}}{z \times f_{st}} = 11,69 \text{ cm}^2$$

We go back to the compound bending:

$$A = A_1 - \frac{N_u}{f_{st}} = -10,40 \text{ cm}^2/\text{ml} < 0 \rightarrow A = 0 \text{ cm}^2$$

- Calculation under  $M_{max}$  and  $N_{corr}$ :

$$e_G = \frac{M}{N} = \frac{149,26}{772,62} = 0,19 \text{ m} < \frac{h}{2} = \frac{0,45}{2} = 0,225 \text{ m}$$

The pressure center is in the interior of the section between the reinforcements AA'.

So, the calculation is made as following:

We need to verify the following condition:

(a) ≤ (b) ... .. (I)

$$a = (0,337h - 0,81d') \times b \times h \times f_{bu}$$

$$b = N_u \times (d - d') - M_{UA}$$

$$M_{ua} = M + N \times (d - h/2) = 149,26 + 772,62 \times (0,43 - 0,45/2) = 307,64 \text{ KN.m}$$

$$a = 0,5 > b = 0,009$$

(I) is not verified.

So, the section is partially compressed.

The calculation method is done by assimilation to the simple bending:

$$\mu_{bu} = \frac{M_{ua}}{f_{bu} \times b \times d^2} = \frac{307,64 \times 10^{-3}}{18,48 \times 0,45 \times 0,43^2} = 0,199$$

$$\mu_{bu} < 0,358 \rightarrow \text{pivot } A ; A = 0 ; f_{st} = \frac{f_e}{\gamma_s} = 500 \text{ MPa}$$

$$\alpha = 1,25[1 - \sqrt{1 - 2\mu_{bu}}] = 0,28$$

$$z = d(1 - 0,4\alpha) = 0,38 \text{ m}$$

$$A_1 = \frac{M_{ua}}{z \times f_{st}} = 16,19 \text{ cm}^2$$

We go back to the compound bending:

$$A = A_1 - \frac{N_u}{f_{st}} = 0,73 \text{ cm}^2/\text{ml}$$

$$A_{min} = 0,23 \times b \times d \times \frac{f_{t28}}{f_e} = 1,87 \text{ cm}^2$$

We reinforce with  $A_{min}$

The table below resume the results of the reinforcements for different levels:

**Table III.35:** The longitudinal reinforcement of the columns.

LEVEL	Section (cm <sup>2</sup> )	A <sub>cal</sub> (cm <sup>2</sup> )	A <sub>min</sub> (cm <sup>2</sup> )	A <sub>ROBOT</sub> (cm <sup>2</sup> )	A <sub>adopted</sub> (cm <sup>2</sup> )	BARS CHOICE
GROUND	45 × 45	0,73	18,2	7,7	20,36	4HA16 + 8HA14
LEVEL 1	45 × 45	0,73	18,2	7,7	20,36	4HA16 + 8HA14
LEVEL 2	40 × 40	3,78	14,4	3,6	17,29	4HA16 + 6HA14
LEVEL 3	40 × 40	3,78	14,4	3,6	17,29	4HA16 + 6HA14
LEVEL 4	40 × 40	3,78	14,4	3,6	17,29	4HA16 + 6HA14

**Transversal reinforcement:**

Calculation example:

We take for example the ground column (45x45) cm<sup>2</sup>:

$$A = 4HA16 + 8HA14 = 20,36cm^2$$

$$\text{So: } \phi_l^{min} = 1,4cm$$

$$\frac{A_t}{t} = \frac{\rho_a \times V_u}{h_1 \cdot f_e}$$

$$l_f = 0,7l_0 \text{ with } l_0 = h_{floor} - h_{beam}$$

$$\lambda_g = \frac{0,7 \times 2,61}{0,45} = 4,06 \rightarrow \rho_a = 3,75$$

$$A_t = \frac{3,75 \times 115,66 \times 10^{-3}}{45 \times 500} \times 10^2 = 2,89cm^2$$

**Spacing:**

In the nodal zone:  $t \leq \min(10\phi_{l\ min}; 15cm) = \min(14; 15) \rightarrow t = 12cm$

In the current zone:  $t' \leq 15\phi_{l\ min} = 15 \times 1,4 = 21cm \rightarrow t' = 15cm$

**The minimal quantity of reinforcements:**

We have:  $3 < \lambda_g = 4,06 < 5$ , therefore:

$$0,8\%(t \times b) \leq A_{t\ min} \leq 0,3\%(t \times b)$$

$$(5 - 3) \rightarrow 0,3\% - 0,8\%$$

$$(5 - 4,06) \rightarrow 0,3\% - x$$

$$A_{t\ min} = 0,455\%(t \times b) = 2,45cm^2$$

**Overlap length:**

$$L_r \geq 40\phi_l \rightarrow \phi_l = 1,6cm \rightarrow L_r = 64cm$$

**Table III.36:** The transversal reinforcements of the column

LEVEL	GROUND AND 1 <sup>ST</sup> FLOOR	OTHER FLOORS
SECTION (CM <sup>2</sup> )	45x45	40x40
$\phi_{l\ max}$ (cm)	1,6	1,4
$\phi_{l\ min}$ (cm)	1,6	1,4
$L_0$	2,61	2,61
$L_f$	1,827	1,827
$\lambda_g$	4,06	4,06
$\rho_a$	3,75	3,75
$V_u$ (KN)	115,66	104,73
$L_r$ (cm)	64	64
$S_t$ nodal zone (cm)	12	12
$S_t$ current zone (cm)	15	15
$A_{cal}^t$ (cm <sup>2</sup> )	2,89	1,96

$A_{min}^t$ (cm <sup>2</sup> )	2,45	2,18
$A_{adopted}^t$	3,93	3,93
NUMBER OF FRAMES	5HA10	5HA10

According the code of reinforced concrete (Art A7.1.3), the diameter of the transversal reinforcements should be as following:

$$\phi_t \geq \frac{\phi_l^{max}}{3} = \frac{16}{3} = 5,33mm$$

**Necessary verifications:**

Buckling verification:

The columns are subjected to compound bending, for that, the (Art B.8.2.1) requires us to justify them according to the limit state of form stability. The relation to verify is the following:

$$B_r \geq B_r^{cal} = \frac{N_u}{\alpha} \times \frac{1}{f_{c28}/(0,9 \times \gamma_b) + f_e/(100 \times \gamma_s)}$$

With:  $B_r = (b - 2) \times (h - 2)$  : reduced column section.

**Table III.37:** Buckling verification.

LEVEL	$N_u$ (KN)	$i$ (m)	$\lambda$	$\alpha$	$B_r \geq B_r^{cal}$		OBSERVATION
					$B_r$	$B_r^{cal}$	
GROUND	1104,27	0,1299	11,778	0,831	0,1849	0,05811	Verified
LEVEL 1	877,71	0,1299	11,778	0,831	0,1849	0,04619	Verified
LEVEL 2	653,16	0,1155	13,25	0,826	0,1444	0,03458	Verified
LEVEL 3	432,72	0,1155	13,25	0,826	0,1444	0,02291	Verified
LEVEL 4	214,84	0,1155	13,25	0,826	0,1444	0,01137	Verified

Stress verification:

Since the cracking's are little harmful, this verification consists on only controlling the compressive stress in the concrete of the most solicited column in every floor.

$$\sigma_{bc\ 1,2} \leq \overline{\sigma_{bc}} = 0,6f_{c28}$$

$$\begin{cases} \sigma_{bc\ 1} = \frac{N_{ser}}{S} + \frac{M_{ser}}{I_{yy'}} V \\ \sigma_{bc\ 1} = \frac{N_{ser}}{S} - \frac{M_{ser}}{I_{yy'}} V' \end{cases} \leq \overline{\sigma_{bc}}$$

With:

$S = b \times h + 15(A + A')$ : homogenized section.

$$I_{yy'} = \frac{b}{3}(V^3 + V'^3) + 15A'(V - d')^2 + 15A(d - V)^2$$

$V = h/2$  and  $V' = h - V$  (symmetrical section).

The results are shown in the table below:

**Table III.38:** Stress verification

LEVEL	GROUND	1 <sup>ST</sup> FLOOR	2 <sup>ND</sup> FLOOR	3 <sup>RD</sup> FLOOR	4 <sup>TH</sup> FLOOR
SECTIONS	45x45	45x45	40x40	40x40	40x40
$d$ (cm)	43	43	38	38	38
$A$ (cm <sup>2</sup> )	20,36	20,36	17,29	17,29	17,29
$V$ (cm)	22,5	22,5	20	20	20
$V'$ (cm)	22,5	22,5	20	20	20

$I_{yy'} (m^4)$	0,01495	0,01495	0,009373	0,009373	0,009373
$N_{ser} (KN)$	803,4	638,85	475,40	315,27	157,01
$M_{ser} (KN.m)$	1,2	0,72	0,77	1,12	3,96
$\sigma_{bc1} (MPa)$	2,2	3,16	2,98	1,99	1,06
$\sigma_{bc2} (MPa)$	3,9	3,14	2,95	1,94	0,89
$\overline{\sigma}_{bc} (MPa)$	15	15	15	15	15
<b>OBSERVATION</b>	verified	verified	verified	verified	verified

Shear stress verification:

According to the (Art 7.4.3.2), the shear stress in the concrete has to be less or equal to the ultimate shear stress:

$$\tau_{bu} = \frac{V_u}{b \cdot d} \leq \overline{\tau}_{bu} = \rho_d \times f_{c28}$$

$$\text{With: } \rho_d = \begin{cases} 0,075 & \text{if } \lambda_g \geq 5 \\ 0,040 & \text{if } \lambda_g < 5 \end{cases}$$

The results are shown in the table below:

**Table III.39:** Shear stress verification.

LEVEL	Sections (cm)	$l_f$ (cm)	$\lambda_g$	$\rho_d$	$d$ (cm)	$V_u$ (KN)	$\tau_{bu}$	$\overline{\tau}_{bu}$	OBSERVATION
<b>GROUND</b>	45x45	1,827	4,06	0,04	43	115,66	0,59	1,00	Verified
<b>1<sup>ST</sup> FLOOR</b>	45x45	1,827	4,06	0,04	43	109,97	0,56	1,00	Verified
<b>2<sup>ND</sup> FLOOR</b>	40x40	1,827	4,06	0,04	38	103,03	0,67	1,00	Verified
<b>3<sup>RD</sup> FLOOR</b>	40x40	1,827	4,06	0,04	38	104,73	0,68	1,00	Verified
<b>4<sup>TH</sup> FLOOR</b>	40x40	1,827	4,06	0,04	38	55,10	0,36	1,00	Verified

Nodal zone determination:

The determination of the nodal zone is necessary because it is at that level that we dispose the transversal reinforcements in a way to have reduced spacings. This is done because that place is very exposed to shear risk.

Overlap joints should be made if possible outside of these sensible nodal zones. (According to the RPA99/2003).

**Table III.40:** Values of the nodal zone.

LEVEL	GROUND	1 <sup>ST</sup> FLOOR	2 <sup>ND</sup> FLOOR	3 <sup>RD</sup> FLOOR	4 <sup>TH</sup> FLOOR
<b>SECTIONS</b>	45x45	45x45	40x40	40x40	40x40
$h'$ (cm)	60	60	60	60	60
$L'$ (cm)	90	90	90	90	90

Constructive disposition:

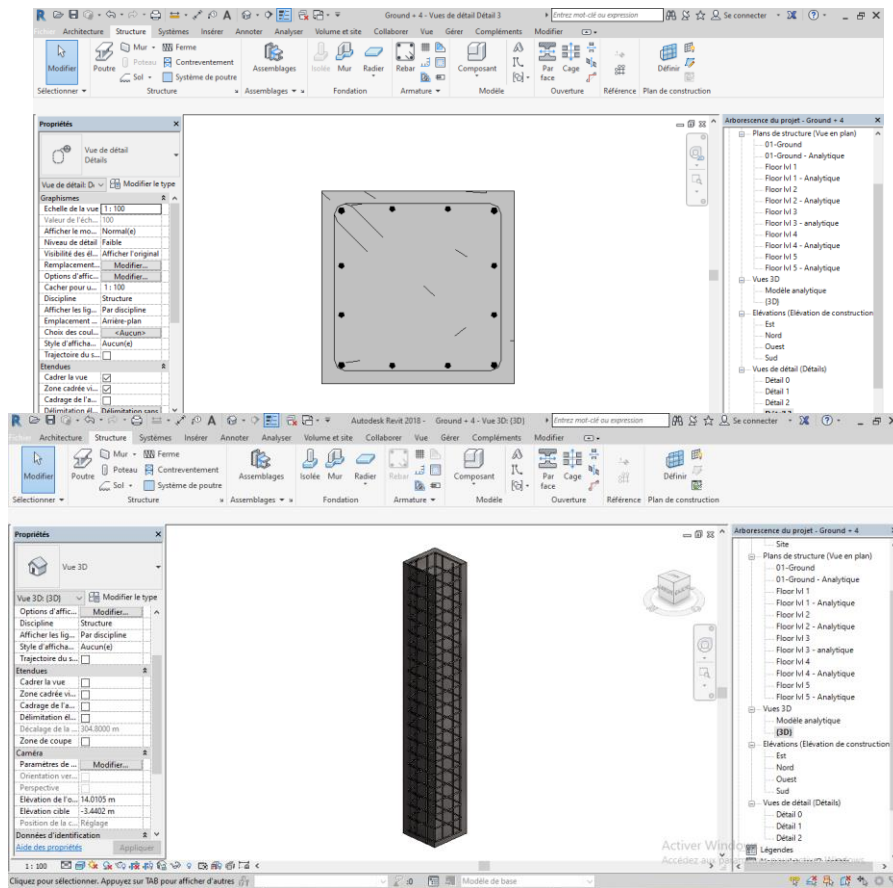
The distance between vertical bars of one face of the column should not go past 25cm.

Hook length:  $L = 10\phi_l = 10 \times 1,2 = 12m$ .

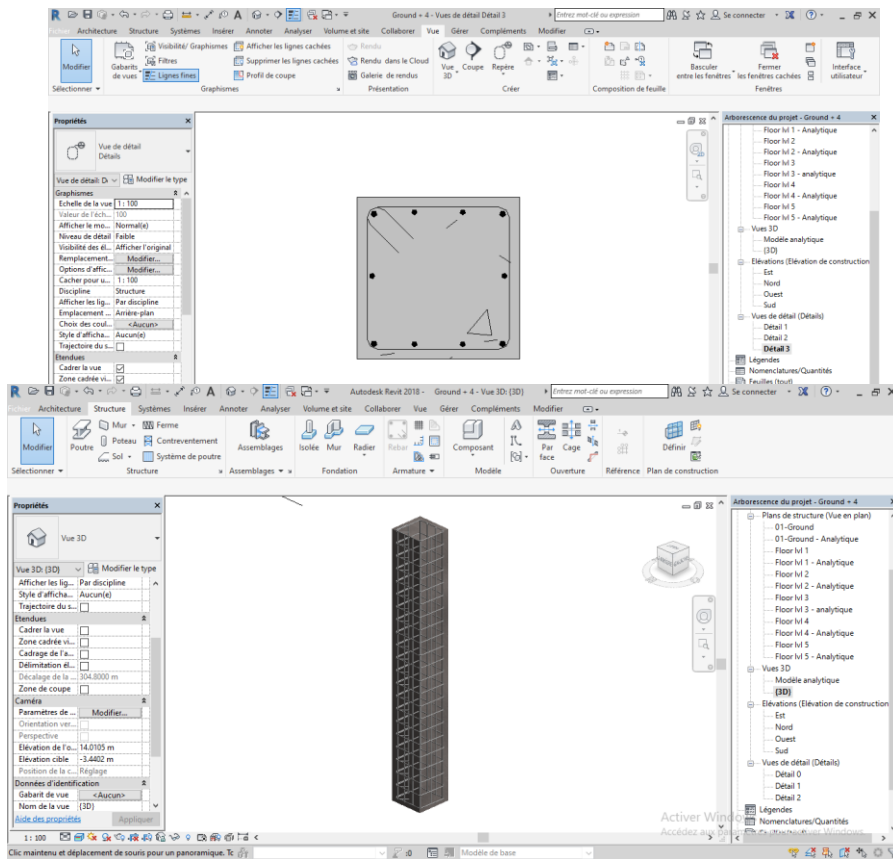
Overlap length:  $L_r \geq 40 \times \phi$

For  $\phi = 16mm \rightarrow L_r = 40 \times 1,6 = 64cm \rightarrow$  we take:  $L_r = 65cm$

For  $\phi = 14mm \rightarrow L_r = 40 \times 1,4 = 56cm \rightarrow$  we take:  $L_r = 60cm$



**Figure III.16:** Reinforced column (45x45) cm<sup>2</sup>



**Figure III.17:** Reinforced column (40x40) cm<sup>2</sup>.

b. Beams

The beams are calculated in simple bending under the most unfavorable solicitation action (bending moment and shear effort) resulting from the following combination:

$$\begin{cases} 1,35G + 1,5Q \dots \dots \dots ULS \\ G + Q \dots \dots \dots SLS \\ G + Q \pm E \dots \dots \dots ACC \\ 0,8G \pm E \dots \dots \dots ACC \end{cases}$$

**Solicitations and reinforcement of beams:**

The calculation solicitations are drawn directly from Autodesk RSA 2018.

**Table III.41:** Primary beams reinforcements (25x40) cm<sup>2</sup>

LEVEL	LOCALIZATION	M (KN.m)	A <sub>cal</sub> (cm <sup>2</sup> )	A <sub>Robot</sub>	A <sub>adopted</sub> (cm <sup>2</sup> )	A <sub>min</sub> (cm <sup>2</sup> )
<b>CURRENT FLOOR</b>	In span	61,18	3,82	7,4	<b>9,11</b>	5
	At supports	-183,7	11,48	15,4	<b>16,22</b>	5
<b>INACCESSIBLE TERRASSE</b>	In span	45,28	2,44	4,9	<b>8,01</b>	5
	At supports	-99,42	5,84	7,1	<b>8,01</b>	5

Bars choice:

*Current floor:* in span: **3HA16 + 2HA14 = 9,11 cm<sup>2</sup>**

At supports: **5HA16 + 4HA14 = 16,22 cm<sup>2</sup>**

*Inaccessible terrasse:* in span: **3HA14 + 3HA12 = 8,01 cm<sup>2</sup>**

At supports: **3HA14 + 3HA12 = 8,01 cm<sup>2</sup>**

**Table III.42:** Secondary beams reinforcements (25x35) cm<sup>2</sup>

LEVEL	LOCALIZATION	M (KN.m)	A <sub>cal</sub> (cm <sup>2</sup> )	A <sub>Robot</sub>	A <sub>adopted</sub> (cm <sup>2</sup> )	A <sub>min</sub> (cm <sup>2</sup> )
<b>CURRENT FLOOR</b>	In span	51,4	3,31	6,4	<b>6,88</b>	4,38
	At supports	-101,82	7,02	13,2	<b>14,68</b>	4,38
<b>INACCESSIBLE TERRASSE</b>	In span	27,12	1,69	3	<b>6,88</b>	4,38
	At supports	-54,25	3,5	6,2	<b>6,88</b>	4,38

Bars choice:

*Current floor:* in span: **3HA14 + 2HA12 = 6,88 cm<sup>2</sup>**

At supports: **5HA16 + 3HA14 = 14,68 cm<sup>2</sup>**

*Inaccessible terrasse:* in span: **3HA14 + 2HA12 = 6,88 cm<sup>2</sup>**

At supports: **3HA14 + 2HA12 = 6,88 cm<sup>2</sup>**

**Longitudinal reinforcements:**

Calculation example:

We take for an example of calculation the reinforcement of the primary beam (25x40) cm<sup>2</sup> with the following solicitation:

$$\{ M_a = -183,7 \text{ KN.m} \dots \dots \dots (ALS)$$

$$\{ M_t = 61,18 \text{ KN.m} \dots \dots \dots (ALS)$$

Reinforcements at supports:

$$\mu_{bu} = \frac{M_a}{b \times d^2 \times f_{bu}} = \frac{183,7 \times 10^{-3}}{0,25 \times 0,38^2 \times 18,48} = 0,275 < \mu_l = 0,392$$

Tense reinforcements only.

$$\alpha = 1,25(1 - \sqrt{1 - 2\mu_{bu}}) = 0,41$$

$$Z = d(1 - 0,4\alpha) = 0,32$$

$$A_s = \frac{M_a}{Z \times f_{st}} = \frac{183,7 \times 10^{-3}}{0,32 \times 500} \times 10^4 = 11,48 \text{ cm}^2$$

Reinforcements in span:

$$\mu_{bu} = \frac{M_t}{b \times d^2 \times f_{bu}} = \frac{61,18 \times 10^{-3}}{0,25 \times 0,38^2 \times 18,48} = 0,091 < \mu_l = 0,392$$

Tense reinforcements only.

$$\alpha = 1,25(1 - \sqrt{1 - 2\mu_{bu}}) = 0,12$$

$$Z = d(1 - 0,4\alpha) = 0,36$$

$$A_s = \frac{M_t}{Z \times f_{st}} = \frac{61,18 \times 10^{-3}}{0,36 \times 500} \times 10^4 = 3,82 \text{ cm}^2$$

**Transversal reinforcements:**

The diameter of the transversal reinforcements is given by the following relation:

$$\phi_t \leq \min\left(\phi_{l \min}; \frac{h}{35}; \frac{b}{10}\right) \rightarrow \begin{cases} \text{primary beams: } \phi_t \leq \min(12; 11,42; 25) \\ \text{secondary beams: } \phi_t \leq \min(12; 10; 25) \end{cases}$$

$$\text{With: } \begin{cases} \text{primary beams: } (25 \times 40) \text{ cm}^2 \\ \text{secondary beams: } (25 \times 35) \text{ cm}^2 \end{cases}$$

We take:  $\phi_t = 8 \text{ mm}$  and  $A_t = 4\phi_8 = 2,01 \text{ cm}^2$  (1 frame + 1 stirrup)

**Spacing calculation:**

According to the (Art.7.5.2.2) RPA99 mod 2003:

$$\text{In the nodal zone: } S_t \leq \min\left(\frac{h}{4}; 12\phi_t^{\min}\right) \rightarrow \begin{cases} \text{primary beams: } S_t = 10 \text{ cm} \\ \text{secondary beam: } S_t = 8 \text{ cm} \end{cases}$$

$$\text{In the current zone: } S_t \leq \frac{h}{2} \rightarrow \begin{cases} \text{primary beams: } S_t = 15 \text{ cm} \\ \text{secondary beams: } S_t = 15 \text{ cm} \end{cases}$$

**Verification of the minimal section of transversal reinforcements:**

We have:  $A_{\min} = 0,3\% \times S_t \times b = 1,13 \text{ cm}^2 < A_t = 2,01 \text{ cm}^2$  verified.

**Calculation of the overlap length:**

For  $\phi = 16 \text{ mm}$   $\rightarrow L_r = 40 \times 1,6 = 64 \text{ cm}$   $\rightarrow$  we take:  $L_r = 65 \text{ cm}$ .

For  $\phi = 14 \text{ mm}$   $\rightarrow L_r = 40 \times 1,4 = 56 \text{ cm}$   $\rightarrow$  we take:  $L_r = 60 \text{ cm}$ .

For  $\phi = 12 \text{ mm}$   $\rightarrow L_r = 40 \times 1,2 = 48 \text{ cm}$   $\rightarrow$  we take:  $L_r = 50 \text{ cm}$ .

**Necessary verifications:**

The maximum longitudinal reinforcements percentage verification:

For all the beams:

In the overlap zone:  $A_{\max} = 4\% (b \times h)$

Primary beam:  $A_{\max} = 40 \text{ cm}^2$

Secondary beam:  $A_{\max} = 35 \text{ cm}^2$

In the current zone:  $A_{\max} = 6\% (b \times h)$

Primary beam:  $A_{\max} = 60 \text{ cm}^2$

Secondary beam:  $A_{max} = 52,5 \text{ cm}^2$   
 So, it is verified for all the beams.

Verification in ULS:

**Non fragility condition:**

$$A_{min} = 0,23 \times b \times h \times \frac{f_{t28}}{f_e} \leq A^{cal} \rightarrow \begin{cases} \text{primary beam: } A_{min} = 1,05 \text{ cm}^2 \\ \text{secondary beams: } A_{min} = 0,92 \text{ cm}^2 \end{cases}$$

**Condition verified.**

**Tangential stress verification:**

The condition that we need to verify is the following:

$$\tau_{bu} = \frac{V_u}{b \cdot d} \leq \overline{\tau}_{bu} = \min\left(0,2 \frac{f_{c28}}{\gamma_b}; 5 \text{ MPa}\right)$$

The verification will be done only for the most unfavorable beams, because if these ones are verified, the others will be too.

The results are shown in the table below:

**Table III.43:** Tangential stress verification.

BEAMS	$V^{max}$ (KN)	$\tau_{bu}$ (MPa)	$\overline{\tau}_{bu}$ (MPa)	OBSERVATION
PRIMARY BEAMS	60,08	0,63	3,33	verified
SECONDARY BEAMS	16,45	0,2	3,33	verified

**Longitudinal reinforcements shear verification:**

for the edge supports:  $A_l \geq A_l^{edge} = V^{max} \times \frac{\gamma_s}{f_e}$

for the intermediate supports:  $A_l \geq A_l^{inter} = (V^{max} + \frac{M_a}{0,9d}) \times \frac{\gamma_s}{f_e}$

**Table III.44:** Longitudinal reinforcements shear verification.

BEAMS	$V^{max}$ (KN)	$M_a$ (KN.m)	$A_l$ (cm <sup>2</sup> )	$A_l^{edge}$ (cm <sup>2</sup> )	$A_l^{inter}$ (cm <sup>2</sup> )	OBSERVATION
PRIMARY BEAMS	60,08	-183,7	16,22	1,38	1,37	Verified
SECONDARY BEAMS	16,45	-101,82	14,68	0,37	0,83	Verified

Verification in SLS:

The concerned verifications are the following:

- Verification of the concrete compression limit state.
- Verification of the deformation limit state (bending evaluation).

**Concrete compression limit state:**

$$\sigma_{bc} = \frac{M_{ser}}{I} y \leq \overline{\sigma}_{bc} = 0,6 \times f_{c28} = 15 \text{ MPa}$$



**Table III.45:** Concrete compression limit state verification.

BEAMS	LOCALIZATI ON	$M_{ser}$ (KN.m)	$A_s$ (cm <sup>2</sup> )	$Y$ (cm)	$I$ (cm <sup>4</sup> )	STRESS		OBSERVATI ON
						$\sigma$ (MPa)	$\bar{\sigma}$ (MPa)	
PRIMARY	Span	32,96	4,53	13,99	61989,52	7,44	15	Verified
	Supports	-65,05	4,53	13,99	61989,52	14,68	15	Verified
SECONDA RY	Span	7,51	4,53	12,82	45229,76	2,12	15	Verified
	supports	-16,34	4,53	12,82	45229,76	4,63	15	Verified

**Deformation limit state:**

The bending verification is necessary if one of these conditions is not verified:

$$h \geq h_f = \max\left(\frac{1}{16}; \frac{M_t}{10M_0}\right) \times l$$

$$A \leq A_f = \frac{4,2 \cdot b \cdot d}{f_e}$$

$$L < 8m$$

$$\text{primary beams: } \begin{cases} h = 40cm < h_f = 40,8cm \\ A = 9,11cm^2 > A_f = 7,98cm^2 \end{cases}$$

$$\text{secondary beams: } \begin{cases} h = 35cm > h_f = 34 cm \\ A = 6,88 cm^2 < A_f = 6.93 cm^2 \end{cases}$$

We need to verify the bending for the primary beams:

$$f = \frac{5q_s l^4}{384 \cdot E_v \cdot I_{fv}} \leq \bar{f} = \frac{l}{500}$$

$$q_s = 11,44 \text{ KN/m}$$

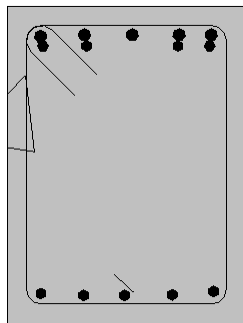
$$E_v = 3700 \cdot \sqrt{f_{c28}} = 10818,86 \text{ MPa}$$

$$V_1 = \frac{S_{/xx}}{B_0} = \frac{\frac{bh^2}{2} + 15A_t d}{bh + 15A_t} = 21,14 \text{ cm}$$

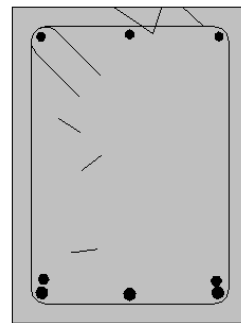
$$V_2 = h - V_1 = 18,86 \text{ cm}$$

$$I_0 = (V_1^3 + V_2^3) \frac{b}{3} + 15A_t (V_2 - c)^2 = 153948,37 \text{ cm}^4$$

$$f = 4,74 \times 10^{-8} \text{ cm} \leq \bar{f} = 0,96 \text{ cm. Condition verified.}$$

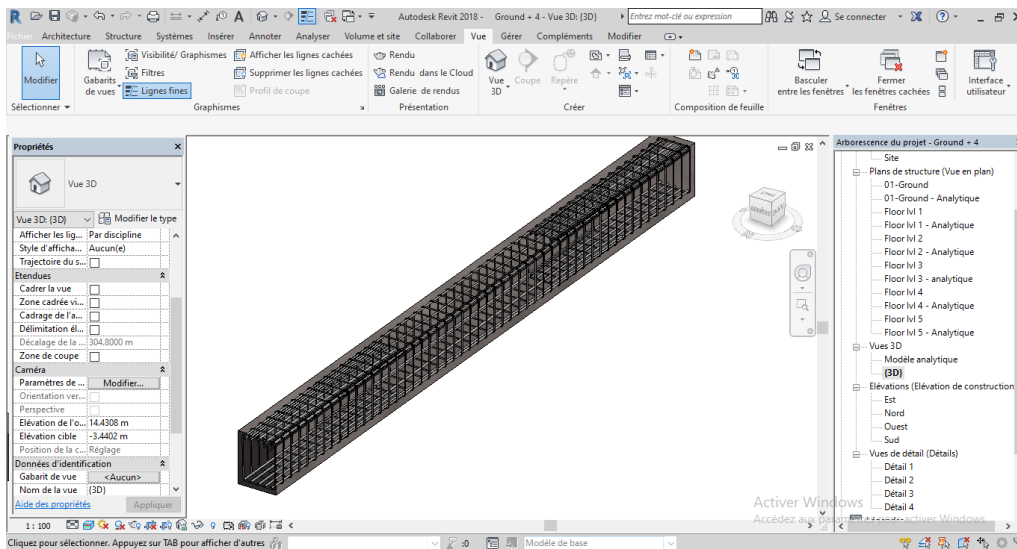


At supports.

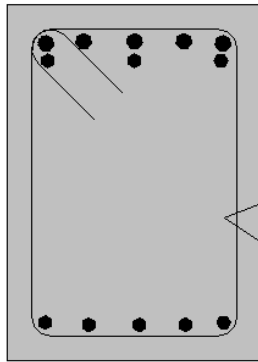


In span.

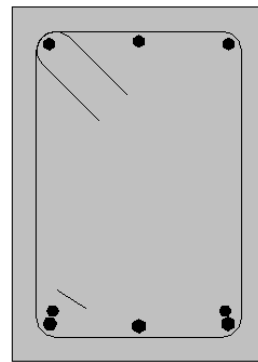
**Figure III.18:** Current floor primary beams reinforcement schema.



**Figure III.19:** Current floor reinforced primary beam in 3D.

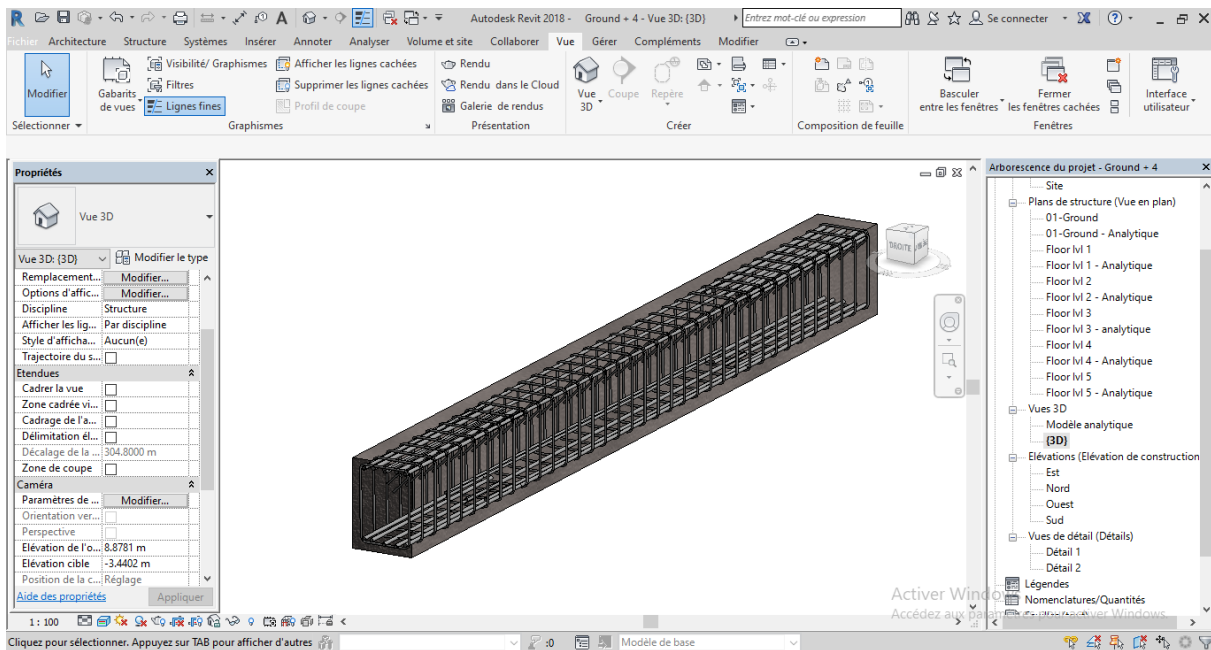


At supports.

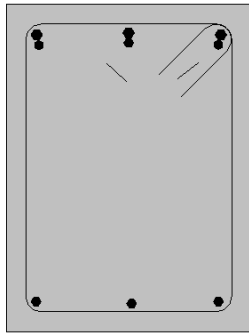


In span.

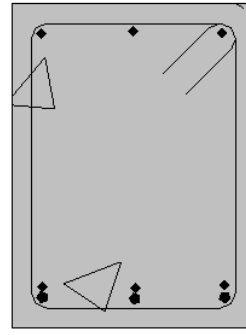
**Figure III.20:** Current floor secondary beams reinforcement schema.



**Figure III.21:** Current floor reinforced secondary beam in 3D.

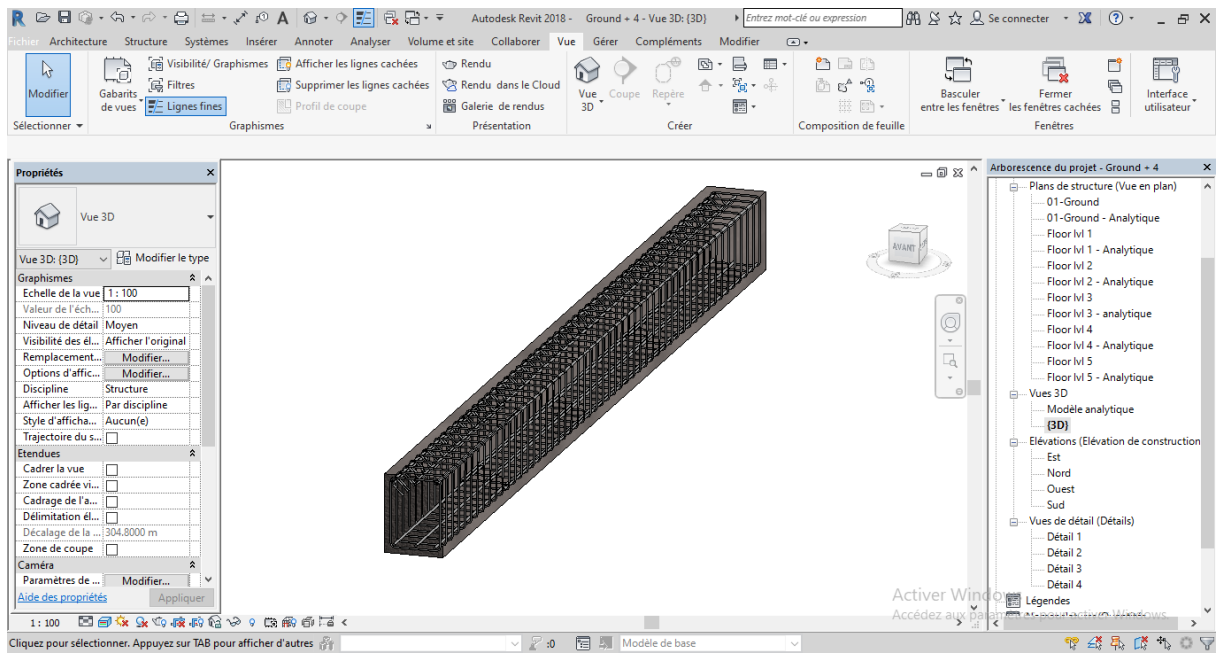


At supports.

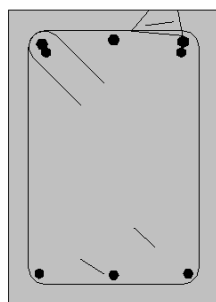


In span.

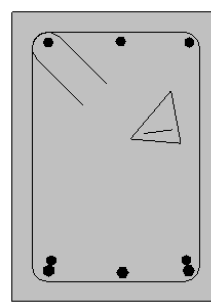
**Figure III.22:** Terrasse primary beams reinforcement schema.



**Figure III.23:** Terrasse reinforced primary beam in 3D.

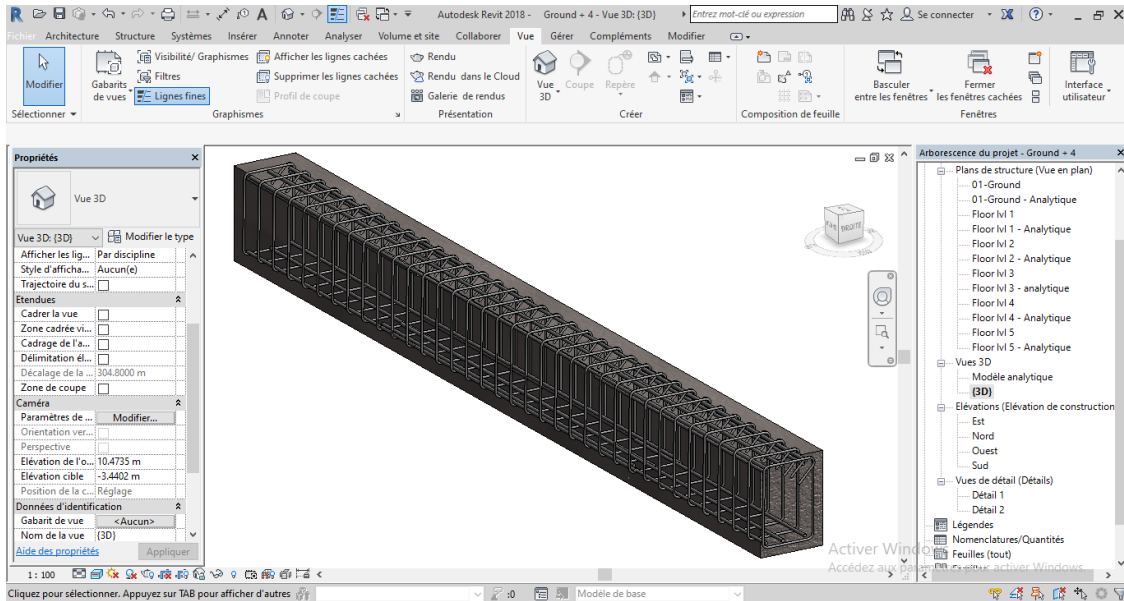


At supports.



In span.

**Figure III.24:** Terrasse secondary beams reinforcement schema.



**Figure III.25:** Terrasse reinforced secondary beam in 3D.

**c. Shear walls**

The (Art.3.4. A.1.a) requires to put shear walls for every reinforced concrete structure that goes past 2 levels or 8m of height in the zone III.

The shear walls could be full or could have some openings, in our case we only have full shear walls.

The shear walls are considered as a bracket embedded at their base; their rupture mode is:

- Rupture by bending.
- Rupture in bending by the shear effort.
- Rupture by embedding or concrete traction.

From which, shear walls will be calculated in compound bending with shear effort considering the moment acting in the direction with the largest inertia.

The calculation will be made with the following combinations:

$$\begin{cases} 1,35G + 1,5Q \\ G + Q \pm E \\ 0,8G \pm E \end{cases}$$

The reinforcement that we will adopt is given by the following solicitations:

$$\begin{cases} M^{max} \rightarrow N_{corresp} \\ N^{max} \rightarrow M_{corresp} \\ N^{min} \rightarrow M_{corresp} \end{cases}$$

**Example of calculation:**

The solicitations are extracted directly from the software RSA 2018, the results are resumed in the following table:

**Table III.46:** Shear walls solicitations.

LEVEL	$N_{max} \rightarrow M_{cor}$		$M_{max} \rightarrow N_{cor}$		$N_{min} \rightarrow M_{cor}$		$V_u$ (KN)
	$N(KN)$	$M(KN.m)$	$M(KN.m)$	$N(KN)$	$N(KN)$	$M(KN.m)$	
GROUND	-1069,9	331,14	334,27	-1042,82	738,95	324,39	93,17
COMBINATION	$G + Q - E_y$		$G + Q - E_y$		$0,8G + E_y$		

**Reinforcements:**

Calculation under  $M_{max}$  and  $N_{cor}$ :

The calculation of the vertical reinforcements is done in compound bending under the most unfavorable solicitation (M, N) for a section ( $e \times l$ ).

The section found (A) will be distributed on all the tense area of the section with respecting the conditions of RPA99

$$L = 3,06 \text{ m} ; d = 3,04 \text{ m} ; e = 0,20 \text{ m}$$

$$M_{max} = 334,27 \text{ KN.m} ; N_{cor} = -1042,82 \text{ KN (compression)}$$

$$e_G = \left| \frac{M}{N} \right| = 0,32 \text{ m} < \frac{l}{2} = 1,53 \text{ m} \rightarrow \text{the center of pression is inside the section.}$$

So, the section is partially compound. The calculation method is done by assimilation to simple bending.

$$M_{ua} = M + N \times \left( d - \frac{h}{2} \right) = 1908,92 \text{ KN.m}$$

$$\mu_{bu} = \frac{M_{ua}}{bd^2 f_{bu}} = \frac{1908,92 \times 10^{-3}}{0,15 \times 3,04^2 \times 18,48} = 0,169$$

$$\mu_{bu} = 0,169 < \mu_l = 0,371$$

$$\alpha = 1,25(1 - \sqrt{1 - 2 \times \mu_{bu}}) = 0,23$$

$$z = d(1 - 0,4\alpha) = 2,76 \text{ m}$$

$$A_1 = \frac{M_{ua}}{z \times f_{st}} = \frac{1908,92 \times 10^{-3}}{2,76 \times 500} = 13,83 \text{ cm}^2$$

$$A = A_1 - \frac{N_u}{f_{st}} = 13,83 \times 10^{-4} - \frac{1042,82 \times 10^{-3}}{500} = -7,03 \text{ cm}^2$$

$$A_s = 0$$

Minimum reinforcement in the shear wall:

According to RPA99/2003 we have:  $A_{min} = 0,15\% b \times h = 9,12 \text{ cm}^2$

The tense part length  $L_t$ :

$$A_{min}^{tense} = 0,2\% b \times L_t$$

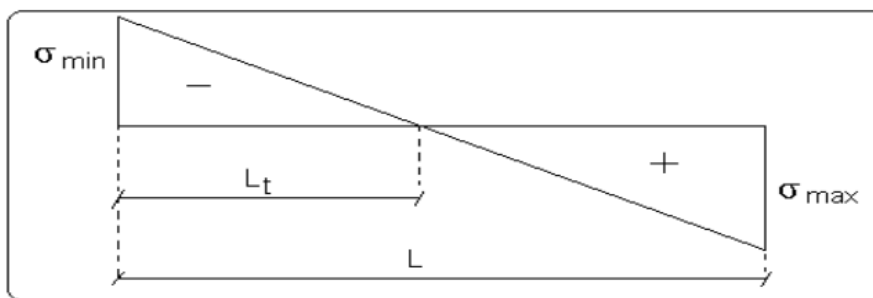


Figure III.26: Stress schema.

$$L_t = \frac{\sigma_{min} \times L}{\sigma_{max} + \sigma_{min}}$$

$$\sigma_1 = \frac{N}{B} + \frac{M}{I} V = \frac{1042,82 \times 10^{-3}}{0,15 \times 3,06} + \frac{334,27 \times 10^{-3}}{0,478} \times 1,53 = 2,77 \text{ MPa} > 0$$

$$\sigma_2 = \frac{N}{B} - \frac{M}{I} V = \frac{1042,82 \times 10^{-3}}{0,15 \times 3,06} - \frac{334,27 \times 10^{-3}}{0,478} \times 1,53 = 0,63 \text{ MPa} > 0$$

The section is all compressed.

Vertical bars spacing:

$$S_t \leq \min(1,5e; 30\text{cm}) = 30\text{ cm}$$

With:  $S_t = 8\text{ cm}$  in  $L/10$  of the shear wall length.

$S_t = 16\text{cm}$  outside  $L/10$  of the shear wall.

Horizontal reinforcements:

The horizontal reinforcements section is calculated with the following formula:

$$V_{max} = 93,17\text{ KN}$$

$$A_h = \frac{\tau_u \times e \times S_t}{0,8 \times f_e}$$

$$\tau_u = \frac{1,4V_u}{e \times d} = \frac{1,4 \times 93,17 \times 10^{-3}}{0,15 \times 3,04} = 0,28\text{ MPa}$$

$$S_t \leq \min(1,5 e ; 30\text{ cm}) = 30\text{cm}$$

We take:  $S_t = 20\text{ cm}$

$$A_h = \frac{0,53 \times 0,2 \times 0,2}{0,8 \times 500} = 0,53\text{ cm}^2$$

Calculation under  $N_{min}$  and  $M_{cor}$ :

$$L = 3,06\text{ m} ; d = 3,04\text{m} ; e = 0,20\text{m}$$

$$N_{min} = 738,95\text{ KN.m} ; M_{cor} = 324,39\text{ KN}$$

$$e_G = \left| \frac{M}{N} \right| = 0,43\text{m} < \frac{l}{2} = 1,53\text{m} \rightarrow \text{the center of pression is inside the section.}$$

So, the section is partially compound.

$$M_{ua} = M + N \times \left( d - \frac{h}{2} \right) = 1440,20\text{ KN.m}$$

$$\mu_{bu} = \frac{M_{ua}}{bd^2f_{bu}} = \frac{1440,20 \times 10^{-3}}{0,15 \times 3,04^2 \times 18,48} = 0,128$$

$$\mu_{bu} = 0,128 < \mu_l = 0,371$$

$$\alpha = 1,25(1 - \sqrt{1 - 2 \times \mu_{bu}}) = 0,17$$

$$z = d(1 - 0,4\alpha) = 2,82\text{m}$$

$$A_1 = \frac{M_{ua}}{z \times f_{st}} = \frac{1440,2 \times 10^{-3}}{2,82 \times 500} = 10,21\text{cm}^2$$

$$A = A_1 - \frac{N_u}{f_{st}} = 10,21 \times 10^{-4} - \frac{738,95 \times 10^{-3}}{500} = -4,56\text{cm}^2$$

$$A_s = 0$$

Calculation under  $N_{max}$  and  $M_{cor}$ :

$$L = 3,06\text{ m} ; d = 3,04\text{m} ; e = 0,20\text{m}$$

$$N_{max} = -1069,9\text{ KN.m (compression)} ; M_{cor} = 331,14\text{ KN}$$

$$e_G = \left| \frac{M}{N} \right| = 0,31\text{m} < \frac{l}{2} = 1,53\text{m} \rightarrow \text{the center of pression is inside the section.}$$

So, the section is partially compound.

$$M_{ua} = M + N \times \left( d - \frac{h}{2} \right) = 1569,92\text{ KN.m}$$

$$\mu_{bu} = \frac{M_{ua}}{bd^2f_{bu}} = \frac{1569,92 \times 10^{-3}}{0,15 \times 3,04^2 \times 18,48} = 0,139$$

$$\mu_{bu} = 0,139 < \mu_l = 0,371$$

$$\alpha = 1,25(1 - \sqrt{1 - 2 \times \mu_{bu}}) = 0,19$$

$$z = d(1 - 0,4\alpha) = 2,8\text{m}$$

$$A_1 = \frac{M_{ua}}{z \times f_{st}} = \frac{1569,92 \times 10^{-3}}{2,8 \times 500} = 11,21 \text{ cm}^2$$

$$A = A_1 - \frac{N_u}{f_{st}} = 11,21 \times 10^{-4} - \frac{1069,9 \times 10^{-3}}{500} = -10,01 \text{ cm}^2$$

$$A_s = 0$$

$$\sigma_1 = \frac{N}{B} + \frac{M}{I} V = \frac{1069,9 \times 10^{-3}}{0,15 \times 3,06} + \frac{331,14 \times 10^{-3}}{0,478} \times 1,53 = 2,80 \text{ MPa} > 0$$

$$\sigma_2 = \frac{N}{B} - \frac{M}{I} V = \frac{1069,9 \times 10^{-3}}{0,15 \times 3,06} - \frac{331,14 \times 10^{-3}}{0,478} \times 1,53 = 0,68 \text{ MPa} > 0$$

The section is all compressed.

**Bars choice:**

Vertical reinforcements: **12HA10 = 9,42 cm<sup>2</sup>**

Horizontal reinforcements: **2HA10 = 1,57 cm<sup>2</sup>**

The table below shows the results of the calculation of vertical and horizontal reinforcements of the shear wall:

**Table III.47:** Solicitations and reinforcements of shear wall on all levels y-y.

LEVEL	GROUND	1 <sup>ST</sup> FLOOR	2 <sup>ND</sup> FLOOR	3 <sup>RD</sup> AND 4 <sup>TH</sup> FLOOR
<b>SECTION</b>	0,15x3,06	0,15x3,06	0,15x3,06	0,15x3,06
<b>N (KN)</b>	-1042,82	-440,12	-207,42	151,54
<b>M (KN.m)</b>	334,27	183,94	105,66	53,49
<b>V (KN)</b>	93,17	95,85	70,10	45,88
<b>τ (MPa)</b>	0,28	0,29	0,22	0,14
<b>τ̄ (MPa)</b>	5	5	5	5
<b>A<sub>cal</sub> (cm<sup>2</sup>)</b>	0	0	0	0
<b>A<sub>min</sub> (cm<sup>2</sup>)</b>	9,12	9,12	9,12	9,12
<b>A<sub>adop</sub> (cm<sup>2</sup>)</b>	9,42	9,42	9,42	9,42
<b>N<sup>bar</sup></b>	12HA10	12HA10	12HA10	12HA10
<b>S<sub>t</sub> (cm)</b>	16	16	16	16
<b>A<sub>h</sub><sup>adop</sup> (cm<sup>2</sup>)</b>	1,57	1,57	1,57	1,57
<b>N<sup>bar</sup></b>	2HA10	2HA10	2HA10	2HA10
<b>S<sub>t</sub> (cm)</b>	20	20	20	20

**Table III.49:** Solicitations and reinforcements of shear wall on all levels x-x.

LEVEL	GROUND	1 <sup>ST</sup> FLOOR	2 <sup>ND</sup> FLOOR	3 <sup>RD</sup> AND 4 <sup>TH</sup> FLOOR
<b>SECTION</b>	0,15x3,06	0,15x3,06	0,15x3,06	0,15x3,06
<b>N (KN)</b>	1112,67	-675,78	154,29	90,60
<b>M (KN.m)</b>	388,0	162,65	87,83	44,65
<b>V (KN)</b>	139,99	49,23	14,98	61,11
<b>τ (MPa)</b>	0,43	0,15	0,046	0,19

$\bar{\tau}$ (MPa)	5	5	5	5
$A_{cal}$ (cm <sup>2</sup> )	0	0	0	0
$A_{min}$ (cm <sup>2</sup> )	9,12	9,12	9,12	9,12
$A_{adop}$ (cm <sup>2</sup> )	9,42	9,42	9,42	9,42
$N^{bar}$	12HA10	12HA10	12HA10	12HA10
$S_t$ (cm)	16	16	16	16
$A_h^{adop}$ (cm <sup>2</sup> )	1,57	1,57	1,57	1,57
$N^{bar}$	2HA10	2HA10	2HA10	2HA10
$S_t$ (cm)	20	20	20	20

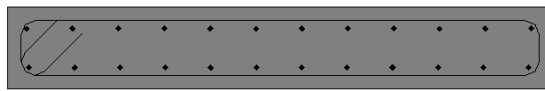


Figure III.27: Shear wall reinforcements schema.

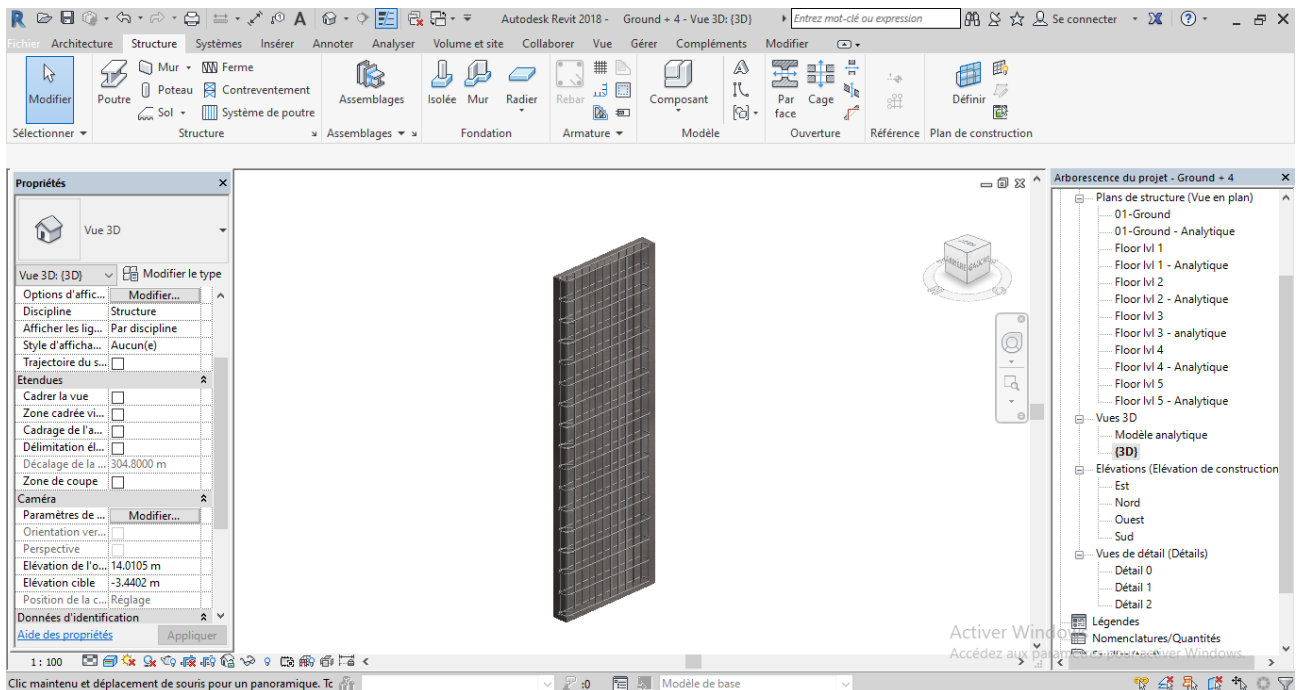


Figure III.28: Reinforced shear wall in 3D.

### III.7. Infrastructure study:

The infrastructure is the inferior part of a building resting on a sitting ground where all the loads supported by the building are transmitted. So, it represents the essential part of the building.

there are different types of foundations, which are:

- superficial foundation (isolated foundation, strip foundation, slab).
- Semi-deep foundation.
- Deep foundation (piled foundation).



Combinations:

$$\left\{ \begin{array}{l} G + Q \pm E \\ 0,8G \pm E \\ \text{ULS} \\ \text{SLS} \end{array} \right.$$

Foundation type choice:

We have:  $\frac{N_{ser}}{S} \leq \overline{\sigma_{sol}}$

$N$ : the normal effort transmitted at the base obtained from RSA 2018.

$$N = 14875,34 \text{ KN.}$$

$S$ : the structure's surface.  $S = A \times B$ .

$$S = 270,175 \text{ m}^2$$

$\overline{\sigma_{sol}}$ : admissible ground stress.  $\overline{\sigma_{sol}} = 2,1 \text{ bar}$

$$S_{nec} \geq \frac{N_{ser}}{\overline{\sigma_{sol}}} = \frac{14875,34}{2,1}$$

$$S_{nec} = 70,83 \text{ m}^2 \leq 50\%S$$

Pre dimensioning the isolated foundations:

$$A \times B = \frac{N}{\overline{\sigma_{sol}}}$$

$$\frac{a}{b} = \frac{A}{B} = 1 \rightarrow A = B \text{ (Square column)}$$

from which:  $B \geq \sqrt{\frac{N_s}{\overline{\sigma_{sol}}}}$

example of calculation:

$$N = 803,49 \text{ KN}$$

$$B \geq \sqrt{\frac{803,49}{210}} \rightarrow B \geq 1,96 \text{ m}$$

We take:  $B = 2 \text{ m}$

Since the smallest distance between axis is 3,05m. There is no overlap risk, so we take isolated base plate for foundation type.

Example of calculation of a Strip base under shear wall:

**Pre dimensioning SF<sub>1</sub>:**

In SLS:

$$\left\{ \begin{array}{l} N_1 = 217 \text{ KN} \\ N_2 = 396,98 \text{ KN} \\ N_3 = 181 \text{ KN} \end{array} \right.$$

$$N_{tot} = 794,98 \text{ KN}$$

$$L = 4,80 \text{ m}$$

$$B \geq \frac{N_{tot}}{\overline{\sigma_{sol}} \times L} = \frac{794,98}{210 \times 4,8} = 0,78 \text{ m}$$

$$B = 1 \text{ m}$$

$$h \geq \frac{B - b}{4} + 5 \text{ cm} = \frac{100 - 45}{4} + 5 = 18,75 \text{ cm}$$

$$h \geq \frac{L}{20} = \frac{480}{20} = 24 \text{ cm}$$

We take:  $h = 25 \text{ cm}$

**Reinforcements SF<sub>1</sub>:***Longitudinal reinforcements:*

$$N_{tot} = 794,98 \text{ KN}$$

$$N_{ml} = \frac{N_{tot}}{L} = \frac{795,98}{4,8} = 165,82 \text{ KN/m.}$$

$$A_{s/ml} = \frac{N_{ml} \times (B - b)}{8 \times d \times \sigma_s} = \frac{165,82 \times 10^3 (100 - 45)}{8 \times 20 \times 210} = 2,71 \text{ cm}^2$$

We take: **5HA12 = 5,65 cm<sup>2</sup>/ml**

$$S_t = 15 \text{ cm}$$

*Transversal reinforcements:*

$$A_t = \frac{5,65}{4} = 1,42 \text{ cm}^2$$

We take: **4HA8 = 2,51 cm<sup>2</sup>/ml**

$$S_t = 20 \text{ cm}$$

**Table III.50:** Calculation of strip foundation under shear walls.

FOUNDATION	SF <sub>1</sub>	SF <sub>2</sub>	SF <sub>3</sub>
<b>N<sub>tot</sub> (KN)</b>	794,98	777,79	755,08
<b>L (m)</b>	4,8	4	3,4
<b>B (m)</b>	1	1	1,10
<b>h (cm)</b>	25	25	25
<b>d (cm)</b>	5	5	5
<b>A<sub>s/ml</sub> (cm<sup>2</sup>)</b>	2,71	3,18	4,29
<b>BAR CHOICE</b>	5HA12=5,65cm <sup>2</sup>	5HA12=5,65cm <sup>2</sup>	5HA12=5,65cm <sup>2</sup>
<b>S<sub>t</sub> (cm)</b>	15	15	15
<b>A<sub>t</sub> (cm<sup>2</sup>)</b>	1,42	1,42	1,42
<b>BAR CHOICE</b>	4HA8=2,51cm <sup>2</sup>	4HA8=2,51cm <sup>2</sup>	4HA8=2,51cm <sup>2</sup>
<b>S<sub>t</sub> (cm)</b>	20	20	20

*Isolated foundation:***Pre dimensioning:**

$$B \geq \sqrt{\frac{N_s}{0,875 \times \sigma_{sol}}} = \sqrt{\frac{803,49}{0,875 \times 210}} = 2,09 \text{ m}$$

$$B = 1,05 \times 2,09 = 2,20 \text{ m}$$

$$A = 0,875 \times 2,20 = 1,95 \text{ m}$$

$$h_t \geq \frac{B - b}{4} + 5 \text{ cm} \geq 48,75$$

$$h_t = 50 \text{ cm}$$

$$e_0 = \frac{M}{N} = \frac{1,2}{803,49} = 0,0014 \text{ m}$$

$$e_0 = 0,0014 \text{ m} \leq \frac{B}{6} = 0,36 \text{ m}$$

**Reinforcements:**

$$e_0 = 0,0014 \text{ m} \leq \frac{B}{24} = 0,091 \text{ m}$$

$$Q' = N \left( 1 + \frac{3e_0}{B} \right) = 805,02$$

$$A_x = \frac{Q'(A - a)}{8d\sigma_{sol}} = \frac{805,02 \times (195 - 45)}{8 \times 45 \times 210} = 15,97 \text{ cm}^2$$

We take:  $8HA16 = 16,09cm^2$

$S_t = 20cm$

$$A_y = \frac{Q'(B - b)}{8d\sigma_{sol}} = \frac{805,02 \times (220 - 45)}{8 \times 45 \times 210} = 18,63cm^2$$

We take:  $10HA16 = 20,11cm^2$

$S_t = 22cm$

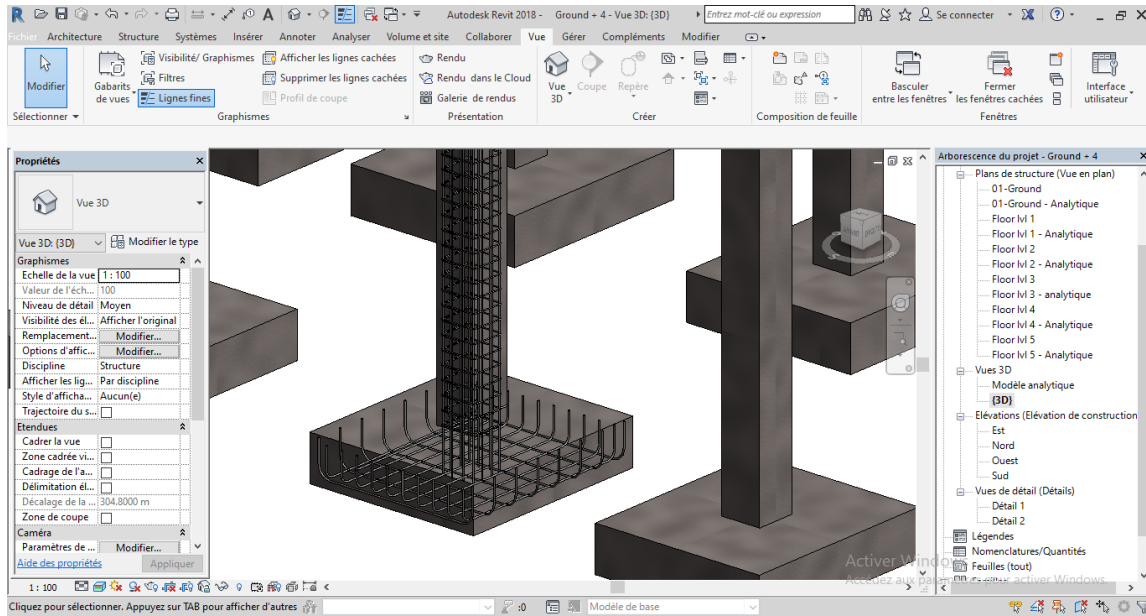


Figure III.29: Reinforced isolated foundation in 3D.

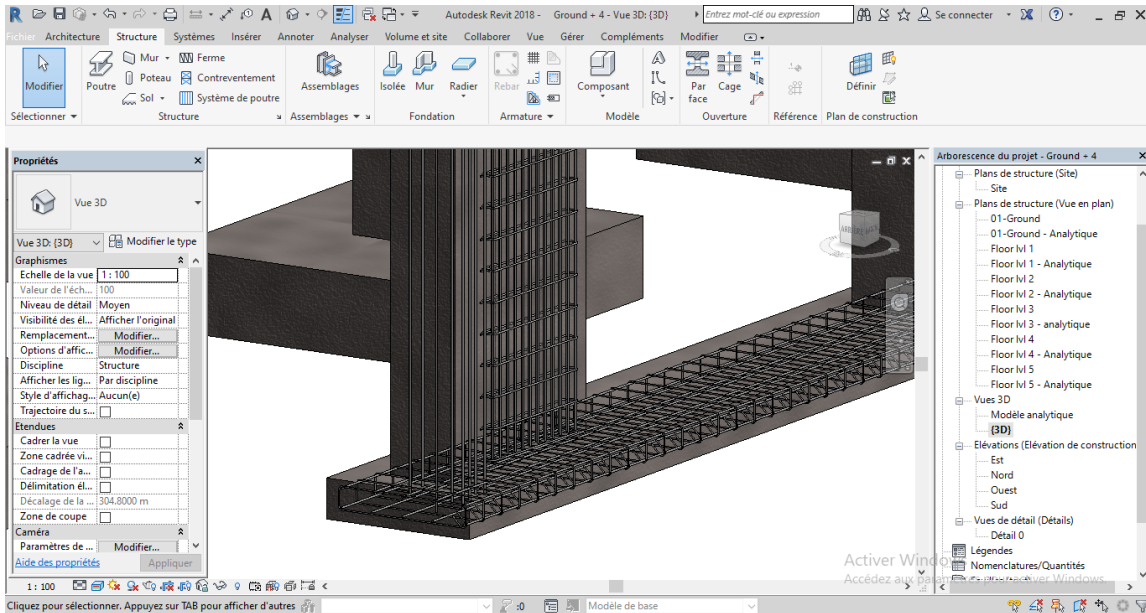


Figure III.30: Reinforced strip foundation in 3D.

## Conclusion

BIM definitely increases productivity in structural engineering, the construction documents are generated automatically reducing the time required with the detailing, it also helps preventing errors in the documentation that can affect the construction.

## GENERAL CONCLUSION

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Building Information Modeling has brought a big change to the AEC industry, this industry had constantly been looking for ways to produce faster and at less cost, and enhance the quality of the product. With BIM all the construction stakeholders work on one common digital model which gives them the chance to work together and better coordinate their work, plus this prevents the errors that are generated from the reinput of the building's information.

This increase of productivity was a significant benefit that BIM provided to structural engineers, it had promised a better conception with a better quality and detection of errors before the project is set in real world. The modulization in 3D improves the consistency and the precision of the study files. It had brought to the industry more clarity and less improvising in the construction site. More complexed projects that are more dependent in scheduling and prefabrication take great advantage from BIM, elements are coordinated more easily which improves the delivery, installation and handling.

BIM also presents the results of structural analysis in an easy and sharable way; the interoperability is one of the major pillars of this process. the project data should be transferred from one part to another without any loss of information or needing to reinput. This means the ability to transfer files either on the network or from a software to another. For this the actors of the construction have created IFC which determines the interoperability Open BIM.

the research that we have made in this thesis, allowed us to see that BIM is a tool that imposes sharing the digital information between the actors of the construction in a new working method. We were also able to learn the best ways of achieving the interoperability with structural analysis software's. And we had the chance to learn and work with Autodesk Revit which is a BIM software that gives us the ability to create a virtual model that is similar to reality. The model is created in 3D and we derive from it 2D plans and cuts and reinforcement details and much more.

The BIM process presents numerous advantages for the AEC sector. The Algerian building sector should adapt this method, because it brings with no doubt a big change to the industry. It is also necessary to integrate it in high education to form more efficient civil engineers. Both engineers and architects should master the new technology to be more creative with their works.

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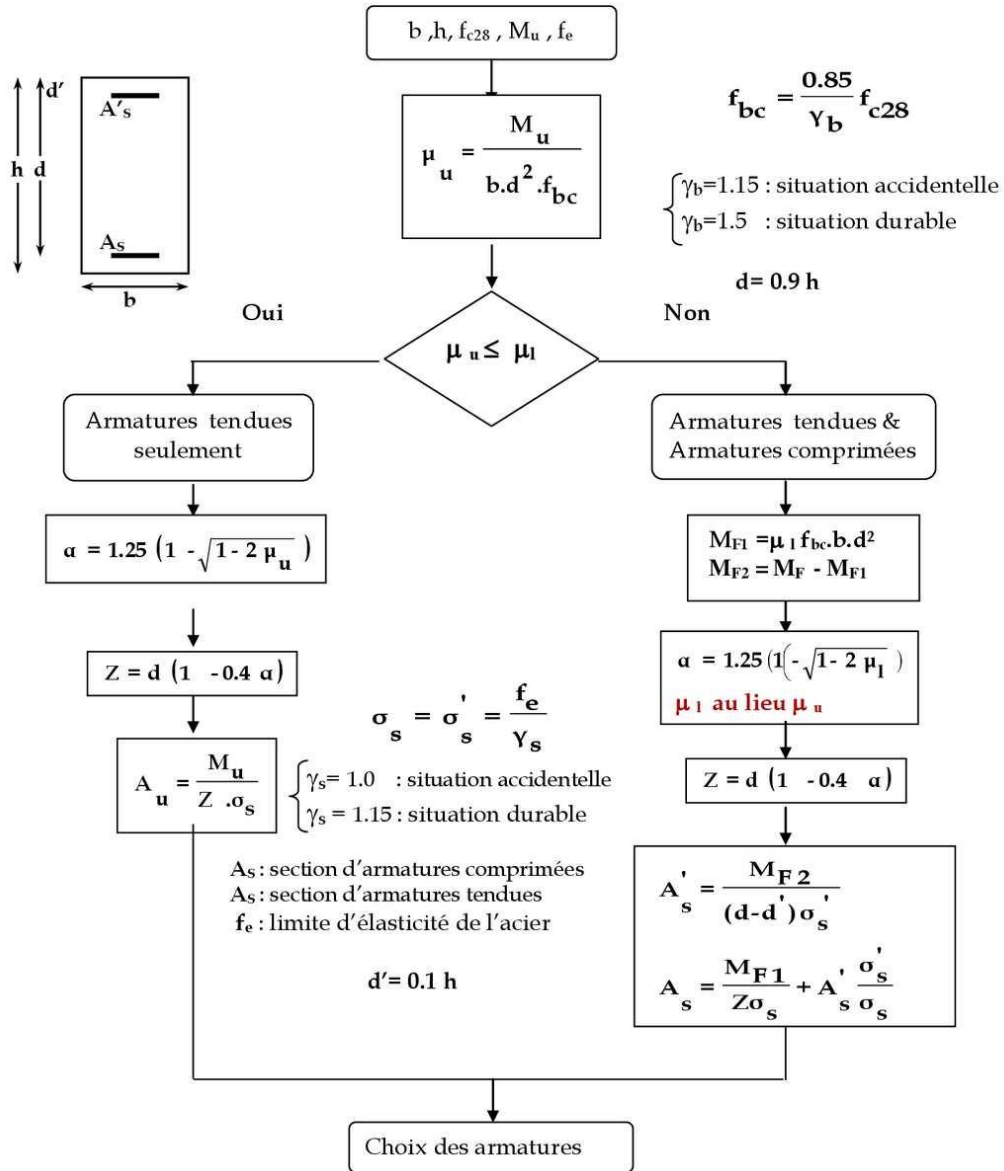
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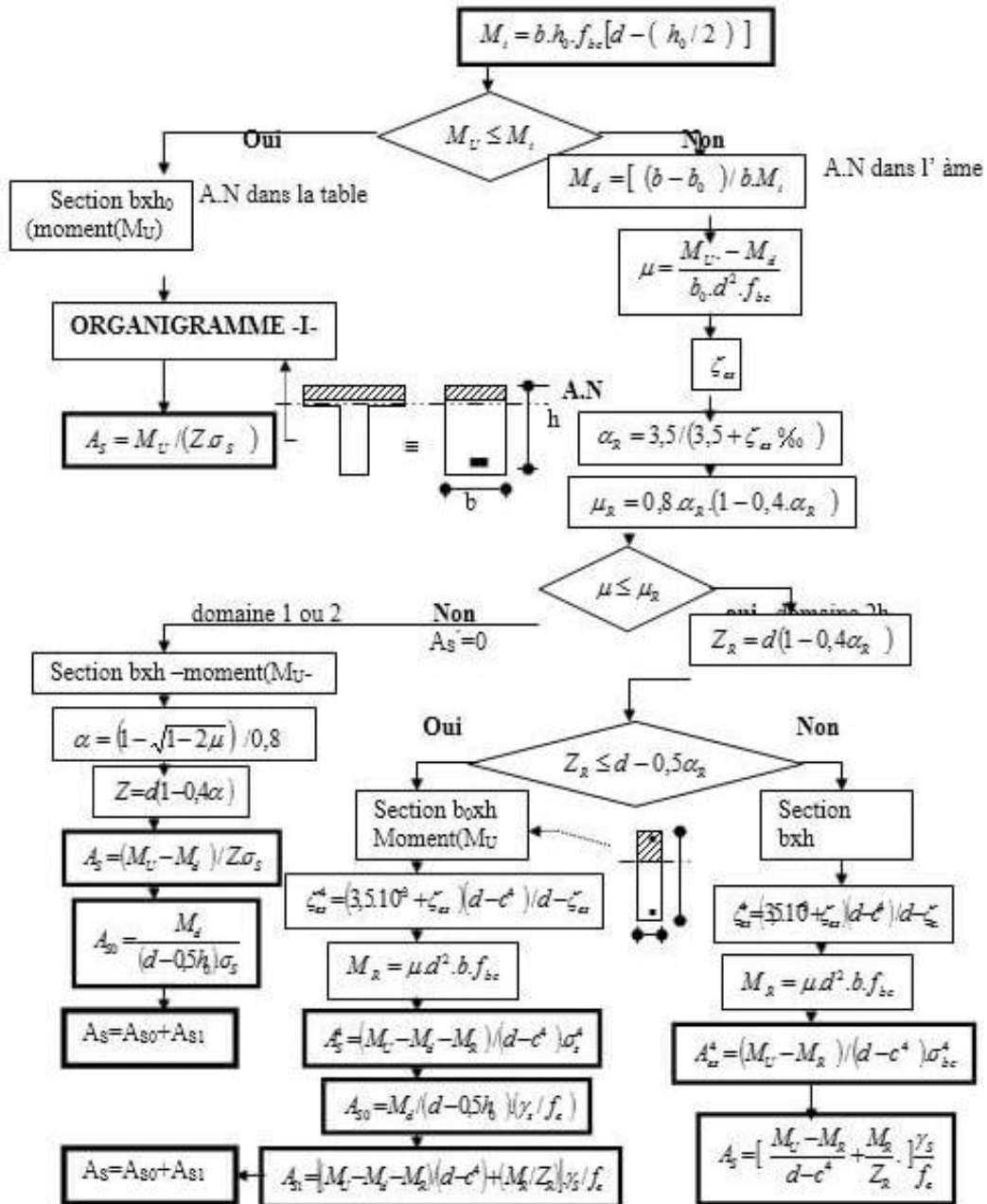
# APPENDIX A

Organigramme pour le calcul d'une section rectangulaire d'une poutre en flexion simple à l'état limite ultime



## APPENDIX B

### ORGANIGRAMME -II- CALCUL D'UNE SECTION EN -TE- A L'E.L.U EN FLEXION SIMPLE





## APPENDIX C

### REAL REINFORCEMENT SECTIONS

Section in cm<sup>2</sup> of N the diameter of the reinforcement  $\phi$  in mm.

$\phi$ :	5	6	8	10	12	14	16	20	25	32	40
1	0.20	0.28	0.50	0.79	1.13	1.54	2.01	3.14	4.91	8.04	12.57
2	0.39	0.57	1.01	1.57	2.26	3.08	4.02	6.28	9.82	16.08	25.13
3	0.59	0.85	1.51	2.36	3.39	4.62	6.03	9.42	14.73	24.13	37.70
4	0.79	1.13	2.01	3.14	4.52	6.16	8.04	12.57	19.64	32.17	50.27
5	0.98	1.41	2.51	3.93	5.65	7.70	10.05	15.71	24.54	40.21	62.83
6	1.18	1.70	3.02	4.71	6.79	9.24	12.06	18.85	29.45	48.25	75.40
7	1.37	1.98	3.52	5.50	7.92	10.78	14.07	21.99	34.36	56.30	87.96
8	1.57	2.26	4.02	6.28	9.05	12.32	16.08	25.13	39.27	64.34	100.53
9	1.77	2.54	4.52	7.07	10.18	13.85	18.10	28.27	44.18	72.38	113.10
10	1.96	2.83	5.03	7.85	11.31	15.39	20.11	31.42	49.09	80.42	125.66
11	2.16	3.11	5.53	8.64	12.44	16.93	22.12	34.56	54.00	88.47	138.23
12	2.36	3.39	6.03	9.42	13.57	18.47	24.13	37.70	58.91	96.51	150.8
13	2.55	3.68	6.53	10.21	14.70	20.01	26.14	40.84	63.81	104.55	163.36
14	2.75	3.96	7.04	11.00	15.83	21.55	28.15	43.98	68.72	112.59	175.93
15	2.95	4.24	7.54	11.78	16.96	23.09	30.16	47.12	73.63	120.64	188.5
16	3.14	4.52	8.04	12.57	18.10	24.63	32.17	50.27	78.54	128.68	201.06
17	3.34	4.81	8.55	13.35	19.23	26.17	34.18	53.41	83.45	136.72	213.63
18	3.53	5.09	9.05	14.14	20.36	27.71	36.19	56.55	88.36	144.76	226.2
19	3.73	5.37	9.55	14.92	21.49	29.25	38.20	59.69	93.27	152.81	238.76
20	3.93	5.65	10.05	15.71	22.62	30.79	40.21	62.83	98.17	160.85	251.33

## DICTIONARY

Reinforcement: armature.  
Reinforced concrete: béton armé.  
Construction site: chantier.  
Allowable or safe load: charge admissible.  
Dead load: charge permanente.  
Live load: charge d'exploitation.  
Seismic load: charge sismique.  
Shear: cisaillement.  
Shear force: effort tranchant.  
Formwork: coffrage.  
Ultimate stress: contrainte limite.  
Riser: contremarche.  
Hook: crochet.  
Slab: dalle.  
Hardy slab or hollow core: dalle en corps creux.  
Staircase: escalier.  
Column: poteau.  
Beam: poutre.  
Acroterion: acrotère.  
Shear wall: voile.  
Landing beam: poutre palière.  
Strip baseplate: semelle filante.  
Isolated baseplate: semelle isolée.  
Ground floor: rez-de-chaussée.  
Slenderness: élancement.  
The arrow: la flèche.  
Span: travée.  
Slope: pente.  
Compound bending: flexion compose.  
Tensile: traction.  
Floor: plancher.  
Supports: appuis.  
ULS (Ultimate Limit State): ELU.  
SLS (Serviceability Limit State): ELS.  
Buckling length: longueur de flambement.

## **ACRONYMS AND DEFINITIONS**

AEC: Architecture Engineering and Construction.

BIM: Building Information Modeling.

BIM: Building Information Model.

CAD: Computer Aided Design.

IFC: Industry Foundation Classes.

LOD: Level of Detail or Development.

HVAC: Heating, Ventilation and Air Conditioning.

ICT: Information and Communications Technology.

BDS: Building Description Systems.

GLIDE: Graphical Language for Interactive Design.

BPM: Building Product Model.

GBM: Generic Building Model.

MIT: Massachusetts Institute of Technology.

IAI: The International Alliance for Interoperability.