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DEPARTMENT OF CIVIL, ARCHITECTURAL AND ENVIRONMENTAL ENGINEERING

Building Information Modelling (BIM) applied to the structural assessment of a multi-story reinforced concrete building: case study of the new administrative building of NASPW

A thesis submitted in partial fulfilment of the requirements for the degree of Master of Engineering (MEng) in Civil Engineering

Curriculum: **Structural Engineering**

Presented by:

TCHAHEU TCHAHEU IDRISS

Student number: 15TP20975

Supervised by:

Dr Paolo Zampieri

Co – supervised by :

Dr Paolo Borin

Academic year: 2019/2020

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DEDICATION

I dedicate this work to my lovely family…

ACKNOWLEDGEMENTS

I would wish, through this thesis to express special thanks to

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LIST OF ABBREVIATIONS

Ψ_0	Is the factor of rare or combination value
Ψ_1	Factor for the frequent value of a variable action
Ψ_2	Factor for the quasi-permanent value of a variable action
ΔC_{dev}	Allowance in design for deviation
σ_c	Concrete stress
λ	Slenderness or correction factor
λ_{min}	Limit slenderness
γ_c	The partial safety factor for concrete
ε_{cu2}	The ultimate strain
$A_{s,min}$	Minimum allowable area of the section
A_s	Area of steel reinforcements
A_{sl}	Area of longitudinal steel reinforcements
C_{com}	Nominal cover
C_{dr}	Directional factor
$C_e(z)$	The exposure factor at the height z
$C_{min,b}$	Minimum cover due to bond requirement
$C_{min,dur}$	Minimum cover due to environmental conditions
C_{min}	Minimum cover
$C_{pe,1}$	Local coefficients²
$C_{pe,10}$	Overall coefficients
C_{pe}	External pressure coefficients

C_{se}	Season factor
E_c	Longitudinal elastic modulus of composite material
E_{cm}	Modulus of elasticity of concrete in N/mm2
E_s	Modulus of elasticity of steel
M_{Ed}	Acting bending moment
M_{Rd}	Resisting bending moment
M_{sls}	Axial load at SLS
N_{uls}	Axial loads at ULS
V_{Ed}	Acting shear force
$V_{Rd,max}$	Maximum allowable shear resistance
V_{Rd}	Shear resistance
V_{Rd}	Shear strength
f_{cd}	Design concrete compressive strength, N/mm2
$f_{ck,cube}$	Concrete cubic compressive strength
f_{ck}	Concrete cylindrical compressive strength, N/mm2
f_{ctm}	Mean value of tensile strength of concrete
f_{yd}	Design yield strength of steel reinforcement, N/mm2
f_{yk}	Yield strength of steel reinforcement, N/mm2
f_{ywd}	Design yield strength of the shear reinforcement
l_e	Effective height of the column

q_k	Uniformly distributed load
q_p	Peak velocity pressure
q_p(z)	The peak velocity pressure at height z
u_0	Perimeter of the column
u_1	Basic control perimeter
2D	2-Dimensional drawing
3D	3-Dimensional drawing, geometric modelling
AEC	Architecture, Engineering and Construction
BIM	Building Information Modeling
CIS/2	CIMSteel Integration Standards Version 2
FFB	Fédération Française du Bâtiment
IAI	International Alliance for Interoperability
IFC	Industry Foundation Classes
ISO	International Organization for Standardization
CAD	Computer Aided Design
CIMsteel	Computer Integrated Manufacturing for constructional steelwork
d	Effective depth
HVAC	Heating, ventilation and air conditioning
MEP	Mechanical, Electrical and Plumbing
SLS	Serviceability limit state
ULS	Ultimate limit state

ABSTRACT

The main objective of this work was to carry out the structural design of a reinforced concrete multi-storey building using the BIM process. To achieve this goal, the new administrative building of the National Advanced School of Public Works consisting of a reinforced concrete office storey building for office use was analysed and designed according to Eurocode 2 using a BIM process. The frame structure of the building was made up of beams with sections 25x40cm, columns of sections 25x40cm whose concrete class is C25/30 was modelled using the software Autodesk Revit. Then, interoperability between Revit and Midas Gen software was performed to transfer the structural model created with Revit to Midas Gen using the midas gen text file format (.mgt). Once the structural model was available in Midas Gen, the loads and load combinations were added to the model. The internal forces (axial forces, moments and shear forces) were used to design structural elements such as columns, beams and shallow foundations using an Excel spreadsheet. The structural design of the ribbed hollow block floor (16+4) cm and the staircase were carried out. Using Revit, data obtained from the structural design allowed to produce structural design deliverables such as the volumes of concrete for structural elements which was a total of 388m^3 of concrete and also 2D and 3D steel reinforcement drawing associated with bar bending details and bar bending schedule of structural elements.

Keywords: BIM, interoperability, structural design**,** reinforced concrete building.

RESUMÉ

L'objectif principal de ce travail était de réaliser la conception structurelle d'un bâtiment à étages en béton armé en utilisant le processus Building Information Modeling. Pour atteindre cet objectif, le nouveau bâtiment administratif de l’école nationale supérieure des travaux publics qui est un bâtiment en béton arme est analysé et dimensionné selon Eurocode 2 en utilisant le processus BIM. La structure du bâtiment constituée de poutres de sections 25x40cm, poteaux de sections 25x40cm dont la classe du béton est C25/30 a été modélisée à l’aide du logiciel Revit. Ensuite l’interopérabilité entre le logiciel Revit et Midas Gen a ete effectué pour transférer le modèle structurel créé avec Revit vers Midas Gen en utilisant le format de fichier Midas Gen texte (.mgt). Une fois que le modèle structurel a été disponible dans le logiciel Midas Gen, les charges et combinaisons de charges ont été ajoutées au modele.Ensuite une analyse linéaire statique a été effectuée. Les sollicitations telles que l’effort axial, le moment et l’effort tranchant obtenus ont été utilisées pour effectuer le dimensionnement des éléments structurels tels que poteaux, poutres, et fondations superficielles à l’aide des feuilles Excel. La dalle a corps creux (16+4) et l’escalier ont été également analysés et dimensionnés. A l’aide du logiciel Revit, les résultats obtenus suite au dimensionnement structurel ont permis de produire des livrables tels que les volumes de béton pour les éléments structuraux avec un volume totale de 388 m^3 de béton et egalement les plans de ferraillage 2D et 3D associés aux détails de façonnages des armatures et quantités d’armatures des éléments structuraux.

Mots clés: BIM, interopérabilité, dimensionnement, bâtiment en béton armé.

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GENERAL INTRODUCTION

The technology which underpinned the working process in Architecture, Engineering and Construction (AEC) industry is in constant evolution over the time around the world. Before 1980s, documentation was made by manual 2D drafting with hand calculations being done by stakeholders of the construction industry exchanging information only on papers. Then, since 1980s, Computer Aided Design (CAD) technology also know as the traditional process use computer software to create independent documentation solving the problems of hand calculations and manual drafting. However, the traditional process is facing multiple challenges reducing the productivity which is solved using Building Information Modelling (BIM) over the past two decades. BIM has revolutionised the working process in the construction sector and improved productivity by bringing all the stakeholders of the AEC industry to collaborate with each other. This process is applied in various developed countries worldwide and considerably varying in African countries with South Africa, Nigeria and Egypt beign the leaders (Matarneh & Hamed, 2017). Cameroon can improve the productivity of its construction industry through the use of BIM, meanwhile Mbarga & Mpele, (2019) highlighted that BIM is not yet used for construction projects in the country.

The AEC industry worldwide is facing multiple difficulties like extensions of deadlines, budget overruns and many non-qualities during the realisation of a project (Eastman et al., 2011). According to Liu et al. (2016), structural analysis and construction documents are usually two independent stages, where multiple construction documents are created separately without any relationship or linkage. This creates difficulties in the coordination and update of structural drawings and bill of quantities, because of successive modifications. Structural drawings are usually available in 2D which require a lot of effort to understand and lead to a high probability of misunderstanding of the design intent. Faced with these different challenges, BIM transforms the process of working without fundamentally changing the deliverables of the project (Kim et al., 2016). The BIM offers the possibility of sharing information with stakeholders between project phases which leads to a reduction of manual re-entery data, opportunities of errors and allows the re-use of digital information (Borrmann et al., 2018). In the BIM process, structural drawings and bill of quantities are generated from the final structural BIM model and are automatically updated when the BIM model is changed, which save hours of work and maintain a high level of data accuracy stored. The use of BIM increase the productivity of structural engineering practices from 15% to 41% as cited in (Arayici, 2015).

The objective of this work is to perform the structural design of a reinforced concrete building using the BIM process, case study being the new administrative building of the National Advanced School of Public Works. This objective is attained by creating a structural BIM model which is use in the structural design and production of structural deliverables like 2D and 3D steel reinforcement drawings including accurate quantity take off of reinforced concrete elements.To achieve this objective, the work is divided into three chapters. The first chapter starts with the generalities on concrete and steel materials, followed by the knowledge on reinforced concrete buildings with basic concepts on BIM. The second chapter presents the methodology used to attain the main objective. Finally, chapter three presents the structural design results of the applied methodology to the new administrative building of the National Advanced School of Public Works.

CHAPTER 1: LITERATURE REVIEW

Introduction

The Building Information Modelling (BIM) concept originated in academic research is nowadays established in the construction industry. BIM influence the process of performing structural analysis and design of buildings and also the production of technical drawings. To design a reinforced concrete structure using the BIM process, it is important to know the concrete and steel materials, the reinforced concrete structural elements, the types of analysis used, the standards used to design a building and an understanding of the BIM concept. Thus this chapter begins with a presentation of concrete and steel materials with emphasis on the constituents of concrete, the mechanical properties of concrete, the types of steel used in reinforced concrete and the mechanical properties of steel, then shifting the focus on a reinforced concrete building by emphasizing the classification of a reinforced concrete building, knowledge on structural reinforced concrete elements, types of analysis which can be performed on reinforced concrete and the different standards used to perform the design of a reinforced concrete building. The chapter ends with an overview of BIM including the definition of BIM, its origin, the dimensions of BIM, the levels of BIM, the contribution of BIM in the construction industry, the BIM initiatives around the world for its adoption and implementation, interoperability in the construction industry, formats of exchange data in BIM process and BIM in the structural design phase.

1.1. Reinforced concrete

Reinforced concrete is the winning combination of concrete and steel reinforcements. When combining concrete and steel, steel provides the tensile strength and probably some of the shear strength while the concrete, strong in compression protects the steel to give it durability and fire resistance(Mosley et al., 2012).

1.1.1. Concrete

Concrete is an interesting building material, every year in the world about 7 billion cubic meters of concrete are used(Granju, 2012).

1.1.1.1. Constituents

Concrete is a composite material made from a mixture of aggregates, cement, water and possible admixtures.

a. Cement

Cement is a binder, a substance used for construction that sets, hardens, and adheres to other materials to bind them together(Granju, 2012). During the formulation of the concrete mix, water added react with the cement which hardens and binds the aggregates into the concrete matrix; the concrete matrix sticks or bonds onto the reinforcing bars.

The ENV 197-1 standard specified that there are 27 products in the family of common cement and that they are grouped into five main types of cement all of which are mixtures of different proportions of clinker and another major constituent. The five groups are as follows, CEM I (Portland cement); CEM II (Portland-composite cement); CEM III (Blast furnace cement); CEM IV (Pozzolanic cement) and CEM V (Composite cement). According to ENV 197-1 standard, the standard strength of a cement is the compressive strength determined at 28 days and shall conform to the requirements in table 1.1. It distinguishes three classes of standard strength available for cement, class 32.5, class 42. 5 and class 52.5. Also, for each standard strength class, it distinguishes two classes of early strength N and R.

Table 1. 1. Mechanical and physical requirements of cement according to ENV 197-1

Strength class	Compressive strength MPa				Initial setting time	Soundness (expansion)
	Early strength		Standard strength			
	2 days	7 days	28 days		min	mm
32,5 N	-	≥ 16,0	≥ 32,5	≤ 52,5	≥ 75	≤ 10
32,5 R	≥ 10,0	-				
42,5 N	≥ 10,0	-	≥ 42,5	≤ 62,5	≥ 60	
42,5 R	≥ 20,0	-				
52,5 N	≥ 20,0	-	≥ 52,5	-	≥ 45	
52,5 R	≥ 30,0	-				

b. Aggregate

Aggregate is the name given to any inert material resulting from the erosion of rocks or their crushing, used in construction in general and entering into the formulation of mortars and concrete.

The aggregates occupy about 70 % of the volume of the material and are directly involved in both fresh and hardened concrete(Michel, 2005). Aggregates in concrete are sand and gravel which are bound together by cement. Aggregate is classed into two sizes, Coarse aggregate which consists of gravel or crushed rock 5 mm or larger in size and fine aggregate which consist of sand less than 5 mm in size. The Natural aggregates are classified according to the rock type, e.g. basalt, granite, flint and limestone. To obtain a dense strong concrete with minimum use of cement, the cement paste should fill the voids in the fine aggregate while the fine aggregate and cement paste fill the voids in the coarse aggregate. The EN 12620 standard defines the classification, geometric characteristics, physical characteristics, chemical characteristics and durability of aggregate to obtain high-quality concrete.

c. Water

The water used for making concrete must be clean, it plays a triple role in making concrete which is, wet the surface of the aggregates so that the cement paste can adhere to it; allow hydration of the cement powder and Promote the pouring of concrete (Michel, 2005).

The final quality of mortar and concrete depends indirectly on the cement content. It depends on the ratio of cement(C)/water (W), which is an essential factor in obtaining good concrete. The mechanical strength of good concrete is proportional to the cement/water ratio as well as the resistance to wear and tear reduced shrinkage and creep, and better protection of the reinforcement. These improvements increase with the C/W ratio as long as the ratio does not exceed 2.5. Beyond this limit, the strength decreases and the concrete becomes too dry(Michel, 2005).

d. Admixtures

Concrete admixtures are natural or manufactured chemicals or additives (accelerating, delaying, and fluidifying) added during concrete mixing to enhance specific properties of the fresh or hardened concrete, such as workability, durability, or early and final strength. The quantity of admixture is not more than 5% by mass of the cement content of the concrete(Bhatt et al., 2014).

1.1.1.2. Concrete properties

The properties of concrete are influenced by many factors mainly due to the mixed proportion of cement, sand, aggregates and water. The exist various concrete properties such as workability, mechanical resistances, shrinkage, creep and durability.

a. Workability

The workability of a concrete mix gives a measure of the ease with which fresh concrete can be placed and compacted. Adequate concrete workability enables fresh concrete to flow readily into the form and go around and cover the reinforcement, the mix retains its consistency and the aggregates do not segregate. A mix with high workability is needed where sections are thin and/or reinforcement is complicated and congested. The factors affecting workability are the water content of the mix, plasticizing admixtures which increase workability, the size of aggregate, its grading and shape, the ratio of coarse to fine aggregate.

The measurement of the workability is performed using the slump test which consists of putting the fresh concrete into a standard cone which is lifted off after filling and the slump is measured. The slump is in the range of 25–50 mm for low workability, 50–100 mm for medium workability and 100–175 mm for high workability.

b. Mechanical resistances

The mechanical resistances of the concrete refers to the concrete strength which can be identified in terms of compressive strength of concrete and tensile strength of concrete.

i. Compressive strength

Concrete compressive strength is the most important property of concrete. It depends on the type of the cement and generally increases with age, the European code allows the measurement of the compressive strength of concrete by crushing a cylinder (50 mm diameter and 300 mm height) or cubic sample (150 mm side) of concrete at age 28 days after the curing. The compressive strength of concrete is denoted by concrete strength classes which are specified in terms of characteristic cylinder strength (f_{ck}) and characteristic cube strength $f_{ck;\ cube}$. The cylinder strength is on average about 0.8xthe cubic strength. The EN1992-1-1 allow the use of concrete of strength up to 90MPa in normal-weight concrete and 80 Mpa in lightweight concrete, all design calculations using EC2 are based on the characteristic cylinder strength (f_{ck}).

ii. Tensile strength

The tensile strength of concrete is about 10 per cent of the compressive strength. It is determined by loading a concrete cylinder across a diameter. The mean characteristic tensile strength f_{ctm} is related to mean cylinder compressive strength f_{cm} as expressed in equations (1. 1), (1. 2) and (1. 3).

$$f_{ctm} = 0.3\ x f_{ck}^{\frac{2}{3}} \quad for \quad f_{ck} \leq 50\ MPa \qquad (1.\ 4)$$

$$f_{ctm} = 2.12 x ln\left[1 + \frac{f_{cm}}{10}\right] \quad for \ f_{ck} > 50\ MPa \qquad (1.\ 5)$$

$$f_{cm} = f_{ck} + 8MPa \qquad (1.\ 6)$$

c. Shrinkage

Shrinkage is a spontaneous shortening largely due to the evaporation of a part of water contained in the concrete(Granju, 2012). The total shrinkage strain is composed of two parts, the drying shrinkage strain and the autogenous shrinkage strain. Drying shrinkage strain is the contraction that occurs in concrete when it dries and hardens. Drying shrinkage develops slowly due to migration of water and is irreversible but alternate wetting and drying causes expansion and contraction of concrete. The autogenous shrinkage strain develops during the hardening of concrete and develops quite fast during the early days after the casting of concrete(Bhatt et al., 2014).

Shrinkage begins to take place as soon as the concrete is mixed, and is caused initially by the absorption of the water by the concrete and the aggregate. Further shrinkage is during the setting process the hydration of the cement causes a great deal of heat to be generated, and as the concrete cools, further shrinkage takes place as a result of thermal contraction. Even after the concrete has hardened, shrinkage continues as drying out persists over many months, and any subsequent wetting and drying can also cause swelling and shrinkage(Mosley et al., 2012). Shrinkage depends on several factors, the ambient humidity, the dimension of the element, the composition of the concrete and the age of the concrete at that time.

c. Creep

Creep is the continuous deformation of a member under sustained load. It is a phenomenon associated with many materials, but it is particularly evident with concrete. Creep depend on several factors, the ambient humidity, the dimension of the element, the composition of the concrete and the age of the concrete at that time. Its effects are particularly important in beams, where the increased deflections may cause the opening of cracks, damage to finishes and the non-alignment of mechanical equipment. The provision of the reinforcement in the compressive zone of a flexural member, however, often helps to restrain the deflections due to creep.

d. Durability

The durability of the concrete is influenced by the exposure conditions, the cement type, the concrete quality, the cover to the reinforcement and the width of any cracks. The severity of the exposure governs the type of concrete mix required and the minimum cover to the reinforcing steel. Adequate cover is essential to prevent corrosive agents from reaching the reinforcement through cracks and previous concrete. The cover is necessary to protect the reinforcement against a rapid rise in temperature and subsequent loss of strength during a fire.

1.1.2. Steel reinforcement

Steel reinforcement can provide the tensile strength to carry tensile forces and also shear strength, steel reinforcement products are with a circular or practically circular cross-section.

1.1.2.1. Types

The common types of steel bars used in the reinforced concrete structure are hot rolled deformed bars and cold-worked steel reinforcement.

The hot rolled deformed bars have deformation (ribs, lugs and indentation) on the surface of the bar, which reduces the major problem of good bonding between concrete. The stress-strain curve presents a separate yield point accompanied by a plastic stage in which strain is raised without raising the stress. It is succeeded by a strain hardening stage as shown in the image (a) of figure 1.1. The cold worked steel reinforcement is obtained by twisting or drawing the bars at room temperature. The Plastic stage in the stress-strain curve is reduced as shown in the image (b) of figure 1.1. The ductility of bars is lower as compared to hot-rolled bars.

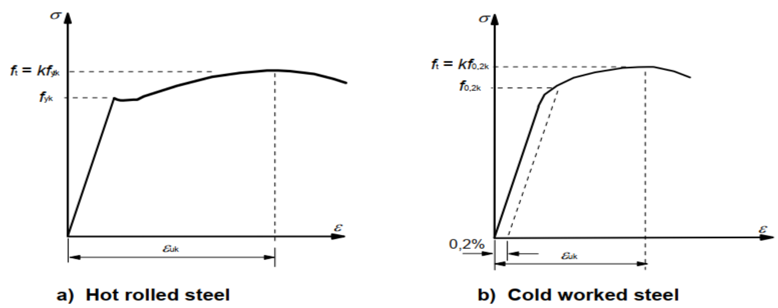


Figure 1. 1. Steel reinforcement Stress-strain curves from BS.EN1992-1-1

1.1.2.2. Mechanical properties

The mechanical properties consist of steel strength and ductility

a. Steel strength

The steel strength is identified by the yield strength f_{yk} and the tensile strength f_{tk} which are defined respectively as the characteristic value of the yield load, and the characteristic maximum load in direct axial tension, each divided by the nominal cross-sectional area. Eurocode 2 permits the use of reinforcement of up to grade 600MPa.

b. Ductility

Ductility is the ability of a material to be drawn or plastically deformed without fracture. Reinforced concrete steel must have adequate ductility, as defined by the ratio of the tensile strength through yield strength (f_{tk} / f_{yk}) and elongation ε_{uk} under maximum load. Its advantage is to be taken off the plastic behaviour of structures. The greater the ductility, the greater the elongation in axially loaded members, and the greater the rotation capacity in members subjected to flexure. In general, ductility is inversely related to yield stress. The values of ε_{uk} and (f_{tk} / f_{yk}) are provided in table 1.2 for the different classes of ductility A, B and C.

Table 1. 2. A different class of ductility of steel reinforcement (BS- EN-1992-1-1)

Product form	Bars and de-coiled rods			Wire Fabrics			Requirement or quantile value (%)
Class	A	B	C	A	B	C	-
Characteristic yield strength f_{yk} or $f_{0,2k}$ (MPa)	400 to 600						5,0
Minimum value of $k = (f_t/f_y)_k$	≥1,05	≥1,08	≥1,15 <1,35	≥1,05	≥1,08	≥1,15 <1,35	10,0
Characteristic strain at maximum force, ε_{uk} (%)	≥2,5	≥5,0	≥7,5	≥2,5	≥5,0	≥7,5	10,0

Ductility class A is considered as the lowest ductility category, ductility class B is the most commonly used for reinforcing bars and ductility class C is the highest ductility class that may be used in earthquake design or similar situations.

1.2. Reinforced concrete building

A reinforced concrete building can be classified as a single-storey building that consist of a ground storey only or a multi-storey building which is a building that has multiple storeys.

1.2.1. Classification of multi-storey reinforced concrete building

A multi-storey reinforced has multiple storeys and typically contains vertical circulation in the form of ramps, stairs and lifts. A multi-storey reinforced concrete building can be classified according to the intended use of the building, the height of the building and also the structural types.

1.2.1.1. Classification according to the intended use

The Eurocode1 Part 1 (EN 1991-1-1) identifies different types of buildings which are residential; social; commercial; administrative and industrial. It can be seen that the classification is based on the functional use of the building. The BS-EN-1991-part 1-1 provide the classification of areas in buildings into categories based on the anticipated use of these areas in the building. Table 1.3 present the categories of areas in a building.

Table 1. 3. Categories of areas in the building (BS-EN-1991-part 1-1, 2002)

Category	Specific Use	Example
A	Areas for domestic and residential activities	Rooms in residential buildings and houses; bedrooms and wards in hospitals; bedrooms in hotels and hostels kitchens and toilets.
B	Office areas	
C	Areas where people may congregate (with the exception of areas defined under category A, B, and D[1)])	**C1**: Areas with tables, etc. e.g. areas in schools, cafés, restaurants, dining halls, reading rooms, receptions. **C2**: Areas with fixed seats, e.g. areas in churches, theatres or cinemas, conference rooms, lecture halls, assembly halls, waiting rooms, railway waiting rooms. **C3**: Areas without obstacles for moving people, e.g. areas in museums, exhibition rooms, etc. and access areas in public and administration buildings, hotels, hospitals, railway station forecourts. **C4**: Areas with possible physical activities, e.g. dance halls, gymnastic rooms, stages. **C5**: Areas susceptible to large crowds, e.g. in buildings for public events like concert halls, sports halls including stands, terraces and access areas and railway platforms.
D	Shopping areas	**D1**: Areas in general retail shops **D2**: Areas in department stores

[1)] Attention is drawn to 6.3.1.1(2), in particular for C4 and C5. See EN 1990 when dynamic effects need to be considered. For Category E, see Table 6.3

NOTE 1 Depending on their anticipated uses, areas likely to be categorised as C2, C3, C4 may be categorised as C5 by decision of the client and/or National annex.

NOTE 2 The National annex may provide sub categories to A, B, C1 to C5, D1 and D2

NOTE 3 See 6.3.2 for storage or industrial activity

1.2.1.2. Classification according to the height

The classification of multi- storey buildings according to the height include low- rise buildings; mid- rise buildings; high-rise buildings; skyscraper buildings; super tall buildings and mega tall buildings as presented in table 1.4.

Table 1. 4. Classification of building according to the height

Types of buildings	Description
Low–rise buildings	A building which is not tall enough to be classified as high- rise building
Mid-rise buildings	A building to five to ten storeys, equipped with lifts
High-rise buildings	A building with more than 7 to 10 storeys
Skyscraper buildings	A building with 40 storeys or more
Super-tall buildings	A building with a height exceeding 300 m
Mega tall buildings	A building with a height exceeding 600 m

1.2.1.3. Classification according to the structural type

The basic structural types of a multi- storey reinforced concrete building include, framed structure; suspended structure; cantilever structure; braced structure; shear wall structure; core structure and hull core structure. These different structural types may be used in combination in a building. Table 1.5 presents the description of the different structural types in a building.

Table 1. 5. Different structural types of reinforced concrete building

Structural types	Description
Framed structure	Consist of a network of columns and connecting beams forming the structural skeleton of the building and carry loads to the foundations.
Suspended structure	Has an internal core and horizontal floors which are supported by high-steel cables hung from cross beams at the top.
Cantilever structure	Has an internal core from which beams and floors cantilever. This removes the necessity of columns.

Braced structure	Bracing is used to give stability so that columns can be designed as pure compression members, the beams and columns carry vertical loads and the bracing system carries the lateral loads.
Shear wall structure	Composed of stiff braced (or shear) panels that counter the effects of lateral and wind loads.
Core structure	Utilises a stiff structural core that houses lifts and staircase. Wind and lateral pressures are transmitted to the core by the floors.

1.2.2. Structural reinforced concrete building elements

A building is generally composed of a superstructure above the ground and a substructure that forms the foundations below the ground. The complete building structure can be broken down into the following elements: beams, columns, slabs, walls and foundations.

1.2.2.1. Super structural reinforced concrete elements

A reinforced concrete structure is a combination of beams, columns, slabs and walls rigidly connected to form a monolithic frame(Mosley et al., 2012)

a. Horizontal elements

The horizontal structural elements in a building such as beams and slabs carry the loads and transfer them to the vertical structural elements.

i. Beams

Beams are horizontal members that span columns, they primarily resist loads applied laterally to the beam's axis and transfer the loads to the columns supporting them. Its mode of deflection is primarily by bending. Beams are characterized by their cross-section profile shape, equilibrium conditions, length, and material. A horizontal structural element is classified as a beam if the span is not less than 3 times the overall section depth(Thonier, 2006). Figure 1.3 present the schematic representation of a simply supported beam.

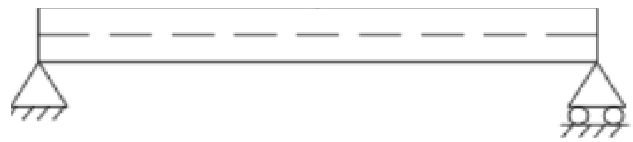


Figure 1. 2. Simply supported beam

ii. Slabs

A slab is a structural element made of concrete that is used to create horizontal flat surfaces such as floors and roof decks in the building. It is a planar element member which may span in one direction or two directions and may be supported by beams, columns, walls or the ground. The minimum panel dimension for the slab has to be not less than 5 times the overall slab thickness(Thonier, 2006). They exist several types of a slab which are, flat slab, beam supported slab, ribbed and hollow block floors.

A flat slab is a two-way concrete slab supported directly by columns without the use of beams with reinforcement in two orthogonal directions as illustrated in figure 1.3. It has the advantages of simple construction and formwork and a flat ceiling. Flat plates are typically economical for span lengths between 4.5 m to 7.5 m when subjected to moderate live loads(Fanella, A; 2011).

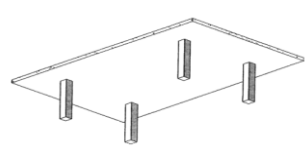


Figure 1. 3. Flat slab directly supported by columns (Fanella, A; 2011)

when the spans are relatively long and/or the live load is 4.78kN/m^2 or greater, special structural element on the top of the column such as drop panel; column head/column capital and/or the combination are provided at column perimeter. Figure 1.4 illustrate a flat slab supported by columns with drop panels.

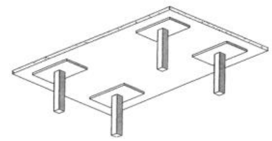


Figure 1. 4. Flat slab supported by columns with drop panels(Fanella, A; 2011)

A Beam supported slab was the original reinforced concrete slab system and can accommodate a wide range of span and loading conditions. It is supported with beams and columns, with the load transferred through beams then columns respectively, figure 1.5 illustrate the beam supported slab.

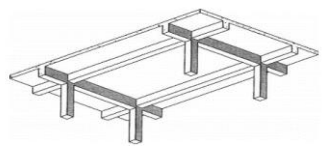


Figure 1. 5. Beam supported slab system(Fanella, A; 2011)

A ribbed hollow block floor has longitudinal voids/cores running through it. The hollow block floor is generally constructed with blocks made of clay tile or with concrete containing a lightweight aggregate (Mosley et al., 2012). Cross-sections through a ribbed and hollow block slab are shown in the figure in figure 1.6 and figure 1.7 respectively.

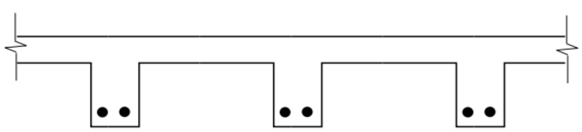


Figure 1. 7. Cross-sections through a ribbed slab (Mosley et al., 2012)

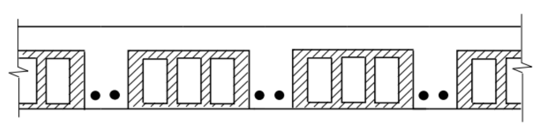


Figure 1. 6. Cross-sections through a hollow block slab (Mosley et al., 2012).

The principal advantage of these floors is the reduction in weight achieved by removing part of the concrete below the neutral axis and, in the case of the hollow block floor, replacing it with a lighter form of construction. (Mosley et al., 2012). When the ribs in the slab are in two directions, the slab is called a waffle slab.

b. Vertical elements

The vertical structural elements such as columns and shear walls carry vertical and horizontal to the building foundation.

i. Columns

The columns in a structure carry the loads from the beams and slabs down to the foundations, they are primarily compression members, although they may also have to resist bending forces. Columns used in buildings have a rectangular shape, square shape and circular shape. According to the geometry, a vertical structural element is classified as a column when the cross-sectional depth does not exceed 4 times its width and the height of the column is at least 3 times the section depth if not the vertical element is classified as a shear wall. Figure 1.8 illustrate a rectangular column.

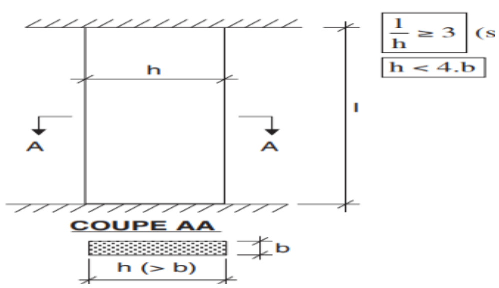


Figure 1. 8. Dimensions properties of a rectangular column (Roux, 2009)

ii. Shear walls

Shear walls are an important type of wall that act as rigid vertical diaphragms which transfer horizontal forces, such as wind and seismic effects to the building foundation in a direction parallel to their planes. Shear walls may also carry vertical loading from the supported structure together with transverse bending effects. Figure 1.9 illustrate a shear wall subjected to axial force.

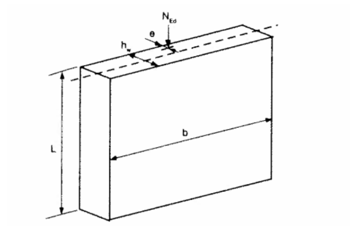


Figure 1. 9. Shear wall (Thonier, 2006)

1.2.2.2. Sub structural reinforced concrete elements

Sub structural elements are elements located below the ground and form the foundations of a building. The foundations can be classified into two categories according to the depth of the foundation which are shallow foundations and deep foundations. The type of foundation to be used depends on several factors such as the soil properties and conditions, the type of structure and loading and also the permissible amount of differential settlement.

a. Shallow foundations

The various type of shallow foundations is pad footing, combined footing, strip footing, strap footing and raft.

i. Pad footing

Pad footing is a structural element located under a single column, pad footing can have a square or rectangular base in the plan. According to the relative magnitudes of the axial load (N) and the moment (M) acting, there is a linear distribution of the bearing pressures across the base which can take one of the three forms illustrated in figure 1.10. The case (a) of figure 1.10 illustrates the constant linear distribution of pressure under a pad footing subjected to axial force only. Case (b) of figure 1.10 illustrates the linear distribution of pressure under a pad footing subjected to axial force and the moment where the soil is fully compressed. The case (c) of figure 1.10 illustrates the linear distribution of pressure under a pad footing subjected to axial force and moment where a part of the soil is compressed and the other part is subjected to tension.

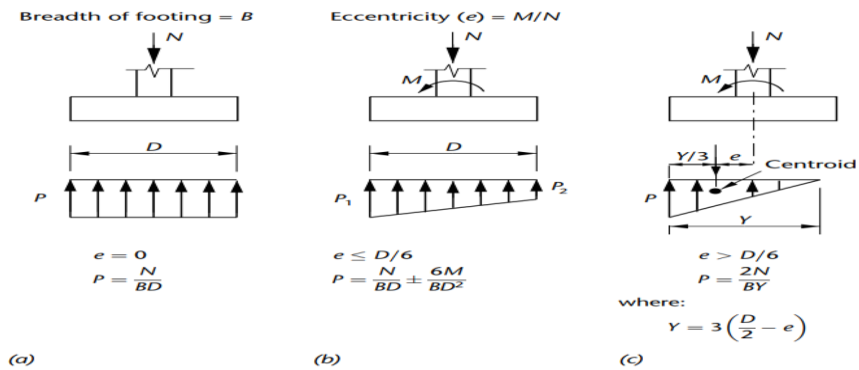


Figure 1. 10. The linear distribution of pressure under pad footing (Mosley et al., 2012)

ii. Combined footings

A combined footing is used when two columns are close together, the individual footing of each of the two columns are combined to form a continuous base. The dimensions of the footing should be chosen so that the resultant load passes through the centroid of the base area to give a uniform bearing pressure under the footing and help to prevent differential settlement.

The shape of the footing may be rectangular or trapezoidal as shown in figure 1.11. The trapezoidal is used where there is a large variation in the loads carried by the two columns and there are limitations on the length of the footing.

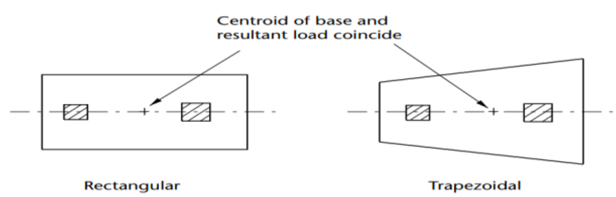


Figure 1. 11. Different combined footings shapes (Mosley et al., 2012)

iii. Strip footing

Strip footings are used under walls or a line of closely spaced columns. Even where it is possible to have individual bases. The footings are analysed and designed as an inverted continuous beam subjected to the ground bearing pressures. If the columns are equally spaced and equally loaded the pressure is uniformly distributed at the base but if the loading is not symmetrical then the base is subjected to an eccentric load and the bearing pressure varies as shown in figure 1.12.

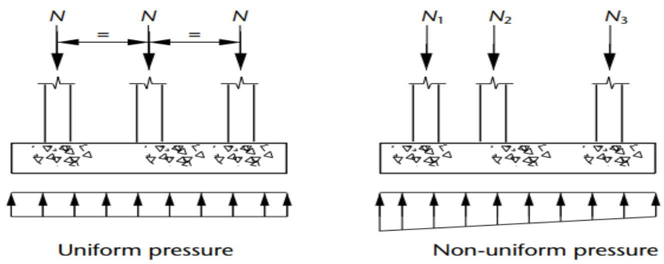


Figure 1. 12. The linear distribution of pressure under a Strip footing (Mosley et al., 2012)

iv. Strap footing

Strap footings, as shown in figure 1.13, are used where the base for an exterior column must not project beyond the property line. A strap beam is constructed between the exterior footing and the adjacent interior footing. The purpose of the strap is to restrain the overturning force due to the eccentric load on the exterior footing. The base areas of the footings are proportioned so that the bearing pressures are uniform and equal under both bases, thus the results of the loads on the two footings must pass through the centroid of the areas of the two bases.

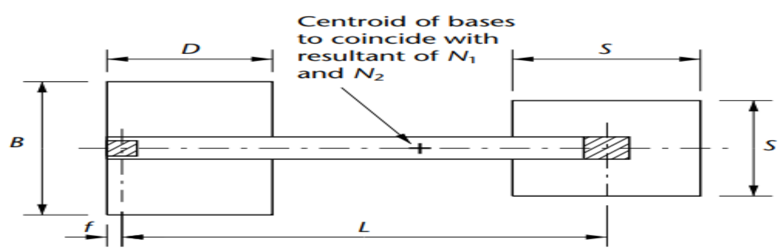


Figure 1. 13. Strap foundation(Mosley et al., 2012)

v. Raft foundation

A raft foundation transmits the loads to the ground through a reinforced concrete slab that is continuous over the base of the structure. The raft can span any area of weaker soil and it spreads the loads over a wide area. The exist three types of raft foundations which are flat slab, down standing beam and upstanding beams as illustrated in figure1.14.

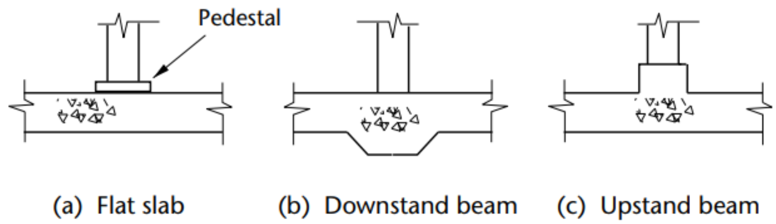


Figure 1. 14. Different types of raft foundation(Mosley et al., 2012)

The flat slab is the simplest type of raft foundation, it consists of uniform thickness supporting the columns. Where punching shears are large, the columns may be provided with a pedestal at the base. The pedestal serves a similar function to the drop panel on a flat slab floor. Other, more heavily loaded rafts require the foundation to be strengthened by beams to form a ribbed construction. The Down standing beams shown in figure 1.14 have the disadvantage of disturbing the ground below the slab and the excavated trenches are often a nuisance during construction, while upstanding beams shown in figure 1.14 interrupt the clear floor area above the slab.

b. Deep foundations elements

Piles are used where the soil conditions are poor, or when it is not possible to provide adequate shallow foundations. The piles must extend down to firm soil so that the load is carried by either end bearing, friction, or a combination of both end bearing and friction. Concrete piles may be precast and driven into the ground, or they may be the cast-in-situ type that is bored or excavated. Depending on the amount of load available, a single pile or group of a pile can be used, the figure 1.15 illustrate the single pile and pile group.

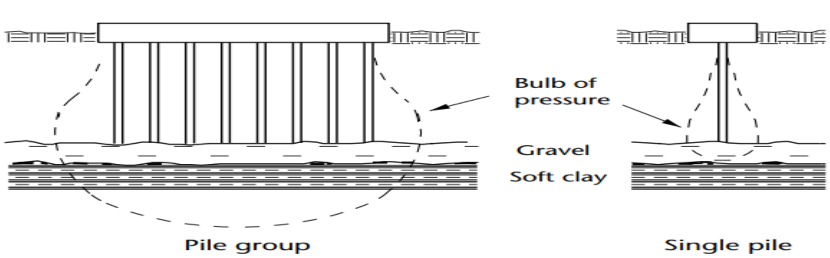


Figure 1. 15. Single pile and pile group (Mosley et al., 2012)

Pile caps are supported on piles and carry the load from columns and walls and transfer them to the piles. Figure 1.16 present the pile caps on four circular piles carrying load coming from a rectangular column.

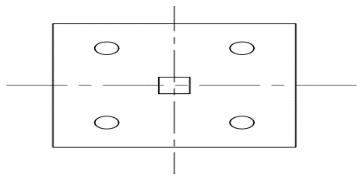


Figure 1. 16. 2D plan of a rectangular pile cap on piles (Ashraf, 2018)

1.2.3. Regulations used for the design of the reinforced concrete building

To perform the structural design of a reinforced concrete building, a set of European standards (EN) developed by Europeans are used.

1.2.3.1. Eurocode

Eurocode whose title Basics of structural design is the first series of European standards for construction. It is informally named Eurocode 0 and abbreviated EN 1990. The EN 1990 establishes principles and requirements for the safety, serviceability and durability of structures, describes the basis for their design and verification and gives guidelines for related aspects of structural reliability.

1.2.3.2. Eurocode 1

Eurocode 1 whose title Actions on structures is abbreviated EN 1991 or informally EC1. EN 1991 describes values for various types of loads and densities for all materials which are likely to be used in construction. Eurocode 1 is divided into seven different parts.

1.2.3.3. Eurocode 2

Eurocode 2 whose title Design of concrete structures is abbreviated EN 1992 or informally EC 2 specifies technical rules for the design of reinforced concrete and prestressed concrete structures. Eurocode 2 is intended to be used in conjunction with EN 1990; EN 1991; EN

1997 and EN 1998. The EC 2 is divided into several parts. Part 1-1 whose title General rules and rules for buildings is suitable for performing structural design and also produce construction details of structural reinforced concrete elements.

1.2.3.4. Eurocode 7

Eurocode 7 whose title Geotechnical design is abbreviated EN 1997 or informally EC7 describes how to design geotechnical structures, using the limit state design philosophy. Eurocode 7 is used in conjunction with EN 1990 and it is applied to the geotechnical aspects of the design of buildings and civil engineering works.

1.2.3.5. Eurocode 8

Eurocode 8 whose title Design of structures for earthquake resistance is abbreviated EN 1998 or informally EC 8 describes how to design structures in a seismic zone. The EC 8 is grouped into six parts. Part 1whose title General rules, seismic actions and rules for buildings are used for the design of buildings and civil engineering works in seismic regions. This part is subdivided into 10 sections and section 5 is specifically devoted to the design of concrete buildings

1.3. Building information modelling (BIM)

1.3.1. Definition

The ISO 19650 defines BIM as the use of a shared digital representation of a built asset to facilitate design, construction and operation processes to form a reliable basis for decisions.

The acronym BIM has three different significations, Building Information **M**odelling; Building Information **M**odel and Building Information **M**anagement.

Building Information **M**odelling is an activity that consists of creating a building information model using the technologies and the process of specialist firms creating the virtual model(Arayici, 2015). Any simulation of a real activity related to a building is an act of building information modelling.

Building Information **M**odel is a product, it consists of a numerical representation of the physical and functional aspects of a building(Arayici, 2015). Any compilation of building information in any form is a building information model.

Building Information **M**anagement is a system, it consists of a business structure of work and communication(Arayici, 2015). It ensures that the process is followed without interfering with the project development. It is in charge of the establishment of the BIM convention.

The ”I-Information” in BIM can be either graphical or non-graphical; either contained directly in the building model or accessible from it through linked data that is stored elsewhere (as cited in Abanda et al., 2014).

The major idea behind BIM is to complete a project virtually before the construction of the project has started physically to find potentials risks, problems and conflicts(Ahmed & Hoque, 2018). So BIM process implies the collaboration and the exchange of data between stakeholders involved in the project. Figure 1.17 present the different stakeholders involved in the BIM process.

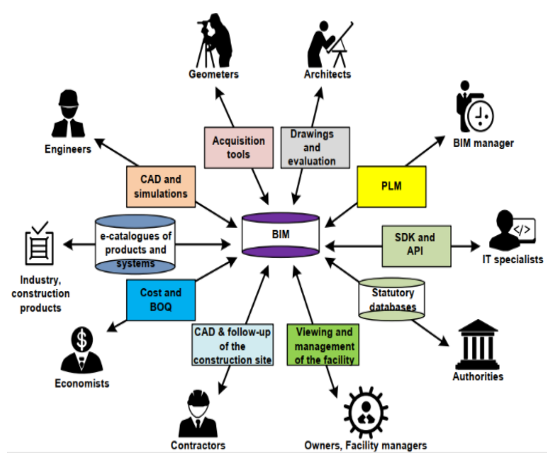


Figure 1. 17. Interaction of different professionals involved in the BIM process(Forgues et al., 2016)

1.3.2. BIM origin

The research around optimizing the building process is not new, starting back in the early '60s with a participant from the United States, Western Europe and the Soviet Blocs country. Research papers about the creation and employment of virtual building models were first published in the 1970s. In 1974**,** Eastman proposed a first model description of the building, this first model was known as Building Description System (BDS). The Building Description System was a structured database that described building components and their relationship to each other(Kensek & Noble, 2014). The official term BIM (Building Information Modelling) was used for the first time in 1992 by the researcher's Van Nederveen and Tolman. However, slightly over 20 years, a large range of software products with powerful BIM functionalities have been published and the concept which originated in academic research has now become established in industry practice.

1.3.3. Dimensions of BIM

The different dimensions of BIM refer to a particular way in which certain types of data are linked to the BIM model. By adding more dimensions of data, the result is a better and deeper understanding of the project, how it will be delivered, what it will cost, and how it should be maintained. The dimensions of BIM start from 3D and can generalize up to nDimensions (nD)(Koutamanis, 2020).

3D BIM this dimension will be used to share information through the BIM process. The geometry and position of any symbol (building element of space) can be fully and unambiguously described based on three geometric dimensions(Koutamanis, 2020).

4D BIM is obtained by adding data (time) to the 3D BIM models. The main current application of 4D BIM is construction scheduling(Koutamanis, 2020).

5D BIM is obtained by adding cost/price to the 4D BIM models. They allow automatic cost estimation at each intermediate step of the construction(Mbarga & Mpele, 2019).

6D BIM (Sustainability) is obtained by adding a life cycle analysis to the 5D BIM models. They allow to analyse the overall cost of a structure or infrastructure over its life cycle and evaluate related environmental impacts and energy consumptions(Mbarga & Mpele, 2019).

7D BIM is involved by adding operations management and maintenance to the 6D BIM model. They allow updating BIM models and facilitating the operation and maintenance of structures or infrastructures (Mbarga & Mpele, 2019).

1.3.4. Levels of BIM

The levels of BIM consist of BIM maturity levels and levels of development where the BIM maturity levels reflect the degree of collaboration in the BIM process and the levels of development describe which graphical and non-graphical information an object should contain.

1.3.4.1. BIM maturity levels

BIM require professionals to collaborate and share projects information. The BIM maturity level reflects the degree of collaboration in the BIM process as well as the levels of sophistication of use of the individual software, BIM Maturity model consists of four levels starting from Level 0 toLevel3(Sacks et al., 2018).

a. Level 0

This level is the era of Computed Aided Design (CAD) which only requires working in 2D with no 3D(Arayici, 2015). The information is shared by traditional paper drawings or in some instances, digitally via PDF, DWG, DGN or DXF file format, essentially separate sources of information covering basic asset information.

b. Level 1

This level integrates working with 2D and 3D data but this is only for visualisation purposes (Arayici, 2015). These data are managed according to BS1192 with file-based collaboration through a common data environment. Models are not shared between project team members.

c. Level 2

This level is about individual discipline-based BIM models used for collaborative working. This level consists of a managed 3D environment held in separate discipline BIM tools with data attached. All parties use their 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. Design information is shared through a common file format, which enables any organization to combine that data with their own to make a federated BIM model, and to carry out interrogative checks on it. This level of BIM can utilise 4D construction planning simulation and/or 5D cost estimations.

d. Level 3

This level represents a full collaboration between all disciplines 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”.This level of BIM will utilise 4D construction sequencing, 5D cost estimations and 6D project lifecycle management information.

The BIM maturity level is illustrated in figure 1.18. For a given construction project, ”maturity level” evaluates BIM level implementation according to used software and means of information exchanges. Different countries have set targets to attain different levels at different times. For example, the UK Government's BIM Strategy Paper calls for the industry to achieve BIM level 2 by 2016.

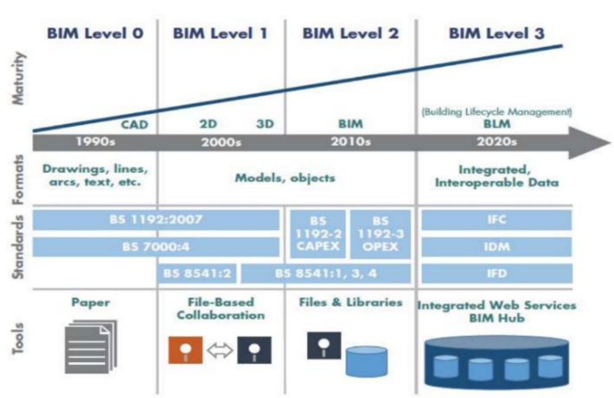
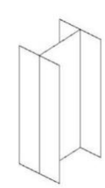


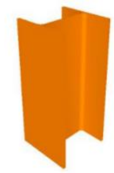
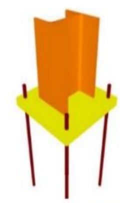
Figure 1. 18. BIM Maturity Level(Sacks et al., 2018a)

1.3.4.2. Levels of development (LOD)

A BIM model contains objects at varying degrees of ”completeness” depending on the intended use of the model(Kensek & Noble, 2014). The LOD defines for each stage of the project how deep the BIM model will be developed regarding the level of detail for the geometry, maturity, and level of information for the quantity of data. It allows designers, engineers, contractors, and any stakeholders to focus their work on what's needed. The US-American BIMForum published the six standardized LODs (100, 200, 300, 350, 400 and 500) and also an extensive catalogue depicting the geometric part of the LODs for typical building components. The six-level of development can be related to project phases. Table 1.6 Present the different levels of development with an example.

Table 1. 6. Description of the different levels of Development with an example for each

LOD	Description	Example
100	✓ It is associated with the preliminary design phase ✓ this level presents a schematic or conceptual design model ✓ The model element may be graphically represented in the Model with a symbol or other generic representation, the overall building model is in an abstract form	 19 B1010.10-LOD-100 Floor Structural Frame (Steel Framing Columns)
200	✓ It is associated with the schematic design phase ✓ The model element is graphically represented within the model as a generic system or assembly with approximate quantities, size, shape, location, and orientation ✓ Non-graphic information may also be attached to the Model Element	 24 B1010.10-LOD-200 Floor Structural Frame (Steel Framing Columns)

300	✓ It is associated with the design development phase ✓ The model element is represented graphically as a specific system or assembly. This can be represented in terms of quantity, size, shape, location and orientation. ✓ Non-graphic information may also be attached to the model element	 25 B1010.10-LOD-300 Floor Structural Frame (Steel Framing Columns)
350	✓ It is associated with construction documents ✓ The model element is represented graphically as a specific system or assembly. This can be represented in terms of quantity, size, shape, location and orientation. ✓ Non-graphic information may also be attached to the model element ✓ LOD 300, LOD 350 is necessary for coordination of the element with nearby or attached elements are modelled	 26 B1010.10-LOD-350 Floor Structural Frame (Steel Framing Columns)
400	✓ It is associated with the construction phase ✓ The model element is graphically represented within the model as a generic system or assembly with approximate quantities, size, shape, location, and orientation ✓ Non-graphic information may also be attached to the Model Element. ✓ The model element is modelled at sufficient detail and accuracy for fabrication of the represented component.	 27 B1010.10-LOD-400 Floor Structural Frame (Steel Framing Columns)
500	✓ The model element is a field verified representation in terms of size, shape, location, quantity, and orientation. ✓ Non-graphic information may also be attached to the Model Elements.	

1.3.5. BIM contributions in the construction Industry

The use of BIM in the construction industry contributes to the increase of communication between stakeholders across each project phase, the reduction of manual re-entering of data and identification and resolve possible conflicts at early phases of a project and significantly decreases the effort required at later phases.

1.3.5.1. Information flow in traditional and BIM process through the building life cycle

The construction project enables the participation of a wide range of stakeholders from different fields of expertise and needs a continuous reconciliation and intense exchange of information among stakeholders for the project to succeed within the delay and budget.

The traditional workflow used in the AEC industry to exchange information between stakeholders is predominately in the form of drawings, either as physical printed plots on paper or in a digital but limited format. The limited information contains in the technical drawings cannot be directly used by other stakeholders between project phases by using specialized software applications but must be re-entered manually which requires unnecessary additional work. The result is a loss of information across phases, many opportunities off errors and omissions, and increased effort to produce accurate project information, as the conceptual diagram in figure 19 illustrate that at each of these information exchange points, data that was once available in digital form is lost and has to be laboriously re-created (Bormann et al., 2018).

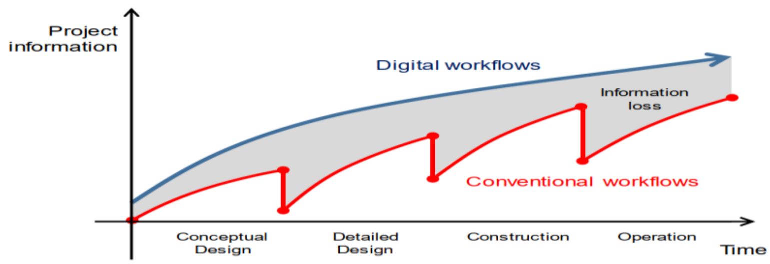


Figure 1. 19. Evolution of the project information within the conventional workflows and the digital workflows (Borrmann et al., 2018)

A digital workflow using BIM processes supported by BIM tools in the AEC industry improve the integrated design process, which increases the project information in each phase and allows greater efficiency for the project team. In contrast to recording information in drawings as in conventional workflow, BIM stores maintain and exchanges information using a BIM model(Borrmann et al., 2018). This approach much more improves the coordination of design activities by reducing manual re-entering of data to a minimum, minimizes opportunities for errors and allowing the re-use of digital information, which increases productivity and quality in construction projects(Borrmann et al., 2018).

1.3.5.2. Design effort and time for the traditional workflow and BIM workflow

BIM is a revolutionary process of working in the AEC industry that transforms the process of designing projects by changing the standard timetable for task completion used in CAD workflow without fundamentally change the deliverables of the project (Drawings, schedules, etc...)(Kim et al., 2016). Patrick MacLeamy developed the concept of shifting ”the effort ”also known as the MacLeamy curve. (Kim et al., 2016). This curve display in figure 1.20 illustrates the general relationship between design effort and time, indicating how effort is traditionally distributed (line 3) and how it can be redistributed as a result of BIM (line 4)(Sacks et al., 2018a). The Mac Leamy curve diagram highlights the importance of shifting labour earlier in the design phase process and also the importance to make higher decisions earlier before changes become too difficult or costly to implement(Kim et al., 2016).

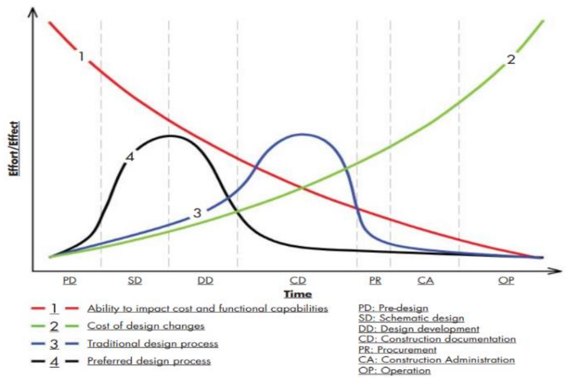


Figure 1. 20. Curves of efforts when designing a project with or Without BIM in the AEC industry(Sacks et al., 2018a)

In conventional planning processes, the main design and engineering effort occurs in the later detailed design phases, sometimes even during the actual construction phase. As a result, the detailed coordination of design disciplines, the integration of analysis and simulation tools and consequently a comprehensive assessment of the building design only occurs at a relatively late point in the overall process. At this point, the possibilities for design changes are more limited and also more costly to implement.

Applying BIM in the planning process results in shifting the design effort to earlier design phases by building up a comprehensive digital building model. Implementing BIM in a project allows teams to make and share information earlier so allows the possibility to evaluate the impact of design decisions more and also to identify and resolve possible conflicts early and significantly decreasing the effort required at later phases and improving the overall design quality.

1.3.6. BIM initiatives around the world for its adoption and implementation

The implementation of BIM in the AEC industry in the country around the world is driven by various initiatives and approaches such as government and industry leadership, development of national and global BIM standards, legal protocol, education and training on BIM(peter Smith, 2014).

1.3.6.1. BIM initiatives in Asia, Europe and Nord America

In Asia, some country was an earlier adopter of BIM practice. In China, the Hong Kong Housing Authority has set the goal to mandate the use of BIM on all its projects by 2014. The Hong Kong government requires the use of BIM for all government projects over 30millions dollars from the 1st January 2018(Sacks et al., 2018a). In Singapore, the Building and Construction Authority (BCA) announced that BIM would be Mandate for architectural submissions of all new building projects larger than 20 000 m^2 by 2013, BIM would be Mandate for engineering submissions for all projects by 2014 and from 2015 all projects with a gross floor area of more than 5000 square meters would mandate architecture and engineering BIM e-submission(Sacks et al., 2018a). In other Asia countries such as Malaysia, South Korea, Japan, India, Iran, and the United Arabs Emirates, BIM is growing with actions taken by governmental or non-governmental organizations and companies in the AEC industry.

In Norway via Statsbygg, in Denmark through bips/MOLIO and in Finland via Senate Properties the adoption and implementation of BIM in the public project are since 2007, also the promotion and adoption of the interoperability is driven by the use of the open standard format IFC as a file format to exchange data on public projects in the AEC industry (Sacks et al., 2018a).

In 2010, United Kingdom Government announced BIM requirements, and from 2016 onwards, the government of the UK mandated the use of BIM in public sector projects(Matarneh & Hamed, 2017). France announced in 2014 via CSTB (Centre Scientifique et Technique du batiment) that it would develop 500 000 houses using BIM by 2017. In December 2015, the German Minister of Transport announced a timetable for the introduction of mandatory BIM for German infrastructure projects by 2020 and potentially also on building construction projects (Sacks et al., 2018a).In Italy, the Italian government announced that public projects over 5 225 000 euros must meet the level 2 BIM of the UK BIM roadmap from October 2016 (Sacks et al., 2018a).

In North America, the United States of America (USA) have been amongst the earlier leader in BIM development and implementation in the AEC industry around the world. In 2003, the General Services Administration (GSA) which is responsible for the construction and of all federal facilities established a national 3D-4D BIM program to support the implementation of this technology for the realization of public projects(peter Smith, 2014). In 2008 GSA has mandated BIM on all its major projects (approximately over USD 35 M). Since 2014, the BIM institute in Canada has conducted several initiatives to enhance a wide BIM adoption. Figure 1.21 illustrate the starting year of BIM initiatives for it adoption in many countries around the world.

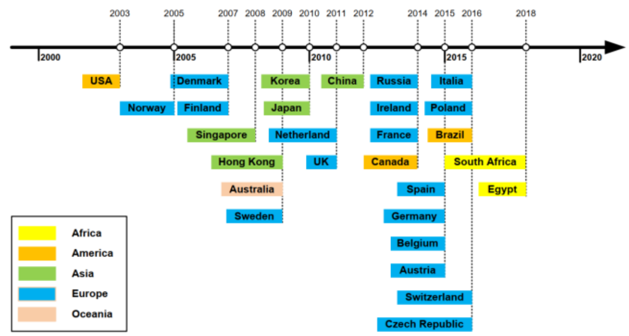


Figure 1. 21. Starting years of major initiatives for BIM adoption around the world (Mbarga & Mpele, 2019)

1.3.6.2. BIM initiatives in Africa

The implementation of BIM in the AEC industry is recent in a developed country and are going at a slow rate in comparison to the digitalization of the manufacturing industry. The situation is worst in developing countries, which are in the majority of cases find themselves on the disadvantaged side of the digital transition(BIM.Africa, 2020). In Africa, BIM implementation varies from one country to another(Matarneh & Hamed, 2017). South Africa, Nigeria and Egypt are the leaders in the implementation of BIM in Africa (Matarneh & Hamed, 2017).

In 2018, a not-for-profit and collaborative membership-based professional organisation called BIM. Africa was founded to enable and regulate the adoption of BIM in the AEC industry across Africa. In 2019, the organization launched “The Student Advocacy Program 2019”. It was a program designed to create BIM awareness amongst students of tertiary institutions in Africa.70 student’s teams for 70 tertiary institutions across 29 countries. The selected Student Advocates were trained virtually on BIM and how to present a BIM awareness program in their various institutions(BIM.Africa, 2019).

In 2020 as in 2019, BIM. Africa launched "The Student Advocacy Program 2020". And the also made the ''African BIM Report (ABR) 2020'' which is the first pan- Africa report on BIM, it was conceived by the Research and Development Committee of BIM Africa. This report aims to be a starting point to provide annual publication on the state of implementation of BIM across the Africa continent(BIM.Africa, 2020). The ABR 2020 consist of an overview of the continental-wide survey of the level of awareness and adoption of BIM across Africa; a selection of Project across Africa that had experimented the implementation of digital technologies, challenges and lessons learned; Opinions of experts on the digital construction(BIM.Africa, 2020).

Always in 2020, the non-governmental organizations' BIM. Africa and BIM CommUNITY Africa launched a free virtual conference called ''BIM Harambee Africa'' from the 5 ^{to} – 30 th of October 2020, this virtual training consisted of 42 presentation sessions, 6 Learning sessions and 5-panel discussions aims to accelerate BIM knowledge across the Africa Continent.

In South Africa, the vulgarization of BIM is based on the establishment of BIM training certification, education and research. The organization BIM Institute founded in May 2015 has the mission to support and help deliver the standard and the requirements for BIM strategy for Africa. In Nigeria, the vulgarization around the country and all over Africa is done by the non-governmental organization, BIM Africa. In Tanzania, the Firm Silicon Valley provides BIM outsourcing services for successful BIM projects to be completed. Concerning the education and training on BIM, Ardhi University has recently introduced the BIM course in the program of Architecture (BIM.Africa, 2020). During the year 2010-2011; the Ethiopian Construction Project Management Institute (ECPMI) which is a federal government organization took initiatives for further adoption and implementation of BIM in 2018, the Ethiopian Standard Agency had released three BIM standards for the implementation of BIM in the country (BIM.Africa, 2020).

1.3.6.3. BIM initiatives in Cameroon

Concerning the state of BIM in Cameroon, its adoption and implementation, research was performed on the use of BIM. Abanda et al, (2014) performed research on the implementation of BIM using a pilot study, electronics email surveys and in-depth phone interviews. A sample of 179

Civil Engineers from the National Advanced School of Engineering Yaoundé-I (NASE/UYI) was involved in the study and a response rate of 25.7% was recorded. The result of the investigation shows that the most popular used software in the construction sector is Autocad, Robot Structural Analysis and MS Project, the result also highlighted Autocad Civil 3D and Midas Gen as emerged BIM tools in the construction industry. It is important to note that any of this software can be considered as a BIM authoring tool, but they represent software which are related and usable in a BIM-enabled process. Another result of the investigation was that the cost of BIM software and the low level of governmental initiatives in the use of BIM was identified as the major difficulties for the adoption of BIM in Cameroon (Abanda et al., 2014).

To vulgarize the use of BIM in Cameroon, the department of Civil Engineering of the National Advanced School of Engineering Yaoundé- I planned to introduce BIM-related modules in the program course of Civil Engineers during the academy year 2019/2020 (Mbarga & Mpele, 2019). In addition Mbarga & Mpele, (2019) argue that “BIM should be supported by a larger number of institutions devoted to training and research in Civil Engineering in Cameroon”. Always in 2019 at NASPW, Abanda through” Programme Secam 2019” create awareness on BIM to the participant during the session dedicated to” modelisation numerique du bati”.

1.3.7. Interoperability in the BIM process

Interoperability is the ability for all the stakeholders (architects, engineers, builder/operator, etc.). In the construction sector to generate, share and transmit, without dissipation at any time, all the data necessary for the project through all the design process and the life cycle of the building starting from the design to demolition of the structure. To achieve the target of collaboration, communication within the project, the software tools used in the BIM workflow must be interoperable.

1.3.7.1. Cost of lack of interoperability

In a traditional workflow, the collaboration across the discipline is based on the exchange of 2D drawings and documents which lead most of the time to the re-entering of data and is time-consuming. BIM as a collaborative process allows the stakeholders to share and exchange information around the digital model (BIM model) throughout the life cycle of the building. However, the success of this process depends on the interoperability of the software used. The interoperability issues in the AEC sector are multiple, economic, time to work, productivity. Two studies conducted successively by NIST(National Institute of Standards and Technology) in 2004 at the United States of America and the FFB(Federation Française du Bâtiment) in 2009 at France on the cost of the lack of interoperability established that the complications encountered in the AEC sector (design errors, gaps in estimates, conflicts between design and execution) were linked to the multiplicity of databases and software platforms that limit the flow of information throughout the design process and the life cycle of the building. The NIST report reveals that interoperability failures in the construction industry cost $16 billion in the United States or 1-2% of the industry's turnover. In 2009, based on the criteria and method used in the American study, a similar study was carried out by the FFB and the report reveals that the costs of lack of interoperability for the French construction sector are a minimum of 35 €/m^2 in construction for companies and 2.3 €/m^2 /year in operation for building owners and asset managers. These two reports highlight the importance of the problem and the potential benefits of solving the problem.

1.3.7.2. Formats of exchange data in the BIM process

In the construction industry, the needs for interoperability in the BIM process enable the creation of file formats of exchange data between stakeholders using software applications.

a. Importance of open standard format for exchange data

In the construction industry, the communication between software used by stakeholders is a huge challenge. If each software has to develop a single interface and language with all the other software with which it is supposed to interact, interoperability is becoming complex and probably unsuitable for the construction sector (Julien, 2015). The left image of figure1.22 illustrates the difficulty of making each software directly interoperable with all the others, therefore, it is necessary to define an open exchange standard format that allows domain-specific software to communicate efficiently. The right image of figure 1.22 shows a framework of interoperability based on a standard and neutral exchange language that can be used by BIM software.

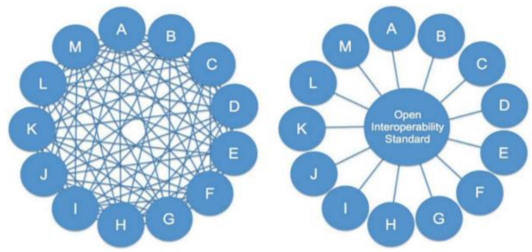


Figure 1. 22. Two conceptual scenarios of interoperability in BIM workflow (Marzia Bolpagni, 2013)

b. IFC format

IFC (Industry Foundation Classes) is an open standard to represent, exchange and share the BIM information among different software applications (building SMART, 2010b). There are approximately 150 different software that supports IFC. IFC is developed by building SMART former known as IAI (International Alliance for Interoperability) created in 1994 and became Building SMART International in 2005. The first version of IFC standards was released in 1999 and are constantly developed and as with other products new features are added to it in each release. Today the current revision of the IFC standard is the IFC4 released in 2013, however, the previous release, IFC2x3 standards since 2005 are already certified and widely used between software in the AEC industry. Figure 1.23 presents the different IFC standard revisions released.

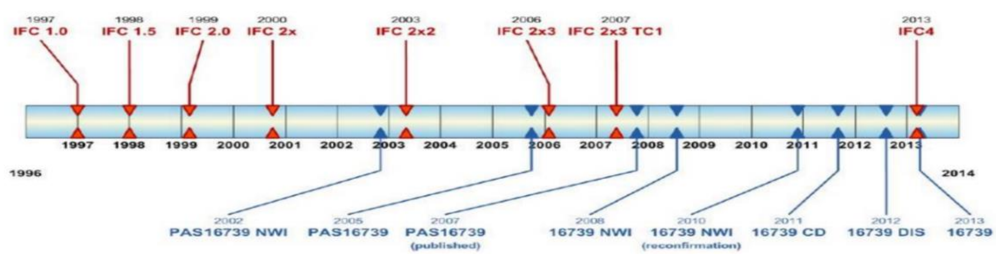


Figure 1. 23. Schematic representation of IFC development (Marzia Bolpagni, 2013)

The Information requirement within the IFC standard is defined via the Information Delivery Manuals (IDMs) and the Model view definitions (MVDs). The objective of the IDM is to provide specifications on information exchanged in each operation of data exchanged during the building process. The MVDs are subsets or views of the IFC standard, each view of the MVDs is used to ensure that the correct information needed by a stakeholder of the project is obtained among all the information within the IFC format.

The structure of the data model within the IFC format is divided into four main layers which are domain, interoperability, core, and resource layers as illustrated in figure 1.24. The domain layer contains domain models for processes in specific disciplines of the AEC or software application, such as architecture, structural engineering and HVAC. (Laakso & Kiviniemi, 2012). In the structural engineering discipline, two domains are distinguished. The Ifc structural elements domain which represents some components of the structure such as rebars and tendons and also the IFC structural analysis domain which contains all the abstract data needed to analyse and design structures such as assumptions and load combinations. Each of these domains contains its own IFC classes, for example in the HVAC domain engineering it exists IfcPipeFitting type class for pipe fittings (Abdelqaddous, C; 2020).

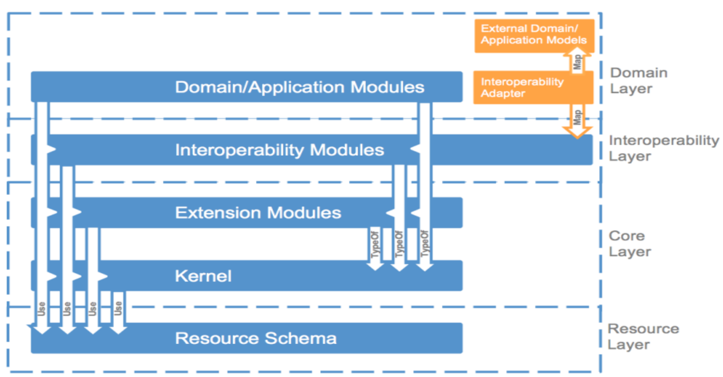


Figure 1. 24. Structure of IFC data model(Laakso & Kiviniemi, 2012)

The interoperability layer provides the interface for domain models, which provides an exchange system for interoperability between domains(Laakso & Kiviniemi, 2012). For example, columns and beams are shared between the architectural and structural domains. The core layer consists of the kernel and extension modules. The kernel determines the model structure and decomposition, providing basic concepts regarding objects, relationships, type definitions, attributes and roles. Core extensions are specializations of classes defined in the Kernel(Laakso &

Kiviniemi, 2012). The resource layer holds the resource schema that contains basic definitions intended for describing objects in the above layers.

c. CIM Steel Integration Standards (CIS/2)

The IFC format is not the only file standard format capable of promoting interoperability in the construction industry. The first release version of the IFC format was not able to integrate all the disciplines of the AEC industry. It was much more focused on the Architectural domain than other domains, it was not able to write, read or share structural engineering data between stakeholders. The file format CIS/2 has been developed for the planning, design, analysis and construction of steel-framed buildings and similar structures.

The CIS/2 format was born as a result of the project “The Eureka CIMSteel (Computer integrated manufacturing for constructional steelwork)” launched in 1987 by the British association constructional steelwork, to create a standardized model dedicated to the steel structure. This project allowed the collaboration of 70 organizations from 9 European countries. In 1995 as a result of the collaboration, the first version of the CIS/2 format was available. The project was officially completed in 1998, the development of this format continued and led to the second publication of the CIS/2 language in 2000(Julien, 2015).

1.3.8. BIM in the structural design phase

1.3.8.1. Structural Building information modelling framework

The enormous mass of information coming from architectural, structural, and MEP disciplines are modelled by Stakeholders of their discipline leading to various linked databases. Amongst all those linked databases, one of which is the structural BIM model as shown in figure 1.25. Figure 1.25 also highlighted the importance of the structural engineer in the BIM process through the collaborative relationship he has with the other actors of the construction industry.

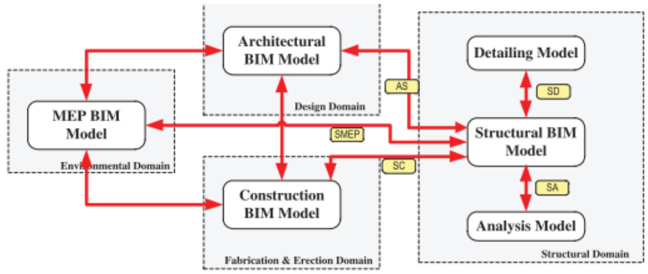


Figure 1. 25. Interaction between various BIM models in the BIM process(Issa & Olbina, 2015)

The information included in the BIM model is produced continually by the different participants in the building process. Thus the BIM model is relevant throughout the entire process starting with the program phases and ending with demolition after the operation phase(Nielsen & Madsen, 2010). However, the structural BIM model is a subset of the BIM model and have useful information and data related to structural engineering (Mohammed, 2018).

The acronym S-BIM is used to designate and characterize digital BIM tools specific to the field of structural engineering (Julien, 2015). In an S-BIM software, the engineer can assign relevant information to the model leading to create a structural BIM model. S-BIM tools are capable to handle several parameter groups such as section properties, geometry, materials properties, loads, boundary conditions, design data and results(Nielsen & Madsen, 2010)

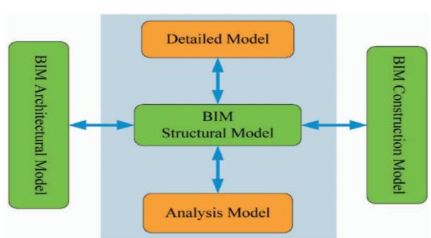


Figure 1. 26. The framework of collaboration amongst BIM model involved in the structural design Phase(Liu et al., 2016)

The structural BIM model as we can see in figure 1.26 is composed of an analysis model and detailed model. The detailed model is constituted by the structural geometrical shapes, sections properties, and materials extracted from the architectural model. The analysis model often contains much geometrical information such as axis position, member size, and space layout and division which also are extracted from the architectural model. Other pieces of information are created by structural engineers such as mechanical property, connection type, boundary conditions, loadings, etc. (Liu et al., 2016).

1.3.8.2. Traditional workflow versus BIM workflow in the structural design

In the traditional structural design workflow, structural analysis and construction drawing are usually two independent stages (Liu et al., 2016). This fragmentation may lead to design disputes in form of deficiencies in details, inadequate coordination, and deviations in submittals, excessive modifications, and failure of the design to meet budgetary or schedule requirements. (Issa & Olbina, 2015). The structural analysis is used for assisting construction drawing and confirming the final size and reinforcement of structural members(Liu et al., 2016). Figure 1.27 present the graphical representation of the traditional workflow in the structural design.

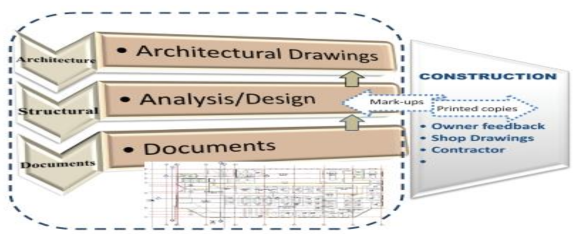


Figure 1. 27. Graphical representation of the traditional workflow in structural design(Issa & Olbina, 2015)

The Structural design BIM workflow integrates both the structural analysis and construction document stages, so as shown in figure 1.28 a real-time data can be shared through BIM structural model, which further reduces errors, losses and code discordance in the construction drawing stage (Liu et al., 2016).

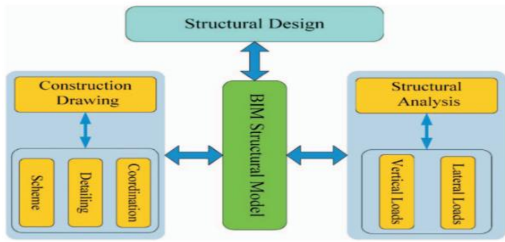


Figure 1. 28. Graphical representation of BIM-based structural design workflow(Liu et al., 2016)

The productivity benefits of the BIM design workflow highlighted by Issa & Olbina, (2015) are:

- BIM facilitate the sharing of structural information among related disciplines and reduces number of errors and omissions that flow from insufficient coordination.
- The improved coordination will allow engineers to detect clashes earlier, establish and adjust schedules more accurately and precisely and save precious time from work coordination and also improve design coherency and productivity.

- The use of BIM enables the sharing of structural details with fabricators and contractors.
- Transferring structure BIM model to the analysis software and the analysis results are delivered back into the model so keeping analysis, design, and documentation all synchronized.

1.3.8.3. Data sharing process for a structural design using BIM through software applications

BIM process as a continuous coordinated workflow where the needed information is available, architectural data model created by an architect can form a basis for the creation of the S-BIM model using S-BIM software application by structural engineers who also add relevant data such as boundary conditions, materials properties, loads and loads combinations. The S-BIM model now available is ready to be transferred into a FEM software where some information can be added or not before performing structural analysis and design via a specified structural design code(Nielsen & Madsen, 2010). Depending on the level of maturity of the interoperability between S-BIM Software and FEM software, some necessary information can be transferred backwards from the FEM software to the S-BIM software. Figure 1.29 present the graphical representation of the continuous workflow from the BIM model to the results in the structural design phase.

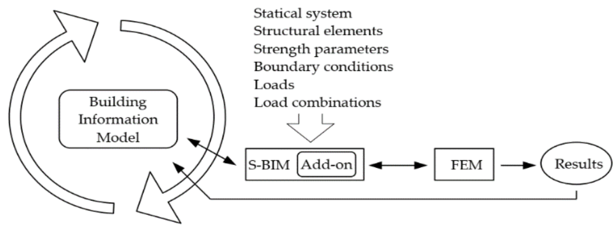


Figure 1. 29. Graphical representation of continuous workflow from the BIM model to the result

Liu et al., (2016) highlighted the use of FEM analysis software such as Etabs, Midas Gen, Abaqus, Ansys and Sap 2000 in the AEC construction industry. Autodesk Revit structure and Telka structures are S-BIM software for structural engineers(Nielsen & Madsen, 2010).

The Different possibilities of performing structural analysis and design using the BIM process until the results of the analysis are presenting in figure 1.30.

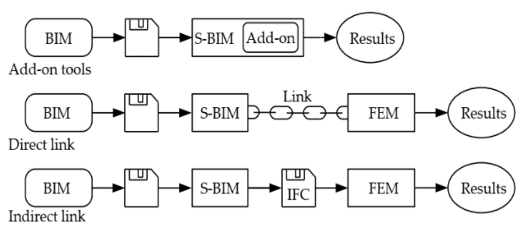


Figure 1. 30. Graphical representation of various possibilities of information flow starting from a BIM model to the results of the structural design calculations (Nielsen & Madsen, 2010)

The process with structural add-on tools within the S- BIM software which is show on the top in figure 1.30, this process gives the most direct information flow from the S-BIM model to the results. The add-ons tools are extended features in S-BIM software that enables analysis of structure inside the S-BIM software. During few years Autodesk Revit Structure with its add-ons

“Revit Extensions for Revit” was able to handle structural analysis directly in the S-BIM Sofware Revit Structure via cloud analysis. Nowadays it is not the case, however, the software companies Graitec developed an add-on called “GRAITEC Powerpack for Autodesk Revit “ which can perform structural analysis within Revit.

The process with a direct link is shown in the centre of figure 1.30 and we can distinguish direct links using an API or direct links using a native file (Mohammed, 2018). A direct link using a native file is a data transferring tool between two software applications of the same software companies. The direct link is an easy method of data exchange extremely efficient which demonstrate a high ability for interoperability, an example is a bidirectional workflow between Autodesk Revit and Autodesk Robot Structural Analysis (Mohammed, 2018). The direct link using API allows the interface tool (API) developed by producer software companies to enable interaction with other software application companies. For example, the tool “Midas link for Revit Structure“ enables to directly transfer a BIM model from Revit to Midas Gen and deliver it back to Revit through a file format called Midas Gen text file “MGT“. MGT file is capable to handle multiple data resources for example linear elements such as beams, columns etc..; planar elements such as the structural floor, structural walls etc..; boundary conditions; loads; materials and levels to transfer data between Revit to Midas Gen(Midas Gen Technical Paper, 2016).

The previous study demonstrated that in most of the cases both direct link with native and using API ensure bidirectional data compatibility and allow engineers to make changes in S-BIM software according to the results of their design in FEM software.

The process with the indirect link shown at the bottom of the part of figure 1.30 is used to transfer needed information from the S-BIM software to FEM software. The difference between indirect link is realized using an independent and natural format such as IFC, CIS/2 whereas direct link can be realized using a proprietary file format. The previous study on data exchange between S- BIM software such as Autodesk Revit, Telka structure and FEM software using indirect link base on IFC format highlighted the losses of information needed in the FEM software during the process of transferring data from S-BIM software to the FEM software.

Conclusion

At the end of this chapter, the main objectives were to talk about reinforced concrete, reinforced concrete buildings and Building Information Modelling (BIM). First of all, it was noticed that reinforced concrete is a composite material widely used in the construction sector all over the world, whose properties are well known. The structural design of a reinforced concrete building requires the use of Eurocodes regulation to ensure the structural safety, durability and safety of a building. Secondly, it can be observed that BIM is a great working opportunity for construction professionals and in particular for civil engineers because it allows performing structural design of structures, providing better visualization of technical drawings, enabling accurate estimation of cost and schedule. To perform the structural design of a multi-storey reinforced concrete building using the BIM process, the next chapter will focus on an adequate methodology to achieve the purpose.

CHAPTER 2 METHODOLOGY

Introduction

In this topic, it will be a question of describing the methodology used to perform the structural design of the reinforced concrete structure using the BIM process. The methodology in the first step consist of a site recognition followed by a site visit then data collection.The preliminary design consisting on actions on the building, materials and the predimensioning of structural elements. After the procedure to perform structural analysis of the structure using the BIM process is given, and followed by the structural design procedure of structural elements. In the end, the process of producing construction documents such as steel reinforcement plans, quantity take-off of materials is provided.

2.1. Site recognition

The general recognition of the site requires documentary research with the aim of defining the physical characteristics of the site such as the geographic location, climate, relief, geology and hydrology as well as the population and economic characteristics.

2.2. Site visit

After the general recognition of the site, a site visit will be made to get a better idea of the state of the place. The purpose of this activity is the description of the site.

2.3. Data collection

This activity is intended to collect data necessary to perform the structural design of the building.

2.3.1.1. Architectural data

The architectural data provide dimensions in plane, the height of the building, plane surface and architectural drawings such as floor plans, sections and elevations.

2.3.1.2. Geotechnical data

The geotechnical data consist of the admissible bearing capacity of the soil at a particular depth of the soil. This data will be used to design the foundations.

2.3.1.3. Structural data

Structural data are related to structural plans where we can identify the position of structural elements such as beams, columns and the concrete core.

2.4.Preliminary design

The preliminary design consists of evaluating the actions on the building, the mechanical characteristic of materials and the predimensioning of structural elements.

2.4.1. Actions

The EN 1990 classify actions as permanent actions, variable actions and accidental actions. The different actions used for the design of the project are obtained from EN 1991-1.1 densities, self-weight and imposed loads EN 1991-1.4 wind actions.

2.4.1.1.Permanent actions

Permanent actions consist of the self-weight of structural and non-structural elements. The permanent loads will depend on the volume weights of structural and non-structural elements.

2.4.1.2. Variable actions

For this project, the variable actions consist of imposed loads and wind loads.

a. Imposed loads

Imposed loads on a building are those resulting from occupancy. The imposed loads in the building are subdivided into categories of loaded areas and are presented in table 1 of Annex A. the partial factors for imposed load are presented in table 2 of Annex A

b. Wind loads

The wind action is represented by a simplified set of pressures or forces whose effects are equivalent to the extreme effects of the turbulent wind. They are determined from the basic values of wind velocity or velocity pressure. The evaluation of the wind actions on a building is performed through the following procedure.

i. The basic wind velocity

The basic wind velocity (V_b) is defined as a function of wind direction and time of year at 10 m above ground of terrain category II. Its formula is given by equation (2.1).

$$\mathrm{V_b} = \mathrm{V_{b,0}xC_{season}xC_{dir}} \tag{2.1}$$

$\mathrm{V_{b,0}}$: is the fundamental value of the basic wind velocity

$\mathrm{C_{season}}$: is the directional factor, the recommended value is 1,0.

$\mathrm{C_{dir}}$: is the season factor, the recommended value is 1,0.

ii. Terrain category definition

The terrain category and terrain parameter (Z_0 and $Z_{\min}$) provided in EN1991-1-4 are presented in table 2.1 a terrain category that fit the context of the building is selected.

Table 2. 1. Terrain category for wind action

Terrain category		z_0 m	z_{min} m
0	Sea or coastal area exposed to the open sea	0,003	1
I	Lakes or flat and horizontal area with negligible vegetation and without obstacles	0,01	1
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights	0,05	2
III	Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)	0,3	5
IV	Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m	1,0	10

iii. Terrain factor

The terrain factor $\mathrm{K_r}$ depend on the roughness length Z_0 and it is given by equation (2.2).

$$K_r = 0.19 \left(\frac{Z_0}{Z_{0,II}}\right)^{0.07} \quad (2.2)$$

Where:

$Z_{0,II} = 0{,}05$ m for terrain category II

Z_0, Z_{min} are height available for the terrain category used

Z_{max} is to be taken as 200 m

iv. Turbulence intensity

The turbulence intensity $I_V(Z)$ is expressed using equations (2.3) and (2.4).

$$I_V(Z) = \frac{K_1}{C_0(Z)\ln\left(\frac{Z}{Z_0}\right)} \quad \text{for} \quad Z_{min} \leq Z \leq Z_{max} \quad (2.3)$$

$$I_V(Z) = I_V(Z_{min}) \ \text{for} \ Z \leq Z_{min} \quad (2.4)$$

Where:

K_1 is the turbulence factor. The recommended value of K_1 is 1, 0.

C_0 is the orography factor. The recommended value of C_0 is 1, 0.

v. Exposure factor

The exposure factor $C_e(Z)$ take into account turbulence and it is defined by equations (2.5) and (2.6).

$$\text{For } Z \leq Z_{min}, \ C_e(Z) = C_e(Z_{min}) = K_r{}^2 C_0 \ln\frac{Z_{min}}{Z_0}\left(7 + C_0 \ln\frac{Z_{min}}{Z_0}\right) \quad (2.5)$$

$$\text{For } Z > Z_{min}, \ C_e(Z) = C_e(Z) = K_r{}^2 C_0 \ln\frac{Z}{Z_0}\left(7 + C_0 \ln\frac{Z}{Z_0}\right) \quad (2.6)$$

vi. Basic velocity pressure

The basic wind velocity (q_b) is given by equation (2.7).

$$q_b = \frac{1}{2}\rho {V_b}^2 \tag{2.7}$$

Where:

ρ is the density of air, the recommended value is 1, 25 kg/m3

V_b is the basic wind velocity

vii. Peak velocity pressure

The peak velocity is given by equations (2.8) and (2.9)

$$\text{For } Z \leq Z_{min}, \quad q_p(Z_e) = C_e(Z_{min})q_b \tag{2.8}$$

$$\text{For } Z > Z_{min}, \quad q_p(Z_e) = C_e(Z)\, q_b \tag{2.9}$$

Z_e is the reference height for the external pressure

viii. Wind pressure on external surfaces

The wind pressure in external surfaces(W_e) is defined using equation (2.10).

$$W_e = q_p(Z_e)C_{pe}\, C_s C_d \tag{2.10}$$

Where:

$q_p(Z_e)$ is the peak velocity pressure

C_{pe} is the pressure coefficient for the external pressure

$C_s C_d$ is the structural factor

The shape coefficient C_{pe} is obtained using values provided in figure2.1

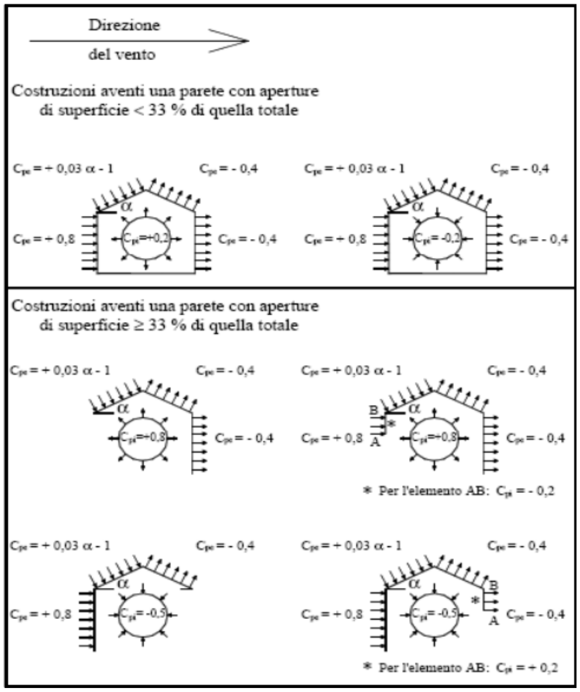


Figure 2. 1. Values of the shape coefficient C_{pe}

The structural factor C_sC_d is provided by EN1991-1-4, it takes into account the effect on wind actions from the non-simultaneous occurrence of peak wind pressures on the surface (C_s) together with the effect of the vibrations of the structure due to turbulence (C_d). C_sC_d is taken as equal to 1.

iv. Wind forces

A simplified approach is used to evaluate wind forces on structural elements. The wind pressure on the external surface is an area load acting on the facade of the building, this area load is converted into line loads acting on columns. Considering the wind pressure (area load) acting on the facade, the facade is assumed to carry the loads as a multi-span girder to the columns using a reference strip length of 1m in each facade of the building. Each columns act as support on the multi-span girder loaded with a line load obtained by multiplying the wind pressure time and the reference strip length. For each reference strip length, the reaction on each support (column) is a point load. However, through the entire facade height of the building, the load acting on the column is a distributed line load.

2.4.2. Materials

In the design of reinforced concrete elements, the properties of concrete and steel are needed.

2.4.2.1.Concrete

a. Exposure classes

Exposure conditions are chemical and physical conditions to which the structure is exposed in addition to the mechanical actions. Environmental conditions influence the durability of concrete structures. Annexe B provides the different Exposures classes defined in EN1992-1-1

b. Concrete strength characteristics

The recommended structural class defined by EN 1992-1-1 is S4 (design working life of 50 years). Base on the exposure classes, the indicative characteristic cylinder strength of concrete f_{ck} is obtained using the table provided in annexe B. The concrete strength characteristics are provided in table 2.2. α_{cc}= 0.85, $\alpha_{ct} = 1$ and γ_c=1.5 .

Table 2. 2. Formulas of concrete strength characteristics

Descriptions	Designations	Formulas
Design compressive strength	f_{cd}	$f_{cd} = \alpha_{cc}(f_{ck}/\gamma_c)$
Mean cylinder strength of concrete	f_{cm}	$f_{cm} = f_{ck} + 8Mpa$
Mean tensile strength of concrete	f_{ctm}	$f_{ctm} = 0.3f_{ck}{}^{2/3}$
Characteristic axial tensile strength of concrete	$f_{ctk,0.05}$	$f_{ctk,0.05} = 0.7f_{ctm}$
Design axial tensile strength of concrete	f_{ctkd}	$f_{ctkd} = \alpha_{ct}\,(f_{ctk}/\gamma_c)$
Tangent modulus of elasticity	E_C	$E_C = 22000(f_{cm}/10)^{0.3}$

c. Concrete cover

The minimum concrete cover (C_{min}) required is evaluated using equation (2.11).

$$C_{min} = \max[C_{min,b}; (C_{min,dur} - \Delta C_{dur,add}); 10mm] \quad (2.11)$$

Where:

$\Delta C_{dur,\ add}$ = 0 mm without further specification, is 0 mm.

The evaluation of the minimum concrete cover starts with the identification of the exposure class for the different structural elements followed by the identification of the minimum strength class for each exposure class and end by the evaluation of the minimum cover for durability ($C_{min,dur}$) and bond ($C_{min,b}$). Once the environmental class and the related concrete strength class have been identified, the structural class is chosen and can be modified based on the criteria listed in table 2.3.

Table 2. 3. Structural class according to EN1992

Criterion	Structural Class Exposure Class according to Table 4.1 *(Eurocode 2)*						
	X0	X1	XC2/XC3	XC4	XD1	XD2 / XS1	XD3 / XS2 / XS3
Design Working Life of 100 years	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2
Strength Class 1) 2)	≥ C30/37 reduce class by 1	≥ C30/37 reduce class by 1	≥ C35/45 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C45/55 reduce class by 1
Member with slab geometry (position of reinforcement not affected by construction process)	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1
Special Quality Control of the concrete production ensured	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1

Based on the environmental and structural classes the minimum concrete cover for durability ($C_{min,dur}$) is identified in table 2.4 .

Table 2. 4. Minimum concrete cover for durability

Structural Class	Environmental Requirement for $c_{min,dur}$ (mm)						
	Exposure Class according to Table 4.1 *(Eurocode 2)*						
	X0	XC1	XC2 / XC3	XC4	XD1 / XS1	XD2 / XS2	XD3 / XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

The nominal reinforcement concrete cover (C_{nom}) is defined by equation (2.12).

$$C_{nom} = \max[C_{min} + \Delta C_{dev}\ ;\ 20mm] \qquad (2.12)$$

Where:

$\Delta C_{dev} = 10mm$ without further specification

2.4.2.2.Reinforcing steel

The characteristic and design value of steel reinforcement are respectively f_{yk} and f_{yd} , the partial safety factor at the ultimate limit state (γ_s) is equal to 1.15 and the modulus of elasticity is consider to be E_s = 200000 MPa. The steel reinforcement characteristics are provided in table 2.5.

Table 2. 5. Formulas for steel reinforcement characteristics

Descriptions	**Designations**	**Formulas**
Design yield strength	f_{yd}	$f_{yd} = f_{yk}/\gamma_s$
Deformation at design yields strength	$\varepsilon_{s,yd}$	$\varepsilon_{s,yd} = f_{yd}/E_s$
Elastic modular ratio	n_0	$n_0 = E_s/E_c$

2.4.3. Predimensioning of super structural elements

2.4.3.1. Preliminary size of Beams

For a rectangular beam cross-section, the height (h) of the beam is governed by the span length (L) and the width of the base (b) is given in function of the height of the beam as presented in table 2.6. The length of the beams, the height and the base are all in centimetres.

Table 2. 6. Preliminary sizes of beams

Descriptions	Height (h) in cm	Base (b) in cm
Isolated beams	$h = \frac{L}{10}$	$15 \leq b \leq \frac{2}{3}h$
Continuous beams	$\frac{L_{max}}{15} \leq h \leq \frac{L_{max}}{12}$	$15 \leq b \leq \frac{2}{3}h$

2.4.3.2.Preliminary design of ribbed hollow block slab

The transversal section of a ribbed hollow slab illustrated in figure 2.2

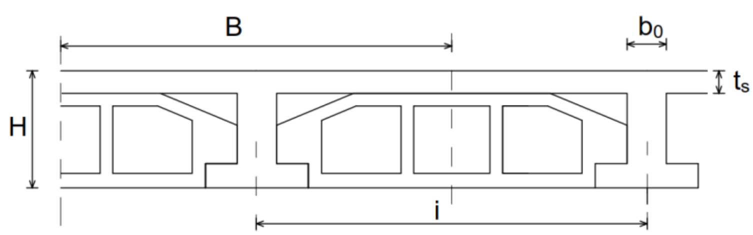


Figure 2. 2 Cross-section of a ribbed hollow block slab

The preliminary design of a ribbed hollow block slab is presented in table 2.7.

Table 2. 7. Preliminary design of a ribbed hollow block slab

Descriptions	Designations	Formulas
Total floor height	H	$H > \frac{L_{max}}{25}$
The thickness of the compressed slab	t_s	$t_s \geq 4cm$
The joist step	i	$i \leq 15t_s$
The web thickness of the rib	b_0	$b_0 \geq \max\left(\frac{1}{8}i, 8cm\right)$
Influence length of loads	B	$B = i$

2.4.3.3.Preliminary size of Columns

The geometrical cross-section for columns used is rectangular. The initial size of the column is obtained using an approach based on the estimation of the load on the column through the influence area of the column. The influence area of a column considered is illustrated in figure2.3.

The preliminary design of columns is made using the column associated with the largest influence area. The influence area is defined by equation (2.13).

$$A_{ij} = \left(\frac{X_1+X_2}{2}\right)\left(\frac{Y_1+Y_2}{2}\right) \tag{2.13}$$

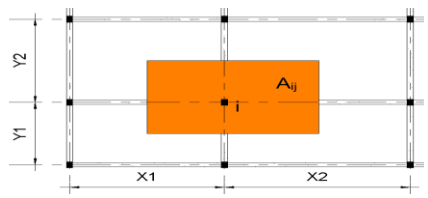


Figure 2. 3. Influence area of a column

For a specific column "i", at a floor "j", the influence area "A_{ij}" is used to evaluate at ULS the associated load "W_{ij}". Considering a building of "n" floors in total, the maximum load "N_{ik}" on the column "i" at the floor "k" is given by equation (2.14).

$$N_{ik} = \sum_{j=k+1}^{n} A_{ij}\, W_{ij} + 1.35G_{b(ij)} \tag{2.14}$$

Where:

$W_{ij} = 1.35G_{s(ij)} + 1.5Q_{ij}$

$Q_{ij} = q_k A_{ij}$, is the imposed load on the slab at floor "j";

$G_{s(ij)} = A_{ij} h_t \rho_c$, is the dead load of the slab at floor "j";

$\mathrm{G_{b(ij)}} = \mathrm{bh_{r0}\rho_c}\left(\frac{\mathrm{X_1+X_2}}{2} + \frac{\mathrm{Y_1+Y_2}}{2}\right)$ is the dead load of beams connected to the specific column "i" considered at floor "j".

The cross-area of the column $A_{Col,jk}$ is given by equation (2.15).

$$A_{Col,jk} \geq \frac{N_{ik}}{0.7\, f_{cd}} \tag{2.15}$$

2.5.Structural analysis method

The structural analysis aims to establish the distribution of either internal forces and moments, or stresses, strains or displacements, over the whole or part of a structure.

2.5.1. Structural building information modelling workflow

The workflow used to perform the structural analysis and design is based on direct link interoperability between S-BIM software and FEM software where the S- BIM software is Revit and the FEM software is Midas Gen. The direct link between the two software is done using the plugin "Midas link for Revit structure" which enable the transfer of the structural model created in Revit to Midas and enable also to deliver back the structural model from Midas to Revit through the file format Midas Gen text file(.mgt). The BIM framework used to perform the structural design of the building is displayed in figure 2.4.

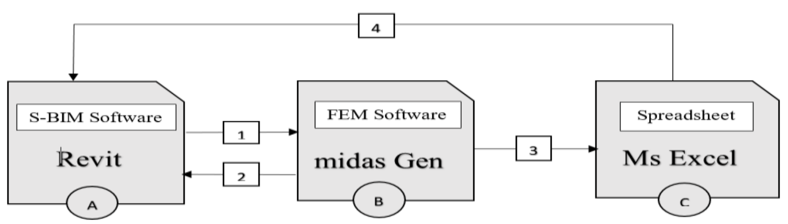


Figure 2. 4. Schematic representation of the BIM framework adopted

The BIM framework displayed in figure 2.4 is implemented following a process starting from the left to the right.

Base on the architectural BIM model, the structural BIM model made of geometrical properties of structural elements, the strength of materials and boundary conditions) is developed using the software Revit. The structural BIM model is available in an analytical view and a physical view. The plugin Midas link for Revit structure is used to link the structural BIM software and the

FEM software enabling the transfer of the structural BIM model into the FEM software, this process starts with the definition of units followed by section mapping and material mapping between the two software. These operations are useful to share section and material properties of structural elements from Revit to Midas Gen.

The structural model now available in Midas Gen is enriched with data such as rigid floor diagrams for the floors, loads (dead loads, live loads, wind loads) and loads combinations. Structural analysis is performed and the results of the analysis such as mode shape, internal forces (bending moments, shear forces, axial forces and reactions) are available.

Suitable internal forces are used to design structural elements using an Excel spreadsheet developed base on the procedure to design structural elements according to Eurocodes. Then the required steel reinforcement area, bar diameter and spacing etc. for structural elements are available.

If the result of the structural design is satisfactory to Eurocode 2, steel reinforcement drawings details of structural elements are performed using the software Revit and also the quantity take-off for concrete and steel material for structural elements is provided using the software Revit. If the result of the structural design is not satisfactory to Eurocode 2 or some optimization of the design are needed, some changes on the geometrical cross-section of the structural elements and materials properties can be made in Midas Gen and updated in Revit

2.5.2. Loads

Different types of loads are applied to the building, the relevant activities here is to bring out a resume of permanent and variable loads acting on the building and partial safety factors associated to variable loads available on the building.

2.5.3. Loads cases

Load cases referred to the possible arrangement of actions to evaluate the most enormous solicitations in member or structure, The UK NA to Eurocode 2, Part 1–1 allows any of the sets of load arrangements provided in figure 2.5 to be used for both the ultimate limit state and serviceability limit state.

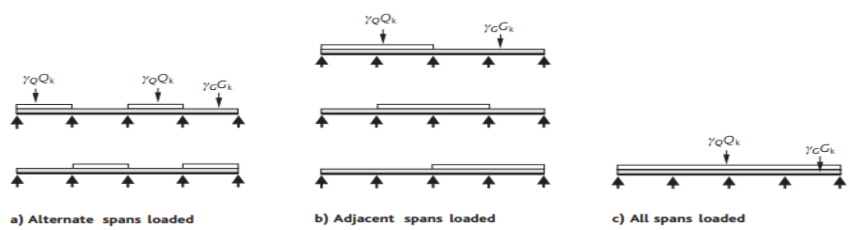


Figure 2. 5. Load arrangements for beams and slabs according to UK NA- EN1992-1-1

2.5.4. Loads combinations

Loads combinations given by EN1990 are used to verify both the ultimate limit state and serviceability limit state when designing structural elements. All the permanent loads are supposed to act at every time, the wind load in X-direction and Wind load in Y-direction are supposed to never act at the same time.

The load combination at the ultimate limit state is given by equation (2.16).

$$\sum_{j>1} \gamma_{G,j,sup} G_{k,j,sup} + \sum_{j>1} \gamma_{G,j,inf} G_{k,j,inf} + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \qquad (2.16)$$

Where:

$\gamma_{G,j,sup}$ = 1.35 is used when permanent loads are unfavourable

$\gamma_{G,j,inf}$ = 1 is used when permanent are favourable

γ_Q = 1.5 when variable loads are unfavourable and equal to 0 when favourable

For the serviceability limit state, characteristic load combination, frequent load combination and quasi-permanent load combination are defined respectively by equations (2.17), (2.18) and (2.19). The serviceability limit state is used for the verification of deflection, stress and cracking.

$$\sum G_{k,j} + Q_{k,1} + \sum_{i>1} \psi_{0,i} Q_{k,i} \quad (2.16)$$

$$\sum G_{k,j} + Q_{1,1} + \sum_{i>1} \psi_{2,i} Q_{k,i} \quad (2.17)$$

$$\sum G_{k,j} + \sum_{i>1} \psi_{2,i} Q_{k,i} \quad (2.19)$$

Where:

$G_{k,j}$ is the characteristic value of the permanent action j

$Q_{k,1}$ is the characteristic value of the variable action 1

$Q_{k,i}$ is the characteristic value of accompanying variable action i

Ψ is the combination factor, it depends on the use category of the building

2.6. Structural design of reinforced concrete structural elements

The structural design of reinforced concrete structural elements aims to provide the procedure to design reinforced concrete beams, columns, and ribbed slabs, staircases and foundations.

2.6.1. Design of the beam

2.6.1.1. Flexural design

The structural design of a beam start after carrying out an analysis of the structure and obtained bending moments, axial forces and shear forces on the beam element. The procedure used to design a rectangular section with the height h, the width b and the effective depth d is valid for concrete materials with concrete class less than C50/60.

a. Determination of the relative flexural bending

The relative flexural bending (K) is determined using equation (2.20) and the limit relative flexural bending $K_{bal} = 0.167$ is determined by considering the neutral axis x $= 0.45d$.

$$\mathrm{K} = \frac{M_{Ed}}{\mathrm{bd^2\ f_{ck}}} \quad (2.20)$$

For $\mathrm{K} \leq \mathrm{K_{bal}} = 0.167$, no compression reinforcements are required. The section is called a singly reinforced rectangular section.

For $\mathrm{K} > \mathrm{K_{bal}} = 0.167$, compressive reinforcements are required. The section is called a doubly reinforced rectangular section.

b. Determination of longitudinal steel reinforcement

i. Singly reinforced rectangular section

For singly reinforced rectangular section ($\mathrm{K} \leq \mathrm{K_{bal}} = 0.167$), the lever arm Z and the steel reinforcement required are determined respectively using equations (2. 21) and (2. 22)

$$Z = \frac{d}{2}\left[1 + \sqrt{1 - 3.53K}\right] \leq 0.95\mathrm{d} \qquad (2.\ 21)$$

$$A_s = \frac{M_{Ed}}{f_{yd} Z} \qquad (2.\ 22)$$

ii. Doubly reinforcement rectangular cross-section

For doubly reinforcement rectangular cross-section($\mathrm{K} \geq \mathrm{K_{bal}} = 0.167$) as illustrated in figure 2.6, the compression $\mathrm{A'_s}$ and the tension steel $\mathrm{A_s}$ are calculated respectively using equations (2.23) and (2.24).

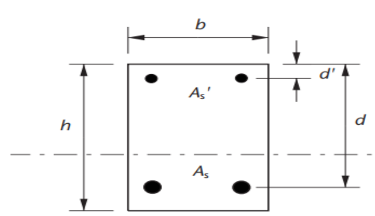


Figure 2. 6. Doubly reinforced rectangular cross-section

$$\mathrm{A'_s} = \frac{(\mathrm{K}-\mathrm{K_{bal}})\mathrm{f_{ck}bd^2}}{\mathrm{f_{yd}(d-d')}} \qquad (2.23)$$

$$\mathrm{A_s} = \frac{\mathrm{K_{bal}\,f_{ck}\,bd^2}}{\mathrm{f_{yd}\,Z_{bal}}} + \mathrm{A'_s} \qquad (2.24)$$

With $Z_{bal} = \frac{d}{2}\left[1 + \sqrt{1 - 3.53K_{bal}}\right] \leq 0.95\text{d}$

iii. Minimum and maximum steel reinforcement area

The minimum and maximum steel reinforcement area are obtained using respectively equations (2.25) and (2.26)

$$\mathrm{A}_{s,mim} = \max\left(\frac{0.26 f_{ctm} bd}{f_{yk}};\ 0.013bd\right) \tag{2.25}$$

$$\mathrm{A}_{s,max} = 0.04A_c \tag{2.26}$$

2.6.1.2. Shear reinforcement design

The design of shear reinforcement in structural member for concrete ≤ C50/60 is based on a truss model approach, in which compression and tension chords are spaced apart by a system consisting of inclined struts and vertical or inclined reinforcement bar. Having the shear V_{Ed} at distance d from the support, the following procedure is used to design the shear reinforcement in the beam element.

a. Evaluation of the shear capacity of the concrete

The shear capacity of the concrete is given by equation (2.27)

$$V_{rdc} = \left[\left(\frac{0.18}{\gamma_c}\right) k(100\rho_1 f_{ck})^{\frac{1}{3}} + 0.15\sigma_{cp}\right] b_w d \geq (\mathcal{V}_{min} + 0.15\sigma_{cp}) b_w d \tag{2.27}$$

Where:

$$K = 1 + \sqrt{\frac{200}{d}} \leq 2$$

$$\rho_1 = \frac{A_{sl}}{b_w} \leq 0.02$$

$$\sigma_{cp} = \frac{N_{Ed}}{A_c} \leq 0.2\, f_{cd}$$

$$\mathcal{V}_{min} = 0.035K^{\frac{3}{2}} f_{ck}{}^{\frac{1}{2}}$$

b. Evaluation of shear reinforcement

If $V_{Ed} < V_{rdc}$, no shear reinforcement is needed, however the minimum area of stirrups

Provided by EN 1992 is defined by equation (2.28)

$$\left(\frac{A_{sw}}{s}\right)_{min} = \frac{0.08 b_w \sqrt{f_{ck}}}{f_{ywk}} \qquad (2.28)$$

If $V_{Ed} \geq V_{rdc}$, shear reinforcement is needed, shear reinforcement is evaluated based on a truss model approach as presented in figure2.7.

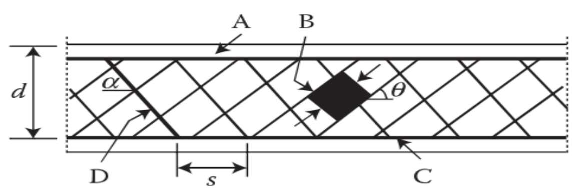


Figure 2. 7. Truss model approach for shear reinforcement design

. The value of angle θ between the struts and the axis of the member used is in the range $21.8^0 \leq \theta \leq 45^0$ .the angle θ is found using equation (2.29).

$$\theta = arsin\left(\frac{2\,V_{Ed}}{\alpha_{cw}\, b_w Z \mathcal{V}_1 f_{cd}}\right) \qquad (2.29)$$

where :

$\alpha_{cw} = 1,\ \mathcal{V}_1 = 0.5,\ Z = 0.9d$

if $\theta < 21.8^0$, the value of $\theta = 21.8^0$ used

if $21.8^0 \leq \theta \leq 45^0$, the value of θ used is the one obtained

if $\theta > 45^0$, the cross-section has to be change

The transversal reinforcement and the maximum spacing between bars are evaluated using respectively equations (2.30) and (2.31).

$$\frac{A_{sw}}{s} = \frac{V_{Ed}}{Zf_{ywd}\cot\theta} \quad (2.30)$$

$$\mathrm{S_{max}} = \min(\,0.75\mathrm{d};\ 600\ \mathrm{mm}\,) \quad (2.31)$$

The maximum shear resistance is defined by equation (2.32)

$$V_{\mathrm{Rd}} = \min(V_{\mathrm{Rdc}};\ V_{\mathrm{Rds}}) \quad (2.32)$$

Where:

$$V_{\mathrm{Rdc}} = \frac{\alpha_{cw}\, Zbf_{cd}}{cot\theta + tan\theta} \text{ and } V_{\mathrm{Rds}} = \left(\frac{A_{sw}}{S}\right) Zf_{ywd}\cot\theta$$

2.6.1.3. Design of beam at the serviceability limit state

The EN 1992-1-1 allows the control of the deflection, verification of stress limitation and the control of deflection at serviceability limit states.

a. Verification of stress limitation

The verification of the allowable stress on the beam is done with characteristic (rare) combination and quasi-permanent combination. The maximum stresses in both the concrete (σ_c) and reinforcement (σ_s)are computed and it must be verified that these stresses are less than the maximum permissible stresses values indicated by EC2.

Characteristic combination: $\sigma_c \leq 0.6 f_{ck}$ (2.33)

Quasi–permanent combinations : $\sigma_c \leq 0.45 f_{ck}$ (2.34)

Characteristic combination: $\sigma_s \leq 0.8 f_{ck}$ (2.35)

The evaluation of the stress on concrete and steel is performed starting from the calculation of the modular ratio (n) followed by the position of the neutral axis (x) then the moment of inertia of the uncracked section ($\mathrm{J_{cr}}$) and finally calculation of stress on concrete (σ_c) and steel (σ_s) using respectively equations (2.36), (2.37), (2.38), (2.39) and (2.40)

$$\mathrm{n} = \frac{\mathrm{E_s}}{\mathrm{E_c}} \quad (2.36)$$

$$x = \frac{-n(A_s+A'_s)+\sqrt{n(A_s+A'_s)^2+2bn(A_s d+A'_s d')}}{b} \quad (2.37)$$

$$J_{cr} = \frac{bx^3}{3} + n\,A_s(d-x)^2 + nA'_s(x-d')^2 \quad (2.38)$$

$$\sigma_c = \frac{M_{Ed}X}{J_{cr}} \quad (2.39)$$

$$\sigma_s = n * \frac{M_{Ed}(d-X)}{J_{cr}} \quad (2.40)$$

Where:

A_s and A'_s are the upper and lower reinforcement inside the section

b. Verification of cracking

Using the exposure class of the structural element, the maximum crack allowable for the quasi load combination is determined from table 2.5 then according to the stress in steel bars, cracking can be controlled by restricting either maximum bar diameter or maximum spacing of rebars following requirements provided in table 1 and table 2 of annexe C

Table 2. 8. Recommended value of crack width (Wmax)

Exposure Class	Reinforced members and prestressed members with unbonded tendons	Prestressed members with bonded tendons
	Quasi-permanent load combination	Frequent load combination
X0, XC1	0,4[1]	0,2
XC2, XC3, XC4	0,3	0,2[2]
XD1, XD2, XS1, XS2, XS3		Decompression

Note 1: For X0, XC1 exposure classes, crack width has no influence on durability and this limit is set to guarantee acceptable appearance. In the absence of appearance conditions this limit may be relaxed.
Note 2: For these exposure classes, in addition, decompression should be checked under the quasi-permanent combination of loads.

c. Control of the deflection

The control of the deflection is done by calculating the theoretical value of the ratio $\left(\frac{l}{d}\right)$ using equations (2.42) and (2.43) and also multiplying it by the correction factors given by equation (2.41)

$$\frac{310}{\sigma_s} = \frac{500\, A_{s,prov}}{f_{yk} A_{s,req}} \tag{2.41}$$

$$\left(\frac{l}{d}\right) = K\left[11 + 1.5\sqrt{f_{ck}}\,\frac{\rho_0}{\rho} + 3.2\sqrt{f_{ck}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2}}\right] \quad \text{If} \;\; \rho \le \rho_0 \tag{2.42}$$

$$\left(\frac{l}{d}\right) = K\left[11 + 1.5\sqrt{f_{ck}}\,\frac{\rho_0}{\rho-\rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_0}}\right] \quad \text{If} \;\; \rho > \rho_0 \tag{2.43}$$

Where:

σ_s is the tensile steel stress under the design load at SLS

$A_{s,prov}$ and $A_{s,req}$ are respectively the area of steel provided and the area of steel required for ULS at the section considered.

$^{l}/_{d}$ is the limit span/depth,

K is the factor to take into account the different structural systems, the value is given in Annex D

ρ_0 is the reference reinforcement ratio, $\rho_0 = \sqrt{f_{ck}}\,.\,10^{-3}$

$\rho = \frac{A_s}{bd}$ is the required tension ratio to resist the moment due to the design loads

$\rho' = \frac{A_s'}{bd}$ is the required compression ratio to resist the moment due to the design loads.

2.6.1.4.Detailed design

The detailed design consists of providing adequate spacing between bars, adequate mandrel diameter for the bars and a sufficient anchorage length.

a. Spacing between bars

The minimum spacing between bar ($S_{b,min}$) and the spacing between bars is defined by equations (2.44) and (2.45). Figure 2.8 presents spacing between bars

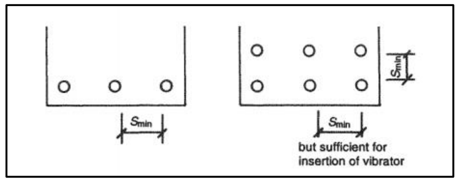


Figure 2. 8. Spacing between bars

$$S_{b,min} = \max\{\varnothing_{Long};\ d_g + 5;\ 20mm\} \quad (2.44)$$

Where:

d_g is the maximum diameter of aggregate

$$S_b = \frac{b - 2(C_{nom} - \varnothing_{link}) - \sum_{i \geq 1} n_{b,i}\varnothing_{Long,i}}{n_b - 1} \quad (2.45)$$

Where:

n_b is the number of bars and S_b is the space between the bar on the bottom and to the top layer of reinforcement.

b. Mandrel diameters

A sufficient mandrel diameter enable to avoid bending cracks in the bar and failure of the concrete inside the bent, the mandrel diameter is given by equation (2.46)

$$\varnothing_m \geq \varnothing_{m,min} \quad (2.46)$$

Where:

$$\varnothing_{m,min} = \begin{Bmatrix} 4\varnothing & \text{if } \varnothing \leq 16\text{ mm} \\ 7\varnothing & \text{if } \varnothing \geq 16\text{mm} \end{Bmatrix}$$

c. Anchorage length

The design anchor length ((l_{bd}) is given by equation (2.47). It depends on the basic anchor length $\left(l_{b,rqb}\right)$ and the minimum anchorage length$\left(l_{b,min}\right)$.

$$l_{bd} = \alpha_1\alpha_2\alpha_3\alpha_4\alpha_5 \, l_{b,rqb} \geq l_{b,min} \tag{2.47}$$

Where:

$\alpha_1\alpha_2\alpha_3\alpha_4\alpha_5 = \alpha = 1$

$l_{b,rqb} = \left(\frac{\varnothing}{4}\right)\left(\frac{\sigma_{sd}}{f_{bd}}\right)$

σ_{sd} is the stress in steel bars

$f_{bd} = 2.25\eta_1\eta_2\left(\frac{f_{ctk,0.05}}{\gamma_c}\right)$ Is the design value of ultimate bond stress,

$$\eta_1 = \begin{Bmatrix} 1 & \text{for good bond condition} \\ 0.7 & \text{for poor bond condition} \end{Bmatrix}$$

$$\eta_2 = \begin{Bmatrix} 1 & \text{for } \varnothing \leq 32\text{mm} \\ \dfrac{132-\varnothing}{100} & \text{for } \varnothing > 32\text{mm} \end{Bmatrix}$$

$l_{b,min} = \max\left[\alpha.\, l_{b,rqb};\, 10\varnothing;\, 100\text{mm}\right]$

$\alpha = 0.6$ for an anchor in compression and $\alpha = 0.3$ for an anchor in tension

2.6.2. Design of column

The procedure used to design a column start with the determination of the slenderness and slenderness limit of the column then the longitudinal steel reinforcement is obtained followed by the M-N interaction verification and finally with a detailing design.

2.4.2.2.1 Determination of slenderness and slenderness limit

The slenderness (λ) of a rectangular column is given by equation (2.48) and it depends on the effective length(l_0) of the column).

$$\lambda = 3.46\frac{l_0}{h} \tag{2.48}$$

Where

$l_0 = 0.5l\sqrt{\left(1+\frac{K_1}{0.45+K_1}\right)\left(1+\frac{K_2}{0.45+K_2}\right)}$ for Braced members

$l_0 = max\left\{l\sqrt{1+10\frac{K_1K_2}{K_1+K_2}}; l\left(1+\frac{K_1}{1+K_1}\right)\left(1+\frac{K_2}{1+K_2}\right)\right\}$ for unbraced members

$K_1 = K_2 = \frac{\text{Column stiffness}}{\sum \text{beams stiffness}} = \frac{\left(I/l\right)_{column}}{\sum 2\left(I/l\right)_{beam}} \geq 0.1$, K_1 and K_2 are the relative flexibilities of rotational restraints at the ends of the column

The slenderness limit λ_{lim} is determined using equation (2.49)

$$\lambda_{lim} = \frac{20ABC}{\sqrt{n}} \leq \frac{15.4C}{\sqrt{n}} \tag{2.49}$$

Where:

$n = \frac{N_{Ed}}{A_c f_{cd}}$

$A = 1/\left(1+0.2\varphi_{ef}\right)$, if φ_{ef} is not known $A = 0.7$

$B = \sqrt{1+2\omega}$, if the reinforcement ratio is not known, $B = 1.1$

For braced members, $C = 1.7 - r_m$ and $r_m = \frac{M_{01}}{M_{02}}$, M_{01} and M_{02} are they moment at the top and bottom end of the column with their sign, $|M_{02}| \geq |M_{01}|$.

For unbraced members, C = 0.7.

$\lambda < \lambda_{lim}$, the column is a non-slender column (short column) and $\lambda \geq \lambda_{lim}$, the column is a slender column.

2.4.2.2.2 Determination of longitudinal steel reinforcement

The ultimate design bending moment adopted for a non-slender column is given by equation (2.50)

$$M_{Ed} = Max\{ |M_{02}|, |M_{01}| \} + e_i N_{Ed} \geq e_0 N_{Ed} \quad (2.50)$$

Where:

$e_0 = Max\left\{\frac{h}{30}, 20\right\}$ and $e_i = \frac{l}{400}$ all units in millimetres.

The longitudinal reinforcement is evaluated considering symmetrical reinforcement. Design charts are used to determine steel reinforcement starting by evaluating the ratios $\mu_{Ed} = \frac{M_{Ed}}{bh^2 f_{ck}}$, $V_{Ed} = \frac{N_{Ed}}{bhf_{ck}}$ and $\frac{d'}{\mathrm{h}} = \frac{C_{nom}+\varnothing_{link}+\varnothing_{bar}/2}{\mathrm{h}}$ then a suitable design chart is selected according to the ratio $\frac{d'}{\mathrm{h}}$. Then using μ_{Ed} and V_{Ed} on the suitable design chart, the ratio $\omega_{Ed} = \frac{A_s f_{yk}}{bhf_{ck}}$ is determined. The figure2.9 illustrate the design chart for a rectangular column with a ratio $\frac{d'}{\mathrm{h}} = 0.2$.

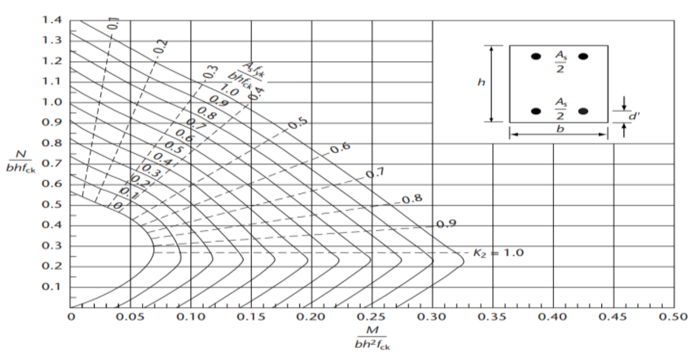


Figure 2. 9. Column design chart for rectangular columns with the ratio $\frac{\mathrm{d'}}{\mathrm{h}} = 0.2$.

The total area of longitudinal steel reinforcement (A_s) and the minimum area of longitudinal steel reinforcement ($\mathrm{A_{s,min}}$) are determined using equations (2.51) and (2.52).

$$\mathrm{A_s} = \frac{\omega_{\mathrm{Ed}} \mathrm{bhf_{ck}}}{\mathrm{bhf_{ck}}} \quad (2.51)$$

$$A_{s,min} = \frac{0.1N_{Ed}}{fyd} \geq 0.002A_c \quad (2.52)$$

2.4.2.3 M-N interaction verification

To verify the structural design of the reinforced concrete column, the M- N interaction diagram representing the moment and axial capacities of the column is plotted. The limit configuration beyond which failure occurs is obtained by representing significant points of the M-N interaction diagram. The expressions of critical points needed to plot the M-N interaction curve are in the number of six.

1st point, the section is completely subjected to tension. The resisting moment and the resisting axial force are given by equations (2.53) and (2.54).

$$N_{Rd} = A_s f_{yd} + A_s{}' f_{yd} \quad (2.53)$$

$$M_{Rd} = A_s f_{yd}\left(\frac{h}{2} - d'\right) - A_s{}' f_{yd}\left(\frac{h}{2} - d'\right) \quad (2.54)$$

2nd point, the section is completely subjected to tension. Verification, if upper steel is yielded, is performed using equation (2.55), the resisting moment and axial force are given by equations (2.55) and (2.56).

$$\varepsilon'_s = \left[(d - d')\left(\frac{\varepsilon_s + \varepsilon_c}{d}\right)\right] - \varepsilon_s > \varepsilon_{yd}, \text{ upper steel yielded} \quad (2.55)$$

If steel yielded then $\sigma_s{}' = f_{yd}$ if not $\sigma_s{}' = \varepsilon'_s E_s$

$$N_{Rd} = A_s f_{yd} + A_s{}'\sigma_s{}' \quad (2.56)$$

$$M_{Rd} = A_s f_{yd}\left(\frac{h}{2} - d'\right) - A_s{}'\sigma_s{}'\left(\frac{h}{2} - d'\right) \quad (2.57)$$

Where:

$\sigma_s{}' = f_{yd}$ if the steel yielded and $\sigma_s{}' = \varepsilon'_s E_s$ is the steel has not yielded

3rd point verification if upper steel is yielded is performed using equation (2.55), the neutral axis (x) is computed using equation (2.58), the resisting moment and the resisting axial force are given by equations (2.59) and (2.60).

$$x = \frac{\varepsilon_c . d'}{\varepsilon_c - \varepsilon'_s} \tag{2.58}$$

$$N_{Rd} = -0.8. b. x. f_{cd} + A_s f_{yd} - A_s{}' \sigma_s{}' \tag{2.59}$$

$$M_{Rd} = 0.8. b. x. f_{cd} \left(\frac{h}{2} - 0.4x\right) + A_s f_{yd} \left(\frac{h}{2} - d'\right) + A_s{}' \sigma_s{}' \left(\frac{h}{2} - d'\right) \tag{2.60}$$

4th point, failure is due to concrete and the lower reinforcement reaches exactly ε_{yd} the resisting moment and the resisting axial force are given by equations (2.61) and (2.62).

$$N_{Rd} = -0.8. b. x. f_{cd} + A_s f_{yd} - A_s{}' \sigma_s{}' \tag{2.61}$$

$$M_{Rd} = 0.8. b. x. f_{cd} \left(\frac{h}{2} - 0.4x\right) + A_s f_{yd} \left(\frac{h}{2} - d'\right) + A_s{}' \sigma_s{}' \left(\frac{h}{2} - d'\right) \tag{2.62}$$

5th point, the neutral axis is equal to the effective depth of the section $(x = d)$ the resisting moment and the resisting axial force are given by equations (2.63) and (2.64).

$$N_{Rd} = -0.8. b. x. f_{cd} - A_s f_{yd} - A_s{}' \sigma_s{}' \tag{2.63}$$

$$M_{Rd} = 0.8. b. x. f_{cd} \left(\frac{h}{2} - 0.4x\right) + A_s f_{yd} \left(\frac{h}{2} - d'\right) + A_s{}' \sigma_s{}' \left(\frac{h}{2} - d'\right) \tag{2.64}$$

6th point: the section is uniformly compressed, $\varepsilon_c = \varepsilon_s = 0.2\%$ the resisting moment and the resisting axial force are given by equations (2.65) and (2.66).

$$N_{Rd} = -b. h. f_{cd} - A_s f_{yd} - A_s{}' f_{yd} \tag{2.65}$$

$$M_{Rd} = A_s f_{yd} \left(\frac{h}{2} - d'\right) - A_s{}' \sigma_s{}' \left(\frac{h}{2} - d'\right) \tag{2.66}$$

2.4.2.4 Detailing design

A minimum of four bars is required in a rectangular column with a minimum bar diameter of 12 mm. In region away from laps, the maximum area of reinforcement ($A_{s,max}$) is equal to $0.04A_c$ and in a region with laps ($A_{s,max}$) is equal to $0.08A_c$.

The diameter of stirrups is given by equation (2.67).

$$\varnothing_{\text{link}} = \frac{1}{4}\varnothing_{\text{long}} \geq 6\text{mm} \tag{2.67}$$

The maximum spacing ($S_{b,max}$) between stirrups is given by equation (2.68).

$$S_{b,max} = \min\left\{20\text{x} \min(\varnothing_{\text{long}}), \min(b,h), 400 \text{ mm}\right\} \tag{2.68}$$

2.6.3. Design of ribbed hollow bloc slab

The procedure used to design the ribbed hollow block slab is closed to the one of designing the beams. The design is carried out at the ultimate and serviceability limit state.

2.6.3.1. Design of ribbed slab at the ultimate limit state

The determination of solicitations on the element considered to be designed is the first step then the design of longitudinal and transversal reinforcement is followed.

a. Determination of solicitations on the rib

Using the influence length B as illustrated in figure 2.10, dead load and live load lying on the slab are linearly distributed on a rib.

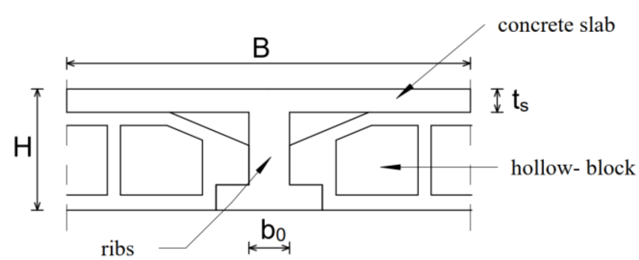


Figure 2. 10. Transversal cross-section of a ribbed hollow bloc slab

Different loads combinations are defined to have a maximum bending moment in span and at supports using the static scheme with a fixed support at the end and pinned support at the end as illustrated in figure 2.11.

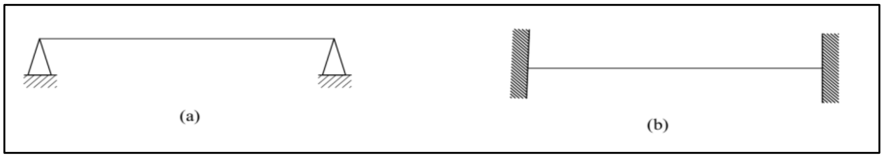


Figure 2. 11 Static scheme considered for the analysis of a ribbed slab

b. Design of steel reinforcement rebars

The amount of steel reinforcing bars inside the ribs, the neutral axis of the section and the resisting moment are calculated using respectively equations (2.69), (2.70) and (2.71).

$$A_S = \frac{M_{Ed}}{0.9df_{yd}} \tag{2.69}$$

$$X = \frac{A_{s,prov}f_{yd}}{0.8bf_{cd}} \tag{2.70}$$

$$M_{Rd} = A_{s,prov}f_{yd}(d - 0.4X) \tag{2.71}$$

Steel reinforcement of diameter 6 m is provided inside the compression table in both longitudinal and transversal directions of the ribbed slab.

c. Design of shear at ultimate limit state

The design value of the shear (V_{Ed}) is the one recorded at distance d from the face of the support member. The design shear resistance of the member without shear reinforcement ($V_{Rd,c}$) is computed using equation (2.27) then the decision of using shear reinforcement is taken. If shear reinforcement is needed, it is computed using equation (2.30).

2.6.3.2. Design of ribbed slab at serviceability limit state

The design of the rib at serviceability limit state is performed by the following steps:

- Determination of solicitations at serviceability limit state on the rib
- Verification of the stress limitation is performed using equations (2.39) and (2.40).
- Verification on cracking performed cracking is performed by restricting either maximum bar diameter or maximum spacing of rebars following requirements provided in table 1 and table 2 of annexe B.

- Verification of the deflection is handle by calculating the theoretical value of the ratio $\left(\frac{l}{d}\right)_{allow}$ using equation (2.42) or (2.43) and compare this value to the actual limit span depth $\left(\frac{l}{d}\right)_{actual}$

2.6.4. Structural design of staircases

Staircases provide means of movement from one floor to another in a structure, the structural design of the staircase is done following the procedure presented.

2.6.4.1. Preliminary design of the staircase

The preliminary design of a staircase illustrated in figure 2.12 consists of defining a suitable value of the waist (h) and also the average thickness (t) of the staircase using respectively equations (2.72) and (2.73).

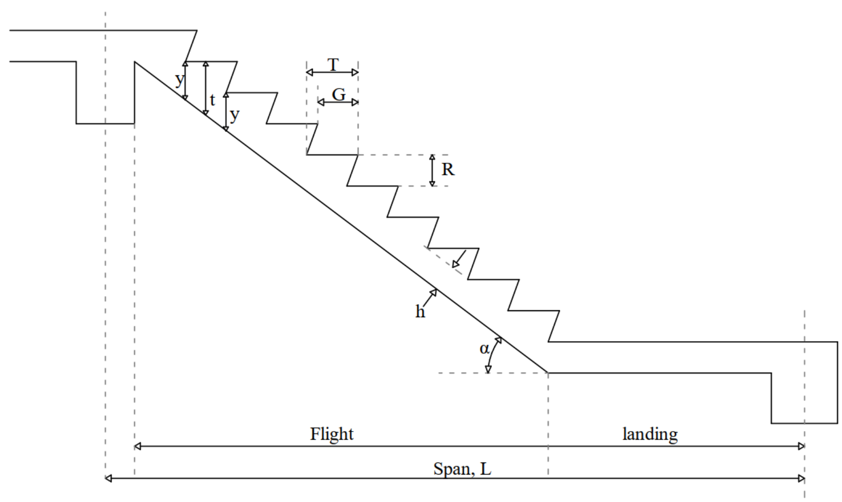


Figure 2. 12 Longitudinal cross-sections of a staircase

$$h \geq \frac{L}{28} \tag{2.72}$$

$$t = \frac{y+(y+R)}{2} \tag{2.73}$$

Where:

$$y = h\left[\frac{\sqrt{(R^2+G^2)}}{G}\right]$$

2.6.4.2. Structural Analysis of the static scheme of the staircase

The analysis of the structural static scheme of the staircase is to obtain maximum bending moments and shear forces. The permanent loads (G) consist of the slab self-weight, dead load of finished materials such as marble, plaster placed on the staircase. The variable load (Q) is selected following EN1991-1-1. The load combination at ULS and SLS is defined are and maximum bending moment in span and at supports are obtained using the static scheme with a fixed support at the end and pinned support at the end as illustrated in figure 2.11.

2.6.4.3.Design of staircase at the ultimate limit state

Longitudinal reinforcements along the stair are designed using a rectangular beam cross-section with a base of 1m width and height equal to the waist of the stair. The transverse reinforcement is considered to be 20 % of the longitudinal reinforcement. The procedure to design shear forces is the same as in the case to design shear for a rectangular beam section, the shear resistance of the member without shear reinforcement ($V_{Rd,c}$) is computed using equation (2.27).

2.6.4.4.Design of stair at serviceability limit state

The verification of the deflection and cracking is performed using the same equations as in the case of a rectangular beam section

2.6.4.5. Detailing

The detailing of a staircase is done with particular attention to opening joints, the figure 2.13 illustrate how longitudinal reinforcement has to be placed at particular openings.

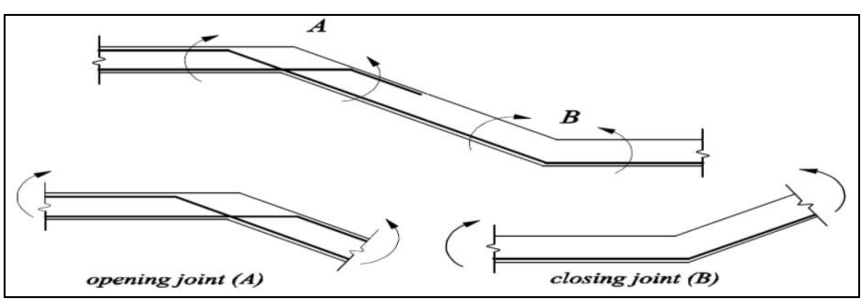


Figure 2. 13 Details of joints on a staircase

2.6.5. Design of foundations

Spread foundations such as pad footing, combined footing and strap foundation are designed using solicitations at serviceability limit state and the allowable bearing pressure of the soil to compute the geometrical size of the foundation then solicitations at the ultimate limit state are used to perform a detailed structural design.

2.6.5.1. Design of pad footing

The prescriptive method is used to design the pad foundation starting with the preliminary design followed by the verification of maximum punching, the determination of the bearing pressure at the base, the determination of steel reinforcement and the verification of punching shear force and verification of shear resistance.

a. Preliminary design

The plan size dimensions of the footing are determined using the allowable bearing pressure (p) and axial force at the serviceability limit state $N_{Ed(sls)}$. The plan size dimensions of the footings are obtained by assuming that the overhangs (d_0) of the footing that exceeds from column face in both directions are identical as presented in figure2.14

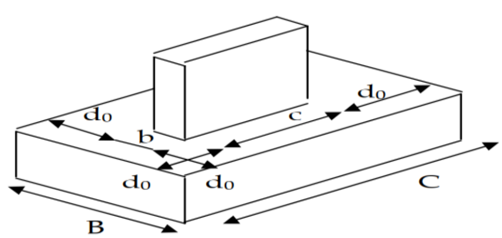


Figure 2. 14 Dimensions of pad footings

Starting with $\frac{N_{Ed(sls)}}{BC} \leq p$ to ensure the equilibrium of pressure and considering $B = b + 2d_0$ and $C = c + 2d_0$, the overhang d_0 is obtained using equation (2.74).

$$d_o = \frac{\left[-(c+b)+\sqrt{\left[(c+b)^2-4\left(cb-\frac{N_{Ed(sls)}}{p}\right)\right]}\right]}{4} \tag{2.74}$$

The provisional size $B = b + 2d_0$ and $C = c + 2d_0$ are determined then they are increased by 5% to 10% and after the value is round up to the nearest 50 mm.

The height h of the footing is defined by equation (2.75).

$$h \geq \frac{B-b}{4} + 50 = \frac{C-c}{4} + 50 = \frac{d_0}{2} + 50 \qquad (2.75)$$

The values of B, C and h are adopted as dimensions of the footing once the verifications on stress under the base of the footing performed using equation (2.76) is valid.

$$\frac{N_{Ed(sls)} + W_{(sls)}}{CB} = \frac{N_{Ed(sls)} + CBh\rho_c}{CB} \leq p \qquad (2.76)$$

b. Verification of maximum punching shear resistance

The verification of maximum punching shear resistance at the column perimeter permits the adoption of the suitable height of the footing. The relation to ensuring maximum punching shear resistance is given by equation (2.77).

$$V_{Ed} < V_{Rd,max} \qquad (2.77)$$

Where:

$V_{Ed} = N_{Ed(Uls)}$ is the shear force at column perimeter

$V_{Rd,max} = 0.5ud\left[0.6\left(1 - \frac{f_{ck}}{250}\right)f_{cd}\right]$ is the maximum punching shear resistance

u is the column perimeter and d the effective depth of the section

c. Determination of the bearing pressure at the base of the footing

For a column centre to the base of the footing A linear constant distribution of pressure occurs at the base of the footing and the soil is compressed everywhere at the base of the foundation. The constant pressure p is defined by equation (2.78).

$$P = \frac{N}{A} = \frac{N_u}{BC} \qquad (2.78)$$

d. Determination of reinforcement

For footing subjected to axial force only, the footing acts as a cantilever loaded by soil pressure P and supported by the column as presented in figure2.15 the maximum bending moments occur at the face of the column.

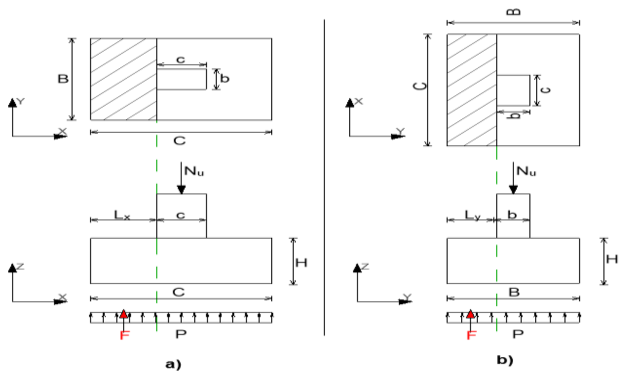


Figure 2. 15 Constant pressure distribution under the pad footing

The bending moment around M_y around the y axis and Mx around the x-axis are defined respectively by equations (2.79) and (2.80)

$$M_x = PC\frac{(B-b)^2}{8} \tag{2.79}$$

$$M_y = PB\frac{(C-c)^2}{8} \tag{2.80}$$

The longitudinal reinforcement in the x-direction is provided using the moment M_y and a rectangular beam cross-section of base B and height H and effective depth d. The longitudinal reinforcement in the y-direction is provided using the moment M_x and a rectangular beam cross-section of base C and height H and effective depth d. steel reinforcement in both x and y direction is obtained using equations (2.20), (2.21), and (2.22).

e. Verification of punching shear

The punching shear force (V_p) is given by equation (2.81).

$$V_{p(Ed)} = P(A - A_p)ud \tag{2.81}$$

Where:

$A = BC$ is the area of the footing

$u = 2(a+b) + 4\pi d$ Is the basic control perimeter

$A_p = 2(a+b)(2d) + 4\pi d^2 + ab$

Ap is the area within the control perimeter as presented in figure 2.16

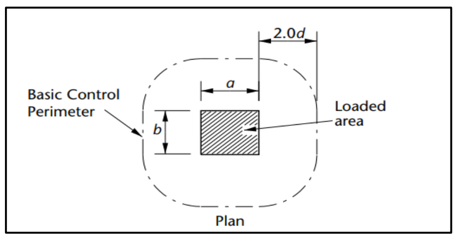


Figure 2. 16. Area of punching shear

The punching shear resistance is given by equation (2.82)

$$V_{rdc} = \text{Max}\left[0.12k(100\rho_1 f_{ck})^{\frac{1}{3}};\ \mathcal{V}_{min}\right]Bd \tag{2.82}$$

Where:

$$\rho_1 = \sqrt{\left(\frac{A_{sly}}{Cd_y}\frac{A_{slx}}{Bd_x}\right)} \leq 0.02$$

The punching shear is verified if equation (2.83) is satisfied

$$V_{rdc} > V_{p(Ed)} \tag{2.83}$$

f. Verification of shear resistance

The shear force at distance d from the face of the column is illustrated in figure 2.17

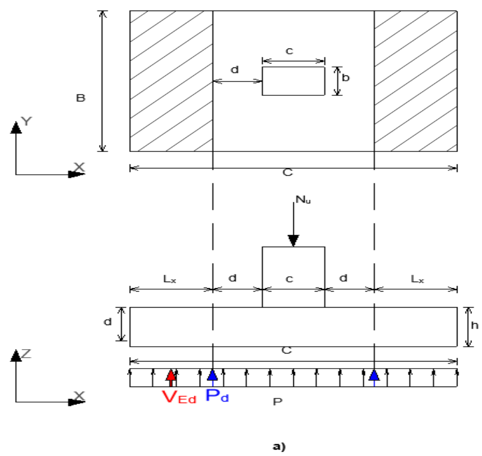


Figure 2. 17. Shear force at distance d from the face of the column

The shear force $V_{(Ed)}$ at distance d from the face of the column is defined by equation (2.84) and the shear resistance V_{rdc} is given by equation (2.85)

$$V_{(Ed)} = PB\left[\left(\frac{C-c}{2}\right) - d\right] \quad (2.84)$$

$$V_{rdc} = \mathrm{Max}\left[0.12k(100\rho_1 f_{ck})^{\frac{1}{3}}\,;\ \mathcal{V}_{min}\right]Bd \quad (2.85)$$

The shear is verified if equation (2.86) is satisfied

$$V_{rdc} > V_{(Ed)} \quad (2.86)$$

g. Detail design

The minimum bar diameter to provide for footing is 12 mm. For a footing with a square base, the reinforcement to resist bending is distributed uniformly across the full width of the footing. For a rectangular base footing, the reinforcement in the short direction is distributed with closer spacing in the region under and near the column.

2.6.5.2. Design of combined footing

A combined footing is used when two columns are close together and the separate footing would overlap. The structural design procedure of a combined footing that supports two columns on soil with a safe bearing pressure starts with the preliminary design, then the verification of maximum punching shear resistance followed by the determination of steel reinforcement and verification of shear resistance.

a. Preliminary design

Using actions at serviceability limit state. The required cross-area (A_{req}) is calculated using equation (2.87) then dimensions B and C of the cross base area illustrated in figure 2.18 are chosen.

$$\mathrm{A_{req}} = \mathrm{B.C} \geq \frac{\mathrm{N_1+N_2}}{\sigma_{\mathrm{net}}} \tag{2.87}$$

Where:

$\sigma_{\mathrm{net}} = \sigma - 0.000025\mathrm{h}$

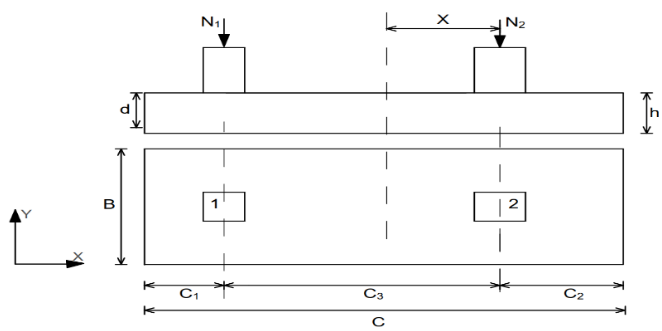


Figure 2. 18. Geometrical size of combined footing

The determination of C_1 and C_2 at the base of the footing respectively from column 1 and column 2 start by calculating the centroid (X) of the combined footing base by taking moments about the centre line of column 2. The value of X, $C_1$and C_2 are given respectively by equations (2.88), (2.89) and (2.90).

$$\mathrm{X} = \frac{\mathrm{N_1\,C_3}}{\mathrm{N_1+N_2}} \tag{2.88}$$

$$\mathrm{C_2} = \frac{\mathrm{C}}{2} - \mathrm{X} \tag{2.89}$$

$$\mathrm{C_1} = \mathrm{C} - \mathrm{C_2} - \mathrm{C_3} \tag{2.90}$$

b. Verification of maximum punching shear resistance

The verification of maximum punching shear resistance punching shear at column 1 perimeter and column 2 perimeter is computed using equation (2.77) and enable the adoption of a suitable height of the footing

c. Structural design of combined footing

The structural design consists of the determination of steel reinforcements and verification of shear resistance.

i. Determination of longitudinal bending and shear forces

The longitudinal bending moment and shear forces are obtained using a static scheme model presented in figure 2.19.

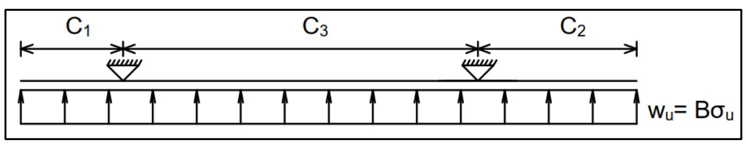


Figure 2. 19. Static scheme model of combined footing

The upward bearing pressure w_u is given by equation (2.91).

$$w_u = B\sigma_u \quad (2.91)$$

Where:

$\sigma_u = \frac{N_{1u}+N_{2u}}{BC}$ Is the bearing pressure at the ultimate limit state

ii. Determination of transversal bending moment

The transversal reinforcement is designed per meter length of the footing width with a bending moment defined by equation (2.92)

$$M_{Ed} = \sigma_u \frac{B^2}{8} \quad (2.92)$$

The procedure to determine the steel reinforcement to resist bending moment is the same as for a rectangular beam using equations (2.21), (2.22) and (2.23).

iii. Verification of shear resistance

The shear force at distance 1d of the column face is defined by equation (2.93)

$$V_{Ed} = V - \sigma_u Bd \qquad (2.93)$$

The verification of the shear resistance is done using equations (2.85) and (2.86)

2.6.5.3. Design of strap footing

A strap beam is constructed between the exterior footing and the adjacent interior footing to restrain the overturning moment due to the eccentric load on the exterior footing. The design assumption is that the strap is infinitely rigid, it is a pure flexural member and does not take soil reaction.

a. Preliminary design

The preliminary design of the strap footing illustrated in figure 2.20 consists of the determination of the geometrical size of the strap beam and the footings.

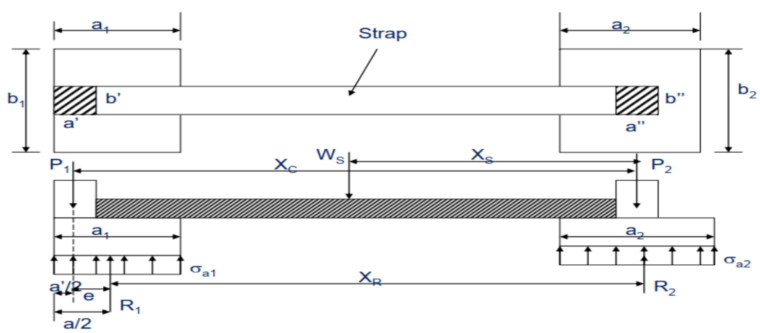


Figure 2. 20. Geometrical size of strap footing

The height of the beam (H), the width of the beam (B) and also itself- weight are determined using equations (2.94), (2.95) and (2.96)

$$H \geq \frac{X_C}{10} \qquad (2.94)$$

$$B \geq \min(a', a'') \tag{2.95}$$

$$W_s = \rho_c B H X_C \tag{2.96}$$

The geometrical size of the footings depends on soils reactions obtained using vertical loading P_1, P_2 and W_s at the serviceability limit state. A trial value of the rectangular outer footing (a_1) is performed then the eccentricity (e) of the soil reaction R_1 and the distance (X_R) between soil reactions R_1 and R_2 are provided following equations (2.97) and (2.98).

$$e = \frac{a_1 - a'}{2} \tag{2.97}$$

$$X_R = X_c - e \tag{2.98}$$

The soil reaction R_1 is defined by equation (2.99) by taking moments about R_2 and the soil reaction R_2 is defined by (2.100 using vertical equilibrium of forces

$$R_1 = P_1 \frac{X_C}{X_R} + W_s \frac{X_C}{2X_R} \tag{2.99}$$

$$R_2 = P_1 + P_2 + W_s - R_1 \tag{2.100}$$

The sizes of footings are defined using equations (2.101), (2.102) and (2.103), the best size selection is the one that gives $b_1 = b_2$

$$b_1 = \frac{R_1}{a_1 \sigma} \tag{2.101}$$

$$a_2 = \left[\frac{R_2}{\left(R_1/a_1\right)}\right] \tag{2.102}$$

$$b_2 = \frac{R_2}{a_2 \sigma} \tag{2.103}$$

The net actual bearing pressure σ_1 and σ_2 respectively under footing 1 and footing 2. Are determined using equations (2.104) and (2.105). If the distribution under footing 1 and 2 are different, the second trial of distance a_1 will be made until σ_1 and σ_2 are closed.

$$\sigma_1 = \frac{R_1}{a_1} \tag{2.104}$$

$$\sigma_2 = \frac{R_2}{a_2} \tag{2.105}$$

b. Verification of punching shear resistance

A trial value of a depth (h) of each footing is selected then verification on the fact that the depth selected can provide sufficient punching shear resistance at the perimeter of the column of each footing is performed using equation (2.77).

c. Structural analysis of the strap footing

The longitudinal bending moment and shear force diagram along the length of the footings are determined using the static presented in figure 2.21

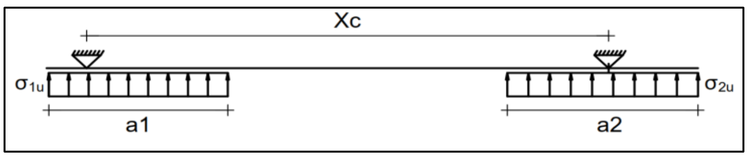


Figure 2. 21. Static scheme model of strap footing

The ultimate bearing pressure σ_{1u} and σ_{2u} under respectively footing 1 and footing 2 are determined by applying the loading associated with the ultimate limit state

d. Structural design of a strap beam

The design of the strap beam is performed using longitudinal moment and shear force obtained from the result of the structural analysis of the static scheme model provided in figure 2.21. Stirrups would be placed at a constant spacing and also it would extend into footings over the supports to give a monolithic foundation.

e. Structural design of the outer footing

The outer footing is designed as a base with bending in one direction and supported by the strap beam. The bending moment given by equation (2.106) is determined at the face of the column using uniform pressure at the base of the pad footing 1.

$$M_{Ed} = \frac{\sigma_{1u} a_1 (b_1 - b')^2}{8} \tag{2.106}$$

Where:

$$\sigma_{1u} = \frac{R_{1(uls)}}{a_1 * b_1}$$

f. Structural design of the inner footing

The inner footing is designed as a pad footing with bending moment in both directions, the longitudinal bending moment is recorded from the result of the structural analysis of the static scheme provided in figure 2.21.

The transversal bending moment given by equation (2.107) is determined at the face of the column using uniform pressure at the base of the pad footing 2.

$$M_{Ed} = \frac{\sigma_{2u} a_2 (b_2 - b'')^2}{8} \tag{2.107}$$

Where:

$$\sigma_{2u} = \frac{R_{2(uls)}}{a_2 * b_2}$$

2.7. Construction documents

After performing the structural design of structural elements of the building, construction documents are used to construct the frame structure of the building on site. The software Revit is used for the production of construction documents such as steel reinforcement plans and also the quantity take-off.

2.7.1. Steel reinforcement plans

Steel reinforcement plans consist of 2D and 3D steel reinforcement drawings associated with bar bending details and bar bending schedule of structural elements. Reinforcement drawings describe and locate the reinforcement in relation to the outline of the concrete work. It contains information such as: sizes of structural members, the concrete cover of the members, rebars diameter and spacing which are communicated to the site works in a clear, concise and unambiguous manner.

2.7.2. Quantity take-off

Quantity take-off of concrete material for structural elements is provided in tabular form. The total volume of concrete material at each story level of the building for beams, columns, floor slab and staircase is provided. Also, the total volume of concrete material for the foundations elements is provided.

Conclusion

This chapter aimed to provide a methodology to perform the structural design of a reinforced concrete building using the BIM process. The structural analysis of the building starts by creating a structural BIM model with the software Revit containing data such as a geometrical section of elements, material properties and boundaries conditions then the structural model is transferred to the software Midas Gen where relevant data such as loads and load combinations are added and the structural analysis is performed. After base on the results of the structural analysis, the structural design of structural elements is performed using an Excel spreadsheet developed based on formulas provided by European standards. In the end, the results of the structural design are used to enrich the structural BIM model within Revit software then construction documents such as steel reinforcement plans and quantity take-off materials is performed.

CHAPTER 3 RESULT AND INTERPRETATION

Introduction

The methodology provided in the previous chapter is applied to the case study and the results are presented here. This chapter start with the General presentation of the site followed by the description of the site and the presentation of the project. In addition the result of the preliminary design consisting of defining the different loads, material properties and initial geometrical size of structural elements are provided. This is followed by a structural analysis using the BIM process to determine the internal forces on structural elements. Base on internal forces, super structural elements and sub structural elements are designed. Then a detailed report of the structural design consisting of steel reinforcement plans, rebar schedules and quantity take-off of concrete materials is provided.

3.1.General presentation of the site

Here, we present the study area through its location, geology, relief, climate, hydrology, population and socio-economic activities.

3.1.1 Geographic location

The building studied is the new administrative building constructed at the national advanced school of publics' works (NASPW) located in the centre region of Cameroon in the political capital Yaoundé at quarter Elig-Effa. Yaoundé is the political capital of Cameroon and the chief town of the Centre region, situated at latitude 3.87^{o} North and longitude 11.52^{o} East at an elevation of 760 meters above sea level. Located 300km from the Atlantic Ocean and surrounded by 7 hills, Yaoundé belongs to the Mfoundi division of the Centre region and measures a total surface area of 183km^2. The plan of Yaoundé presenting the localization of NASPW is presented in figure 3.1

Figure 3. 1. Map of Yaounde showing the location of the NASPW

3.1.2 Geology

The bedrock in Yaoundé is mainly composed of gneiss. This rock is a medium- to coarse-grained, semi schistose metamorphic rock. It is characterized by alternating light and dark bands differing in mineral composition.

3.1.3 Relief

The city Yaoundé is also called the city of seven hills. The city is located in the south of the central region and it is built on a network of hills dominated by the Mbam Minkom hill with an altitude of 1295m and the Nkolodom hill with an altitude of 1221 m. in the north-west sector the city is dominated by the mount Eloundem with an altitude of 1159m.

3.1.4 Climate

The climate in Yaoundé is a tropical climate with four seasons which consist of a light rainy season from May to June, a short dry season from July to October, a heavy rainy season from October to November, and a long dry season from December to May. The annual average temperature is 23 ^{0}C and the annual average precipitation is 1727mm.

3.1.5 Hydrology

The hydrographic network of Yaoundé is very dense and composed of rivers and few lakes. The Mfoundi River crosses the city from North to South and the artificial municipal lake with 800 m long and 300m width is located near the administrative centre of Yaoundé.

3.1.4 Population and socio-economic activities

Yaoundé has a total population estimated at 3.8 million in 2019. The city of Yaoundé is a cosmopolitan city with a considerable portion of the population coming from several other regions of the country. Most of Yaoundé's economy is centred around the administrative structure of the public's services and the diplomatic services. Due to these, Yaoundé has a higher standard of living and security than the rest of Cameroon. However, Yaoundé has a few industries such as breweries, sawmills, carpentry, tobacco, paper mills, machinery and building materials.

3.2. Description of the site

The site of the project is the National Advanced School of Public Works, it was created in 1974 under the name of the National School of Technology and took the name of NASPW in 1982. This school is dedicated to the training of architects, civil engineers, environmental engineers, urban planning engineers, surveying and cadastral engineers. The 3D view of the site is presented in figure 3.2

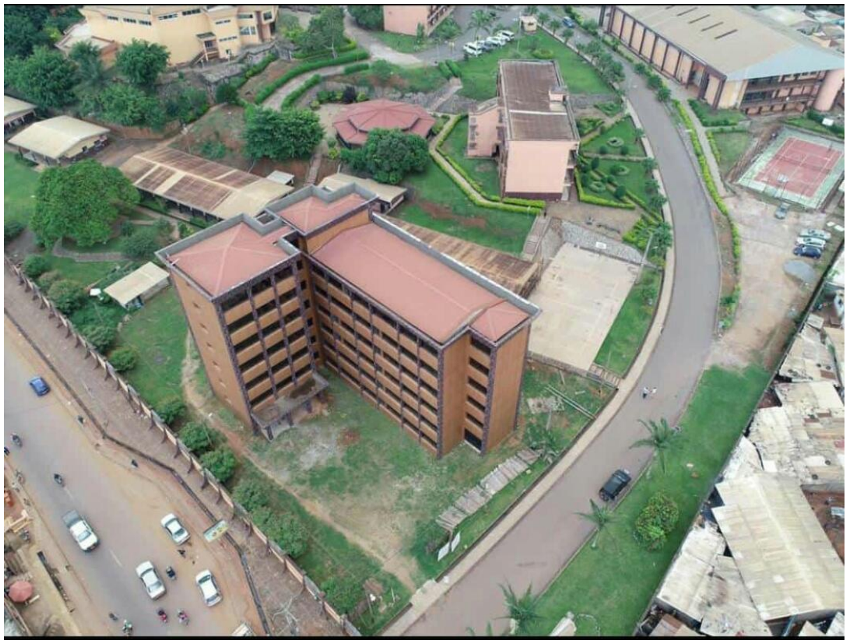


Figure 3. 2. 3D view of the site

3.3. Presentation of the project

The structure analyzed is a reinforced concrete building for administrative purposes located at the National Advanced School of Public Works in Yaoundé. Its construction began in 2014 and the load-bearing structure of the building has already been fully completed, the external envelope of the building is covered with alucobond material. The building is not yet occupied at the moment. The building consists mainly of meeting rooms and offices intended to be used mostly by the administrative staff of the NASPW. The building measures 18.6 m in height from the ground surface to the roof, the dimensions in the plan are 35.9 m long and 24.2 m in width. The 3D view of the building is presented in figure 3.3. The building is made up of three reinforced concrete building block frames (part A, part B and part C) each separated by a rupture joint as illustrated in figure 3.4. The ground surface, part A consist of reinforced concrete containing staircases, part B consist of an office and part C consists of an entrance hall with a reinforced staircase and reinforced concrete elevator shaft.

Figure 3. 3. The new administrative building of NASPW

Figure 3. 4. The first-floor plan view of the building

The assumed bearing pressure of the soil is 0.3 Mpa. The design working life is assumed to be 50years, the frame structure of parts A and B of the building consist of beams and columns with a rectangular cross-section and the slab is a ribbed hollow block slab. The structural framing plan of parts A and B of the building is provided in figure 3.5. The principal beams are in green colour and the secondary beams are in grey colour. The ribs are represented in the blue line running along the length of the building.

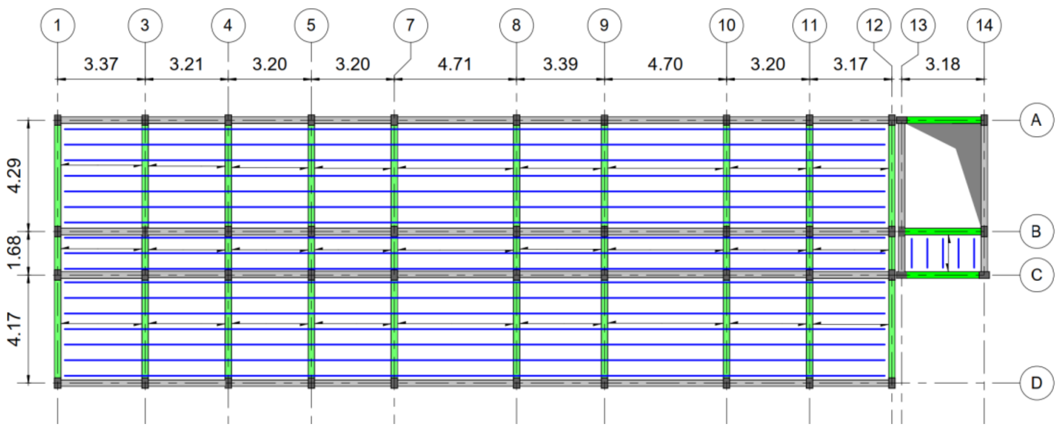


Figure 3. 5 Structural framing plan of parts A and B of the building

3.4.Presentation of the preliminary design

The presentation of the preliminary design consist of evaluating the actions on building, the materials properties and the predimensioning of structural elements.

3.4.1. Actions

The actions here consist of permanent actions which can be subdivided into self-weight of structural and non-structural elements, imposed loads on building and wind loads.

3.4.2.1. Permanents actions

The permanent actions consist of self-weight of the roof, floor slab and walls. The roof is considered as a concrete slab of 10 cm thickness, the self-weight is 2.5 kN/m$^{2.}$.

The self-weight of the ribbed hollow block floor (16+4) is 2.85 kN/m^2 and the self-weight of non-structural load consisting of mortar plaster under the slab, screed and tiles is 1.36 kN/m^2 the total self-weight of the slab is 4.21 kN/m^2.

The self-weight of the Wall with 3m height composed of masonry in hollow blocks of 15 cm thickness and mortar plaster of 1.5cm thickness is 7.7 kN/m.

3.4.2.2.Variables actions

The variable actions considered consist of imposed loads and wind actions.

a. Imposed loads

The imposed loads on the roof, floors and staircase are determined according to Euro code1. Concerning the roof, the imposed load is 1kN/m^2. For office building which refers to category B of building, imposed load on floors is considered as 2.5 kN/m^2 and imposed load on staircase considered as 3kN/m^2.

b. Wind actions

The parameters used to determine the wind force are fundamental basic wind velocity $V_{b,o}$=10.7 m/s, air density ρ = 1.25 kg/m^{3,} coefficients $c_0 = c_{season} = c_s c_d = c_{dir} = 1$, the terrain category is IV.

At 10 m height, the peak velocity pressure is $q_p = 84.16$ N/m^2 and the wind pressure on the external surface is $W_e(10)$= 101 N/m^2. At 18.6 m height, the peak velocity pressure is $q_p =$ 113.97 N/m^2 and the wind pressure on the external surface is $W_e(18.6)$= 113.97 N/m^2 .

Table 3.1 presents the values of loads acting on columns in building B in the Y direction and table 3.2 presents the values of loads acting on the column in the x-direction.

Table 3. 1 Values of wind loads acting on columns in the X- direction

	Wind load acting on columns in kN/m for building B								
Height	**axis 1**	**axis 3**	**axis 4**	**axis 5**	**axis 7**	**axis 8**	**axis 9**	**axis 10**	**axis 11**	**axis 12**
z ≤ 10m	0.13	0.37	0.31	0.29	0.43	0.4	0.4	0.42	0.33	0.13
Z =15.6 m	0.18	0.49	0.41	0.38	0.56	0.53	0.53	0.54	0.43	0.18

Z =18.6m		0.44	0.41	

Table 3. 2. Values of wind loads acting on columns in the Y- direction

Wind load acting on columns in kN/m for building B				
Height	**axis A**	**axis B**	**axis C**	**axis D**
z $\leq$ 10 m	0.22	0.3	0.3	0.2
Z=15.6 m	0.28	0.39	0.39	0.28

3.4.2. Materials

The materials consist of concrete and steel materials.

3.4.2.1. Concrete

The exposure classes, the strength class of structural elements, the strength properties of concrete materials and the concrete cover of structural elements are provided.

a. Exposure classes and concrete strength class

Assuming a 50 years working life and no special concrete production quality control, the exposure classes, the concrete class and the structural class of the different structural elements are presented in table 3.3.

Table 3. 3. Concrete class and the structural class of the different structural element

Element	Exposure	Concrete class	The structural class
Beams, columns and staircases	XC1	25/30	S4
Ribbed Slabs	XC1	25/30	S(4-1) =S3
`Foundations	XC2	25/30	S4

b. Strength of the concrete

Different strength properties of the concrete grade 25/30 are presented in table 3.4.

Table 3. 4. Strength properties of the concrete

Descriptions	Symbols	Value	Units
Characteristic cylinder strength	f_{ck}	25	Mpa
Design compressive strength	f_{cd}	14.2	Mpa
Mean compressive strength	f_{cm}	33	Mpa
Mean tensile strength	f_{ctm}	2.56	Mpa
Secant modulus of elasticity	E_{cm}	18810	Mpa
Characteristic axial tensile strength	$f_{ctk,0.05}$	1.79	Mpa

c. Concrete cover of structural elements

The concrete cover (C_{nom}) of the different structural elements are presented in table 3.5.

Table 3. 5. Concrete cover of different structural elements

Element	$C_{min,dur}$ (mm)	$C_{min,b}$ (mm)	C_{min}(mm)	C_{nom}(mm)
Beams and columns	15	14	15	25
Ribbed Slab	10	12	12	25
Stair cases	15	14	15	25
Foundations	25	14	25	35

3.4.2.2. Steel reinforcement

The steel reinforcement used is the ribbed bars type B450 C, the partial safety factor for steel strength at the ultimate limit state γ_s =1.15 and the modulus of elasticity $E_S = 200$ GPa. The characteristic yield strength f_{yk} is 450 MPa, the characteristic tensile strength f_{tk} is 540 MPa, the design yield strength f_{yd} are 391MPa and the elastic modular ratio n_0 is equal to 10.63

3.4.3. Predimensions of structural elements

3.4.3.1. Beams

The maximum beam bays length for part B of the building is 4.71 m. The overall depth (h) of continuous beams is assumed to follow $\frac{471}{15} \leq h \leq \frac{471}{12}$ so $31.4 \leq h \leq 39.25$. The selected value of h = 40 cm, the width b of the beam is governed by $b \leq \frac{2}{3}h = 26.7$cm, the selected value of b = 25cm.

3.4.3.2. Hollow ribbed slab

The overall height (H) of the slab depends on the maximum span length of the ribs which is 4.79m. The total height of the slab is 20 cm, the thickness of the rib is 7cm and the inter axis distance between ribs is 60 cm. The geometrical size of the one-way ribbed slab is presented in figure 3.6.

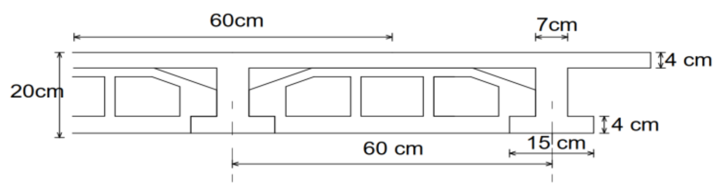


Figure 3. 6 Geometrical size of the ribbed slab

3.4.3.3. Columns

The preliminary design of columns is made using the interior column associated with the largest influence area = 12.01 mm^2 which is column B8. The ultimate axial force at the base is 875.91 kN and the section of column adopted is 25X 40 cm^2.

3.5. Structural analysis method

The structural analysis consists of modelling the load-bearing structure then adding loads and loads combinations then result such as mode shapes of the structure and also internal forces on members are obtained.

3.5.1. Structural building information modelling (S-BIM) workflow

The structural building information modelling workflow consists firstly by creating a structural BIM model of the concrete structure with the software Revit, this structural model is made of beams, columns, materials properties and boundary conditions at the base are fixed support. The physical model view of the structural model is presented in figure 3.7 and the analytical model view is presented in figure 3.8.

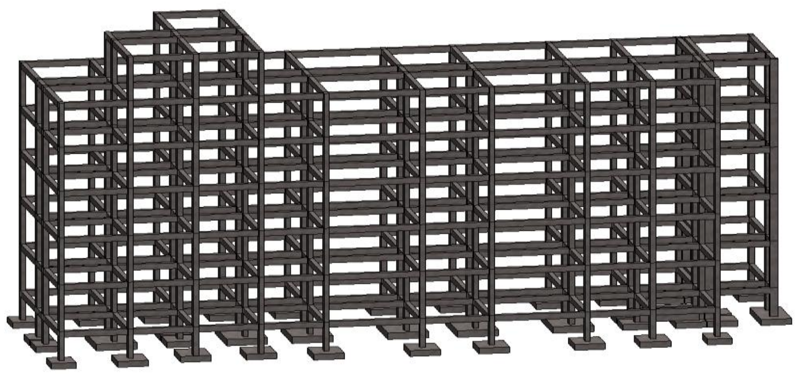


Figure 3. 7. Physical model view of parts A and B of the building

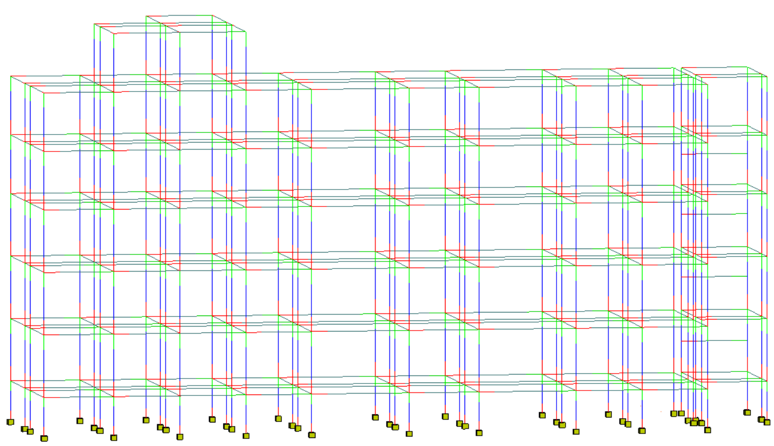


Figure 3. 8. Analytical model view of parts A and B of the building

Once the structural model is created with Revit, the interoperability between Revit and Midas is performed. Using the plugin ‘‘Midas link for Revit’’, the process of exporting the structural model from Revit to Midas start by defining units then the section mapping and material mapping are done. The dialogue box enabling to define section mapping between Revit and Midas is presented in figure 3.9.

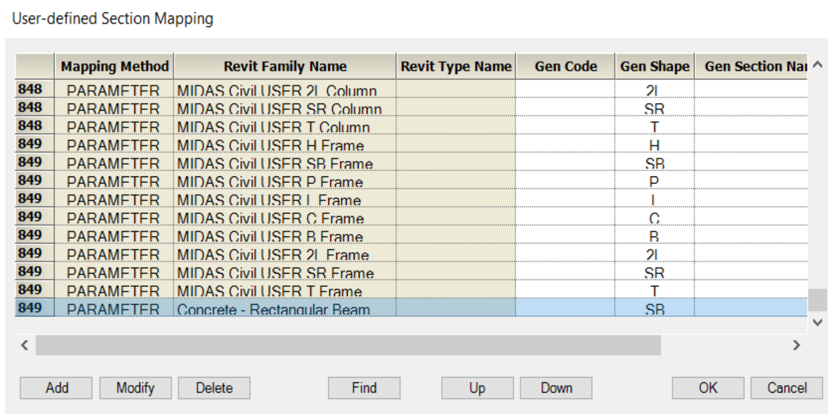


Figure 3. 9. Section mapping dialogue box

The dialogue box enabling the definition of a material mapping between Revit and Midas Gen is presented in figure 3.10.

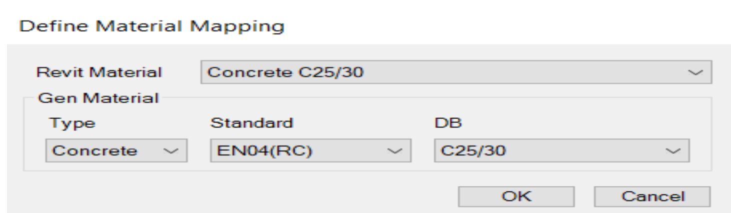


Figure 3. 10. The material mapping dialogue box

Once the process of exporting the model from Revit to Midas Gen is over, an overview of the different categories of data exchange between the structural model done in Revit and Midas Gen is available under the tools ”works” inside the “Tree menu “ properties palette. The different categories of data exchanged from the structural model between Revit and Midas gen such as stories, material properties, sections, boundaries conditions are presented in figure 3.11.

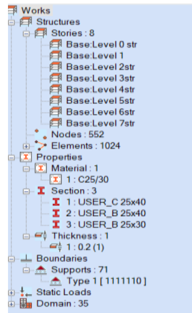


Figure 3. 11. Data exchanged between Revit and Midas Gen

The structural model exported from Revit to Midas Gen is presented in figure 3.12

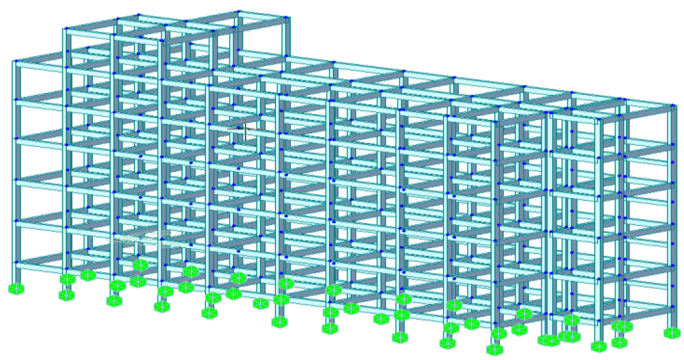


Figure 3. 12. Structural model of parts A and B exported from Revit to Midas Gen

3.5.2. Loads

A summary of the loads and the factor of combination associated are listed in table 3.6

Table 3. 6. A summary of the loads and the factor of combination associated

Class	Load name	Value of load	Factors	
			ψ_0	ψ_2
Dead load	a dead load of Roof	2.5 kN/m^2	-------	
	Total Dead load of ribbed hollow slab	3.95 kN/m^2		
	A dead load of façade wall and interior	7.7kN/m		
	A dead load of the interior partition wall	1 kN/m^2		

Environmental load	Wind load	0.10 kN/m^2 below 10m 0.13 kN/m^2 at 15.6 m 0.14 kN/m^2 at 18.6 m Linear rising between 10	0.6	0
Service load	Live load on roof	1 kN/m^2	0.7	0.3
	Live load for office	2.5 kN/m^2	0.7	0.3
	Live load on staircase	3 kN/m^2	0.7	0.3

3.5.3. Loads cases

The load consist is divided in two groups, load case due to permanent loads and load case due to variable loads.

3.5.3.1.Load case due to permanent load

The dead load of the bearing structure is associated with the load case SW1. It is calculated automatically by the FEM software Midas Gen based on the geometry and unit weight of the material.

The dead load of the roof is associated with the load case SW2. The dead load of the one-way ribbed floor slab with finishes is associated with load case SW3. The dead load of the slab is an area load set directly on the principal beams into a line load as illustrated in figure 3.13

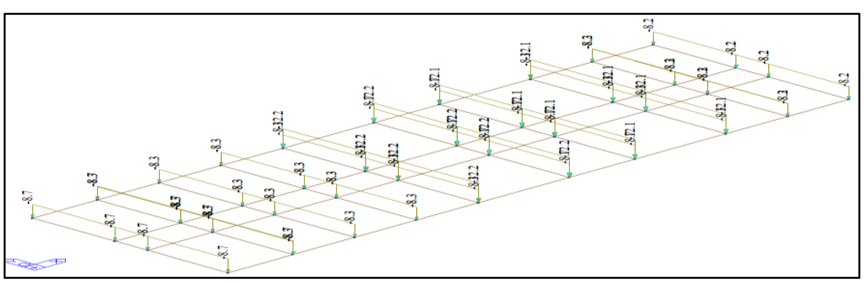


Figure 3. 13. The self-weight of the ribbed floor slab

The dead load of the façade and interior walls are associated with the load case SW4, the dead load of the walls are sets on beams as illustrated in figure 3.14.

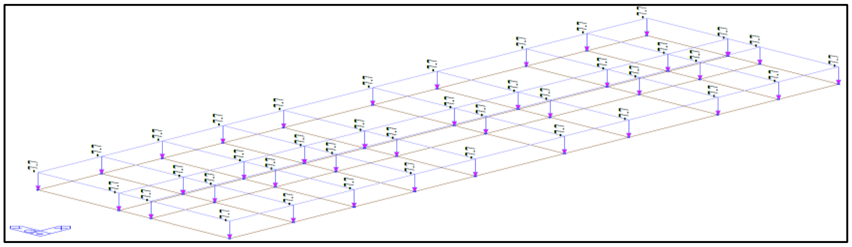


Figure 3. 14. A dead load of façade and interior walls sets on beams

A dead load of partition walls is associated with load case SW5, the dead load of partition walls is an area load that is directly set on the principal beams into a line load.

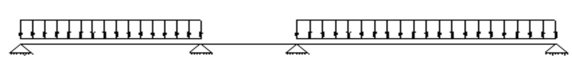
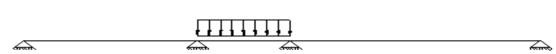
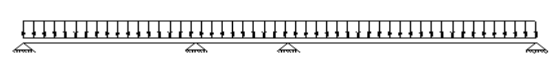
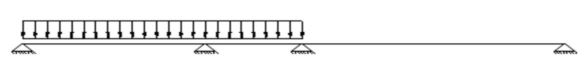
3.5.3.2.Load cases due to variables load

The load case due to variable loads consists of load case due to imposed load on floor levels and load case for wind actions.

a. Load case of imposed loads on floor levels

To obtain critical solicitations on members, different possible arrangements of live load on principal beams are defined. The table 3.7 describe the different load arrangement used.

Table 3. 7. The different load arrangements used

Description	Name	Representation of live load on principal beams
Alternate span loaded	LL(1)	
	LL(2)	
All span loaded	LL(3)	
Adjacent span loaded	LL(4)	
	LL(5)	

Load case LLR (1), LLR (2), LLR (3), LLR (4) and LLR (5) are the possible arrangement for the roof service load, they are implemented similarly as the case of floor service load

b. Load case for wind action

The wind load in the X- direction is set on columns as shown in figure 3.15

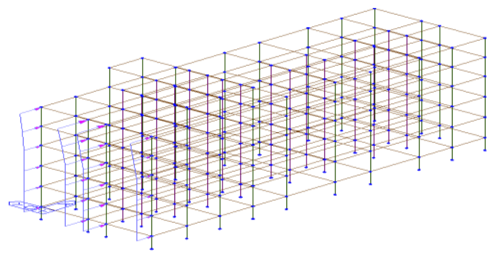


Figure 3. 15. Wind load in the X- direction

The wind in the Y- direction is set on columns, as shown in figure 3.16

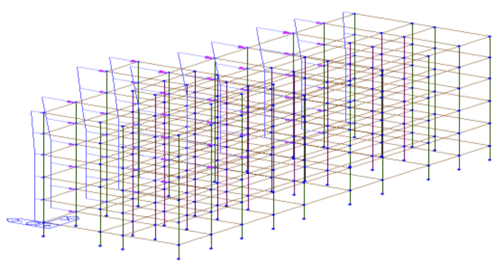


Figure 3. 16. Wind load in the Y- direction

3.5.3.3.Load combinations

The load combinations at ultimate limit state (ULS) to obtain critical solicitations for the principal beams using the imposed live load on the floor as the leading load and imposed load on the roof and wind load as the accompanied load are:

- Comb 1: 1.3(SW1+SW2+ SW3+ SW4+ SW5) + 1.5 LLF1 +1.05LLR1 + 0.9 Win Y;
- Comb 2: 1.3(SW1+SW2+ SW3+ SW4+ SW5) + 1.5 LLF2 +1.05LLR2 + 0.9 Win Y;
- Comb 3: 1.3(SW1+SW2+ SW3+ SW4+ SW5) + 1.5 LLF3 +1.05LLR3 + 0.9 Win Y;
- Comb 4: 1.3(SW1+SW2+ SW3+ SW4+ SW5) + 1.5 LLF4+1.05LLR4 + 0.9 Win Y;

- Comb 5: 1.3(SW1+SW2+ SW3+ SW4+ SW5) + 1.5 LLF5 +1.05LLR5 + 0.9 Win Y.

The loads' combinations at serviceability limit state (SLS) to obtain critical solicitations for the principal beams using the imposed live load on the floor as the leading load and imposed load on the roof and wind load as the accompanied load are:

- SW1+ SW2 + SW3 + SW4 + SW5 + LLF1 +0.7LLR1 + 0.6 Win Y;
- SW1+ SW2 + SW3 + SW4 + SW5 + LLF2 + 0.7LLR2 + 0.6 Win Y;
- SW1+ SW2 + SW3 + SW4 + SW5 + LLF3 + 0.7LLR3 + 0.6 Win Y;
- SW1+ SW2 + SW3 + SW4 + SW5 + LLF4+ 0.7 LLR4 + 0.6 Win Y;
- SW1+ SW2 + SW3 + SW4 + SW5 + LLF5 + 0.7 LLR5 + 0.6 Win Y.

The quasi-permanent combinations defined at the SLS to obtain critical solicitations for the principal beams are:

- SLS (QP) 1: SW1+ SW2 + SW3 + SW4 + SW5 + 0.3LLF1;
- SLS (QP) 2: SW1+ SW2 + SW3 + SW4 + SW5 + 0.3 LLF2;
- SLS (QP) 3: SW1+ SW2 + SW3 + SW4 + SW5 + 0.3 LLF3;
- SLS (QP) 4: SW1+ SW2 + SW3 + SW4 + SW5 + 0.3LLF4;
- SLS (QP) 5: SW1+ SW2 + SW3 + SW4 + SW5 + 0.3 LLF5.

3.5.4. Solicitations

After the definition of load combinations inside the software Midas Gen, structural analysis of the building is performed so internal forces such as bending moments, axial forces and shear forces are available for each structural element. The envelope curve of bending moment on frame axis 8 is given in figure3.17.

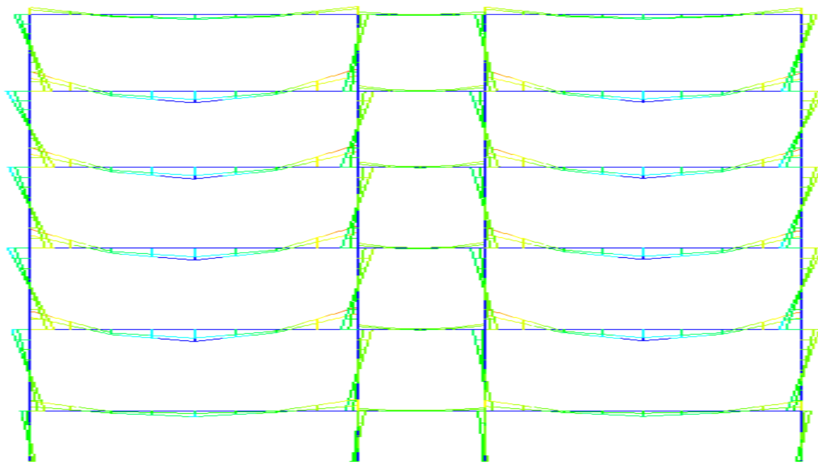


Figure 3. 17. Envelope curve for bending moments on frame axis 8

The envelope curve for axial force on frame axis 8 is given in figure 3.19 and the envelope curve for shear force on frame axis 8 is given in figure 318.

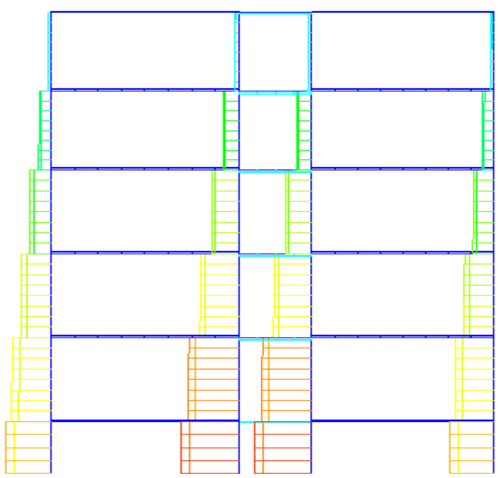


Figure 3. 18. Envelope curve for axial force on frame axis 8

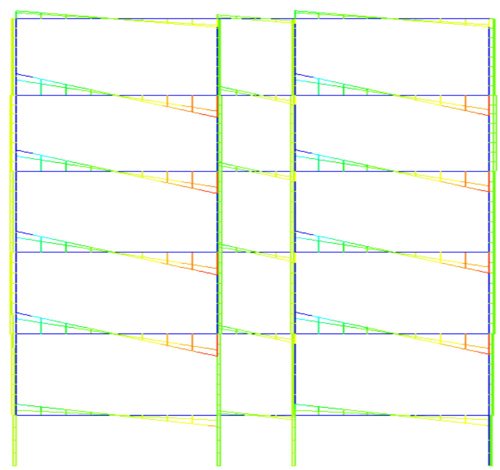


Figure 3. 19. Envelope curve for shear force on frame axis 8

3.5.5. Modal analysis

Assuming a percentage of 30 % mass participation of imposed load, the first three mode of vibration computed from Eigen value analysis are illustrated in figure 3.20, figure 3.21 and figure 3.22.

The first mode of vibration with a period T_1=1.15 seconds, it correspond to a pure vibration in the X direction (81.3444% mass participation)

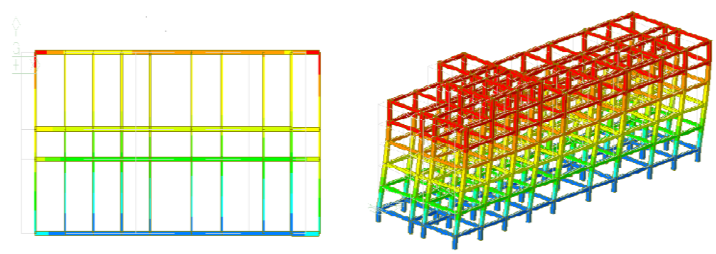


Figure 3. 20. The first mode of vibration

The second mode of vibration with a period T_2=0.90 seconds correspond to vibration primarily in the Y direction (72.9329% mass participation) and a small rotation (6.5711% mass participation)

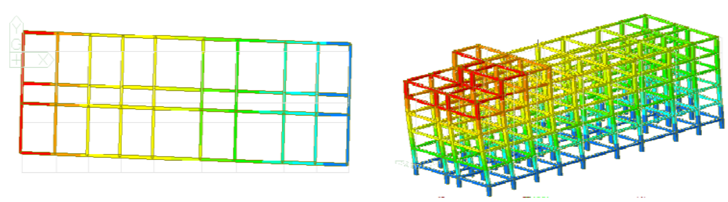


Figure 3. 21. The second mode of vibration

The third mode of vibration with a period T_3=0.85 seconds correspond to vibration with predominate torsion (73.0570% mass participation) associated with a small translation in the Y direction (6.5933% mass participation)

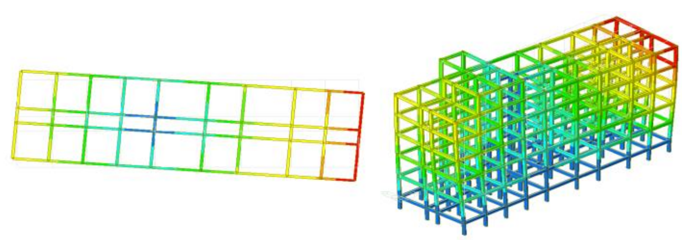


Figure 3. 22. The third mode of vibration: torsion

It can be noticed that T_1>T_2> T_3> T = $CH^{3/4} = 0.44$

3.6. Design of the structural elements

The design of structural elements consists of designing beams, columns, slabs, staircases and foundations.

3.6.1. Structural design of beams

The principal beam located on axis 8 as illustrated in figure 3.23 is the one considered

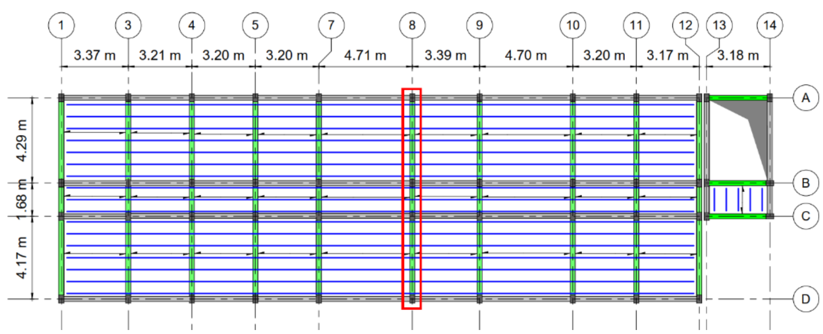


Figure 3. 23. Principal beam on axis 8 of the structural framing plan

3.6.1.1. Design of beam at ultimate limit state

The design of a beam at the ultimate limit state consists of providing and adequate longitudinal and transversal reinforcement and also provide adequate detailing disposition of rebars.

a. Design of longitudinal reinforcement

Considering the principal beam located on axis 8, the envelope curve for bending moment is given respectively in figure 3.24.

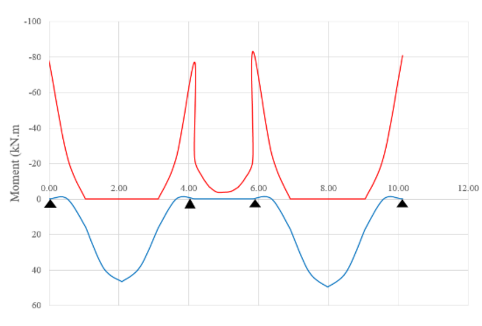


Figure 3. 24. Envelope for bending moment on the principal beam

The maximum bending moment occurs at support with a value of -82.1kN.m, the amount of steel reinforcement required is computed then the resisting moment is also computed. The results are summarized in table 3.8

Table 3. 8. Design of steel reinforcement for the maximum negative bending moment

Designation	Symbols	Values
Ultimate moment	M_{Ed}	82.1 kN.m
Normalized moment	K	0.1
Verification	$K < K_{bal} = 0.167$	yes, no compressive is reinforcement required
Area of reinforcement required	$A_{sl,req}$	638.73 mm^2
Number of bars	n	2⌀14 + 2⌀16
Spacing between bars	S_b	50 mm
Area of reinforcement provided	$A_{sl,prov}$	710 mm^2
0Ultimate resisting moment	M_{RD}	89.67 kN.m
Verification	$M_{RD} \geq M_{Ed}$	Yes, the design is satisfactory

The same procedure has been done for other critical points on spans of the principal beam, the result is summarized in figure 3.25.

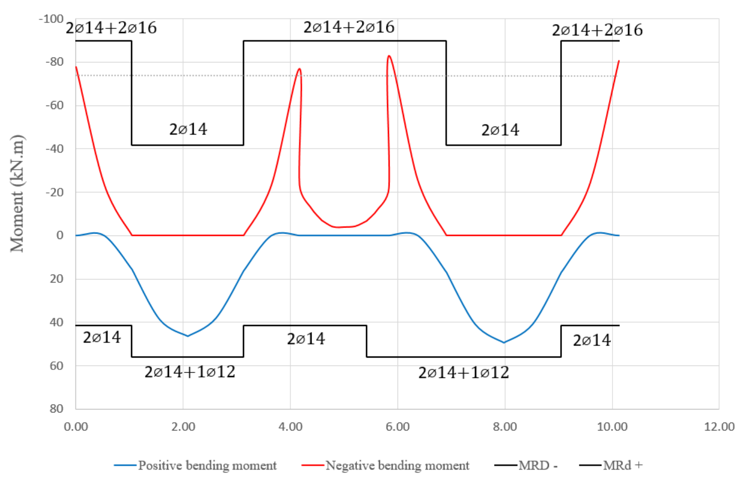


Figure 3. 25. Envelope curve for resisting moments on the principal beam

b. Design of transversal reinforcement

Considering the beam located on axis 8, the envelope curve for shear force is given in figure 3.26.

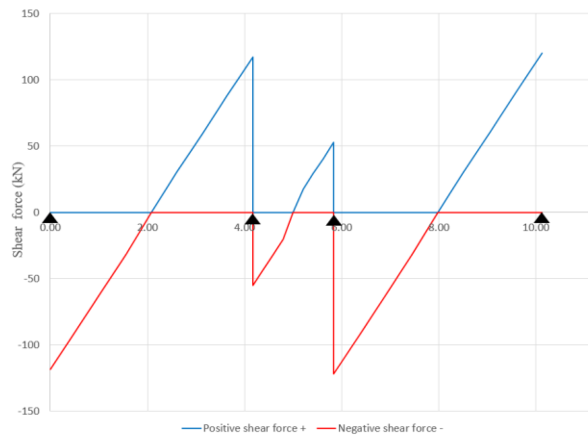


Figure 3. 26. Envelope curve for shear forces on the principal beam

The maximum shear force is 120.5kN the result of the shear design is summarized in table 3.9

Table 3. 9. The result of the shear design for maximum shear force on beam in axis 8

Designation	Symbols	Values
Shear force	V_{Ed}	120.5 kN
Shear resistance without shear reinforcement	V_{rdc}	51.70 kN
Verification	$V_{rdc} \leq V_{Ed}$	shear reinforcement is required
The minimum amount of reinforcement	$\left(\frac{A_{sw}}{s}\right)_{min}$	0.42 mm^2/m
Design amount of reinforcement	$\left(\frac{A_{sw}}{s}\right)$	0.71mm^2/m
Arrangement of stirrups	⌀6 with the spacing of 70 mm	
maximum spacing	S_{max}	279 mm
maximum shear resistance	V_{Rd}	123.48 kN
Verification	$V_{Rd} \geq V_{Ed}$	Yes, shear is verified

The same procedure has been done for other critical points on spans of the principal beam, the result is summarized in figure 3.27

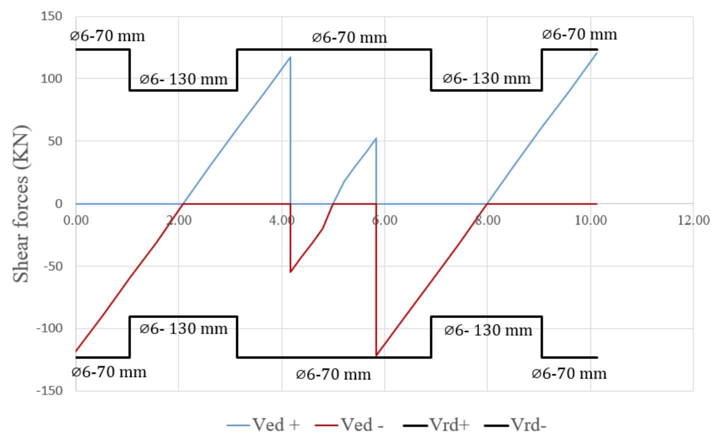


Figure 3. 27. Envelope curve for resisting shear forces on the principal beam

c. Detailing design

The mandrel diameter ($\varnothing_m$) and the design anchorage length (l_{bd}) of bars of 12mm, 14 mm and 16mm are presented in table 3.10.

Table 3. 10. Values of mandrel diameters and design anchorage length

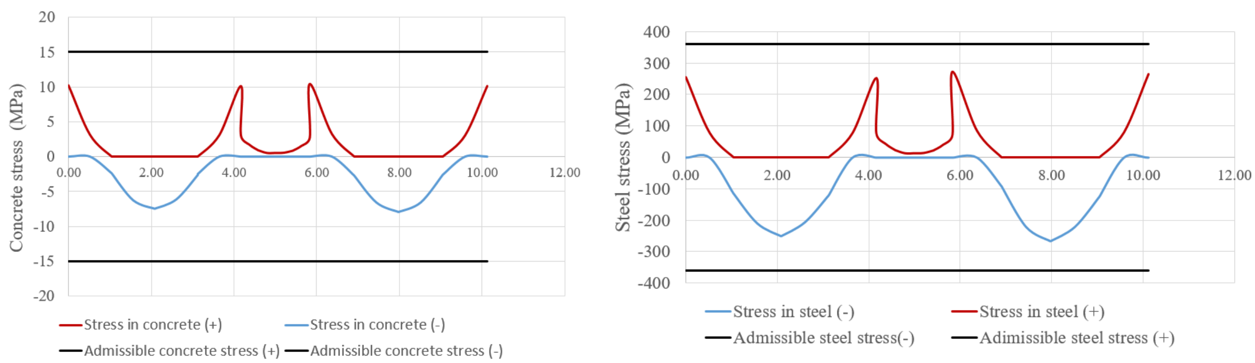
Designation	**symbols**	**Bar diameters**		
		`16 mm	14 mm	12 mm
Mandrel diameter	$\varnothing_m$	64mm	56 mm	48 mm
Design anchorage length	l_{bd}	600 mm	550 mm	450 mm

3.6.1.2. Structural design at serviceability limit state

The design at serviceability limit state is governed by the verification of stress limitation, crack control and deflection.

a. Verification of stress limitation

The bending moments obtained from the characteristic load combinations allows to evaluate the stress in concrete and steel using repectively equations (2.39) and (2.40). The admissible stress in concrete and steel are computed using repectively equations (2.33) and (2. 35). The stress inside the concrete and steel reinforcement is less than the admissible stress inside concrete and steel as illsustrated respectively in case a) and b) of figure 3.28.



a) Stress verification in concrete

b) stress verification in steel

Figure 3. 28. Stress verification inside the concrete and steel reinforcement

b. Crack control

The result of cracking verification is presented in table 3.11

Table 3. 11. The result of cracking verification

Quasi-permanent combinations	For maximum positive bending moment	For the maximum negative bending moment
Exposure class XC 1	W_k = 0.4 mm	W_k = 0.4 mm
Steel stress (Mpa)	215.48Mpa	259.90 Mpa
Maximum bar diameter	20 mm	20 mm
Maximum spacing between	250 mm	250 mm

The maximum bar diameter provided for the maximum positive moment is 14 mm < 20 mm and the maximum spacing provided between bars is 80 mm < 250 mm, hence cracking is verified for the maximum positive moment. The maximum bar diameter provided for the maximum negative moment is 16 mm ≤ 20 mm and the maximum spacing provided between bars is 50 mm < 250 mm, hence cracking is verified for the maximum negative moment.

c. Deflection

The verification of deflection is made on the maximum span length of the beam which is 4.29 m, the result of the verification of the deflection is presented in table 3.12

Table 3. 12. The result of the verification of the deflection

Designation	Symbols	Values
Percentage of reinforcement	ρ	0.36%
Reference reinforcement ratio	ρ_0	0.5%
The factor of structural system	k	1.3
Correction factor	$500\ A_{s,prov}/f_{yk}A_{s,req} \leq 1.5$	1.29
Allowable limit span/depth	$(l/d)_{allow}$	42.39
Actual limit span /depth	$(l/d)_{actual}$	11.85
Verification	$(l/d)_{actual} < (l/d)_{allow}$	Yes, deflection is verified

3.5.1. Design of ribbed slab

The floor is a one-way ribbed slab with ribs running along the longitudinal direction of the structure on nine spans as illustrated in figure 3.29.

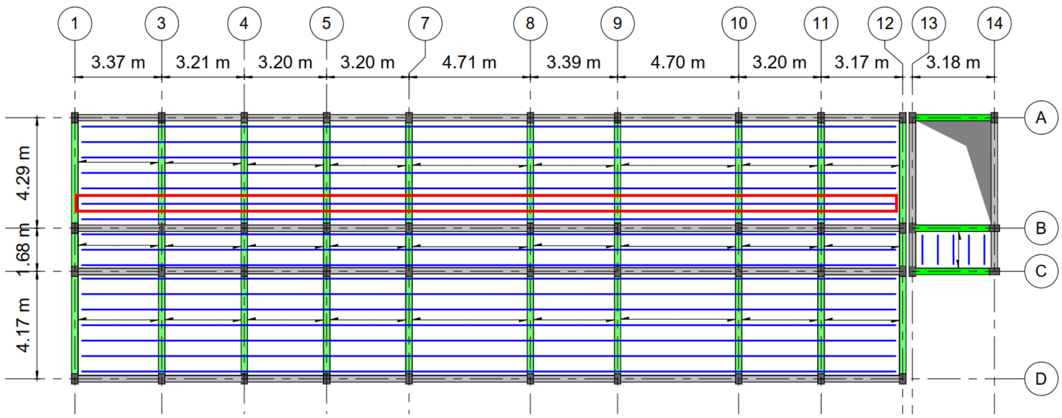


Figure 3. 29. Longitudinal ribs on the ribbed slab floor

3.5.1.1.Loads on ribs

The loads on ribs are permanent loads and variable loads, the table 3.13 present the different loads on ribs.

Table 3. 13. The different loads on ribs.

Nature	Symbol	Loads in kN/m²	Loads in kN/m
Self-weight of the slab	G1	2.85	2.85x 0.6 = 1.71
Nonstructural permanent	G2	1.1	1.1x0.6=0.66
Self-weight of the partition	G3	1	1x0.6 =0.6
Total permanent load	G	4.95	4.95x0.6 =2.97
Variable load	Q	2.5	2.5x0.6 = 1.5

3.5.1.2. Loads combinations

To have the maximum bending moment and shear forces in span and at support, the different combinations used are displayed in figure 3.30 and figure 3.31.

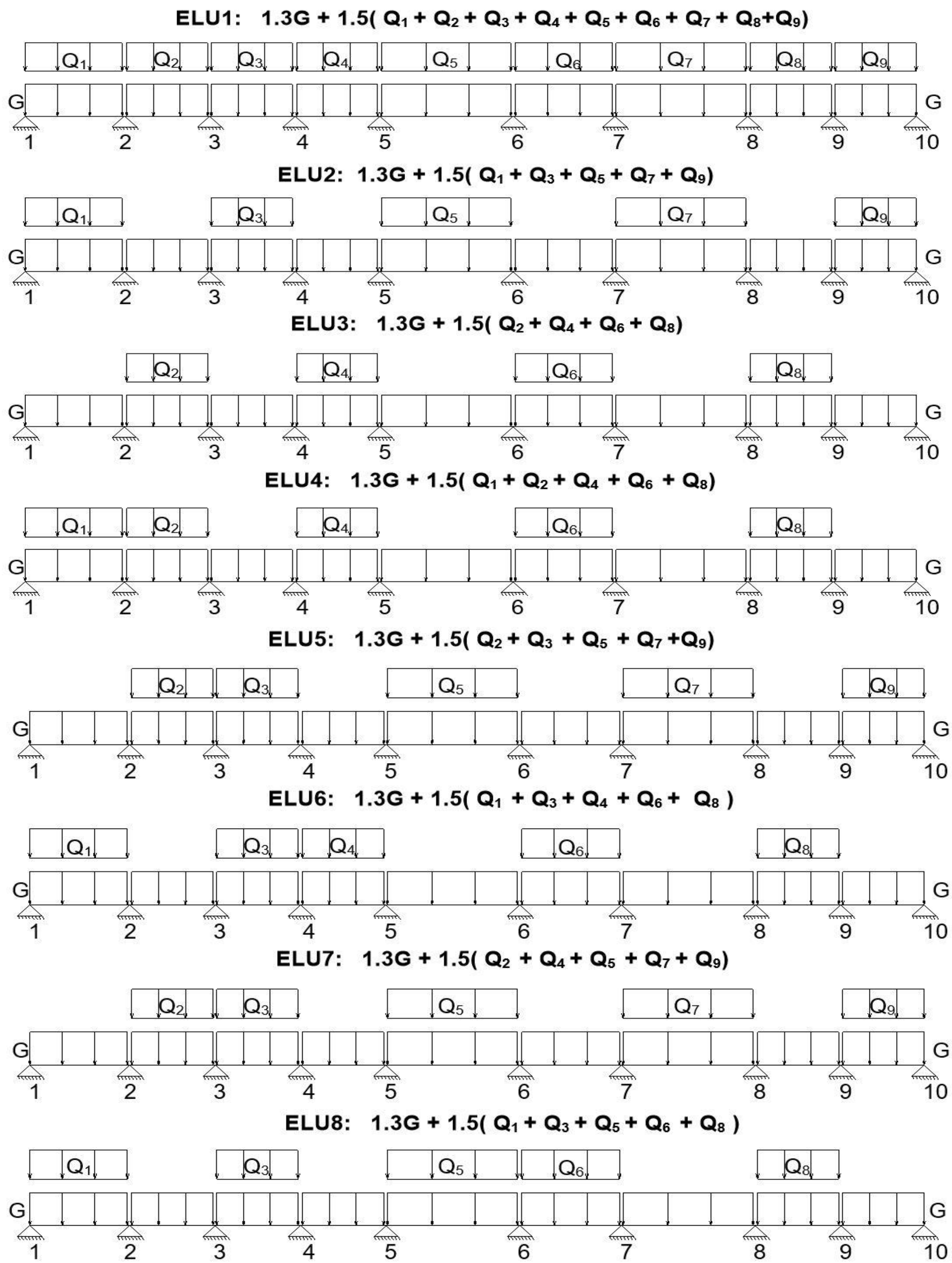


Figure 3. 30. The first eight load combinations on the rib

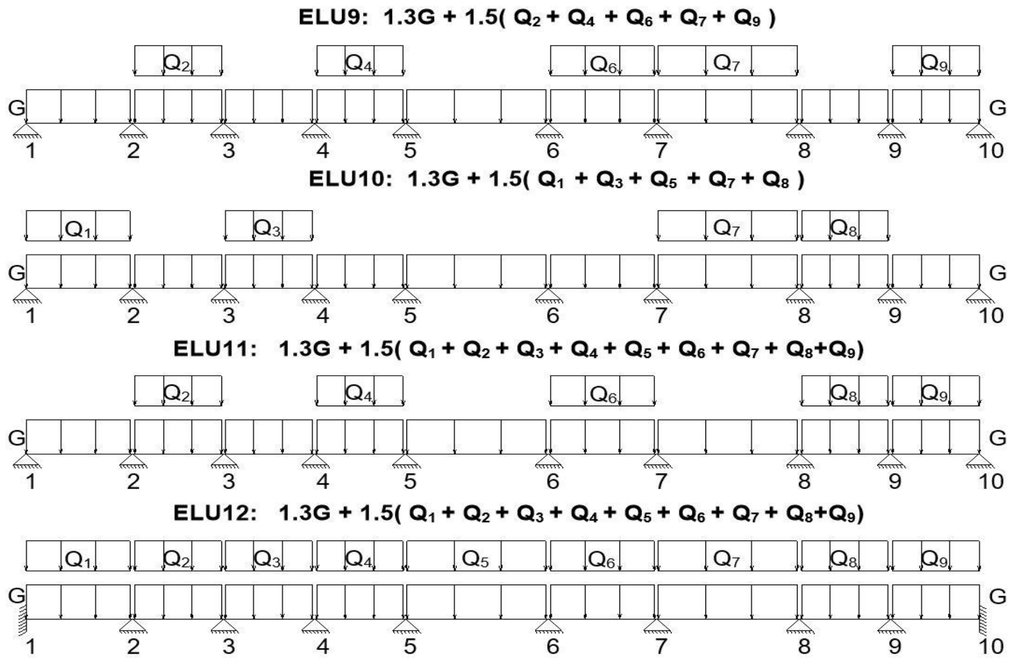


Figure 3. 31. The last fourth load combinations on the rib

3.5.1.3. Solicitations

The envelope curve for the bending moment and shear diagram at the ultimate limit state is presented in figure 3.32 and 3.33 respectively

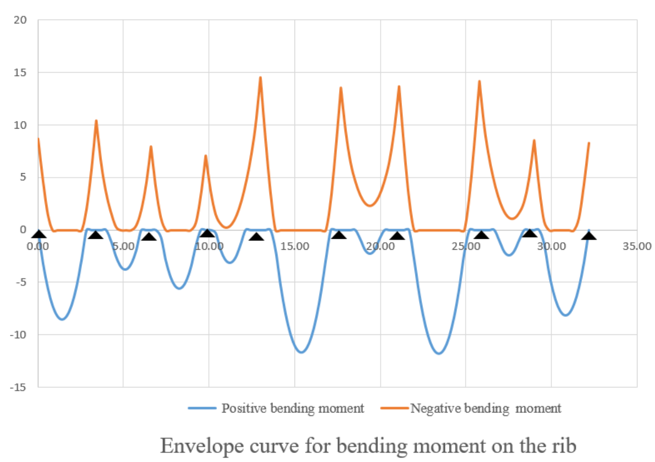


Figure 3. 32. Envelope curve for bending moments

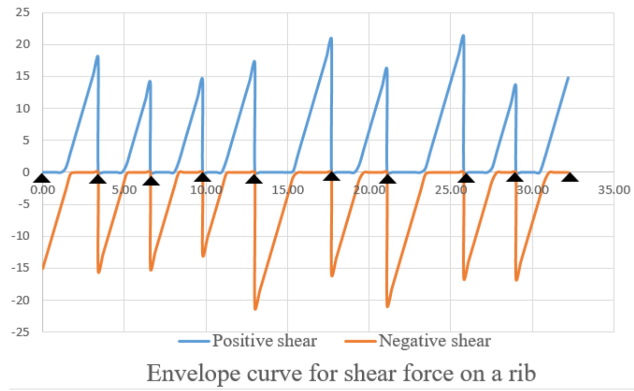


Figure 3. 33. Envelope curve for shear forces

3.5.1.4. Structural design at the ultimate limit state

a. Design of bending moment

The design of steel reinforcement and the resisting moments necessary to resist the envelope curve of the acting bending moments is obtained using equations (2.69) and (2.71). Figure 3.34 present the recapitulative of the longitudinal reinforcement along the rib.

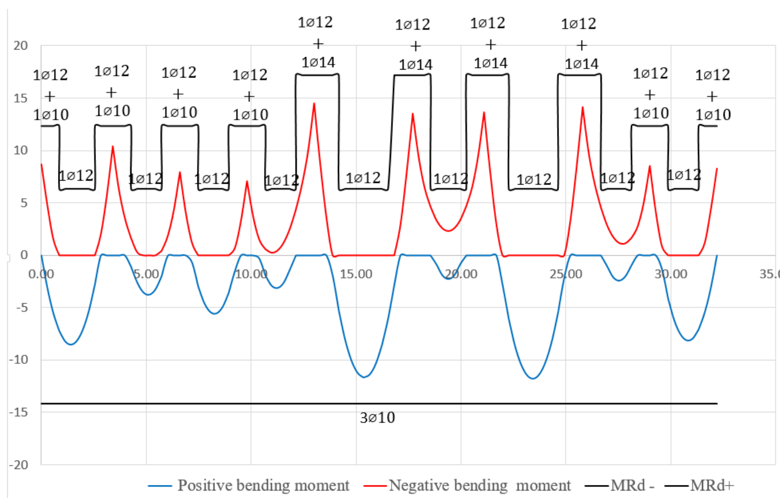


Figure 3. 34. Envelope curve of the resisting moment along the rib

b. Design of shear

The design of shear for the maximum shear force ($V_{Ed,max}$)is summarized in table 3.14.

Table 3. 14. The design of shear for the maximum shear force

$V_{Ed,max}$	ρ_l	V_{RdC}	(A_{sw}/S)	Bar diameter	Spacing
21 kN	0.0178	13.78 kN	0.121 mm²/m	6 mm	200mm

The resisting shear force and the stirrups spacing along the rib is presented in figure 3.35

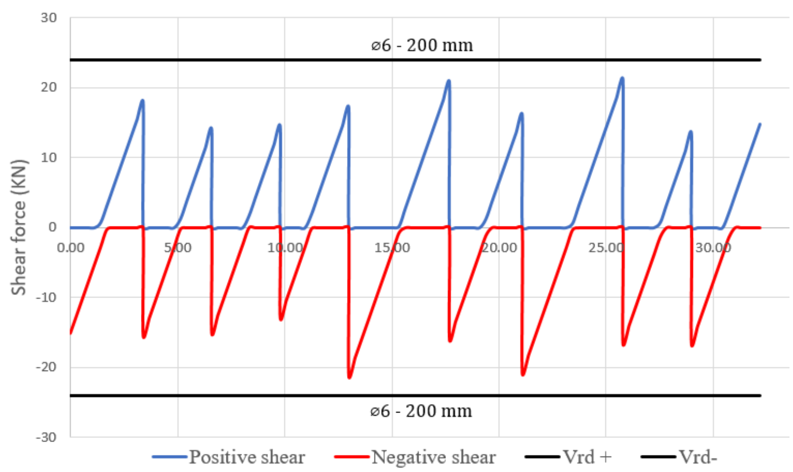


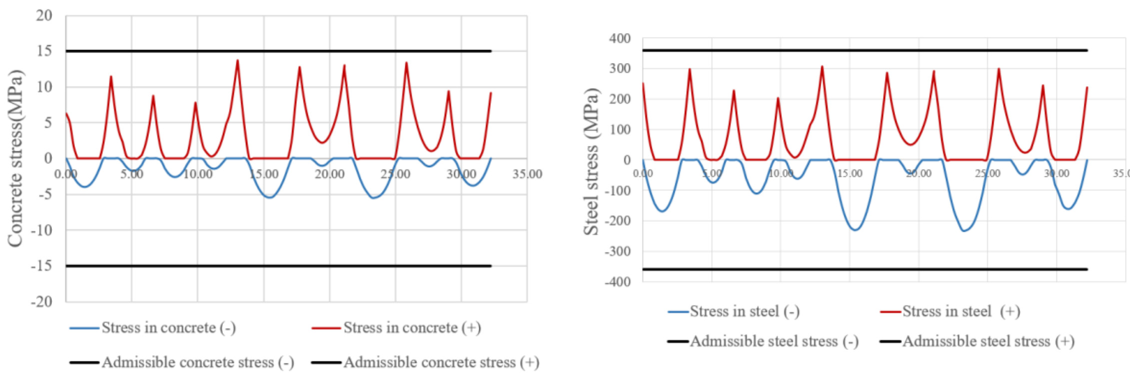
Figure 3. 35. Resisting shear force along the rib

3.5.1.5. Structural design at serviceability limit state

The design at serviceability limit state is governed by the verification of stress limitation, crack control and deflection.

a. Verification of stress limitation

The bending moments obtained from the characteristic load combinations allows to evaluate the stress in concrete and steel using repectively equations (2.39) and (2.40). The admissible stress in concrete and steel are computed using repectively equations (2.33) and (2. 35). The stress inside the concrete and steel reinforcement is less than the admissible stress inside concrete and steel as illsustrated respectively in case a) and b) of figure 3.36.



a) Stress verification in concrete

b) Stress verification in steel

Figure 3. 36. Stress verification for concrete and steel

b. Crack control

Assuming and exposure class XC1, the maximum admissible crack width is W_k = 0.4 mm under quasi load combination. For the steel stress 193 Mpa, the maximum allowable bar diameter is 32 mm which is greater than the 10 mm bar diameter used, so the crack is verified.

c. Deflection

A summary of the verification of the deflection is provided in table 3.15.

Table 3. 15. A summary of the verification of the deflection

Designation	**Symbols**	**Values**
Amount of reinforcement at Bottom	ρ	0.64%
Reference reinforcement ratio	ρ_0	0.5%
Amount of reinforcement at the top	ρ'	0.31%
The factor of structural system	k	1.5
Correction factor	$500\ A_{s,prov}/f_{yk}A_{s,req} \leq 1.5$	1.29
Allowable limit span/depth	$(l/d)_{allow}$	43.96
Actual limit span /depth	$(l/d)_{actual}$	22.38
Verification	$(l/d)_{actual} < (l/d)_{allow}$	Yes,deflection is verified

3.5.2. Design of columns

The columns on the ground floor selected to design are B8 and A8 which are respectively the most loaded interior column and the most loaded exterior column.

3.5.2.1.Solicitations

For column B8, the maximum axial force is 989 kN and the moment at the top and bottom are respectively 14.5kN.m and -2.3 kN.m in-plane ZX.

For column A8, the maximum axial force is 806.5 kN and the moment at the top and bottom are respectively 26.4kN.m and -11.6 kN.m in-plane ZX.

3.5.2.2. Effective length and slenderness of columns

The result for calculation of the effective length of columns and the slenderness limit is provided respectively in table 3.16

Table 3. 16. Effective length and Slenderness of columns

Designation	Column B8		Column A8	
	XZ Plane	YZ plane	XZ Plane	YZ plane
Relative flextibilities ($K_1 = K_2$)	0.1	0.1	0.1	0.1
Effective length L_0(m)	1.65	1.65	1.65	1.65
Slenderness (λ)	14.32	22.92	14.32	22.92

The value of the slenderness limit for columns B8 and A8 is provided in table 3.18

Table 3. 17. Slenderness limit of columns

Columns	r_m	C	n	λ_{limit}
B8	- 0.15	1.85	0.69	34.25
A8	-0.432	2.1	0.56	43.66

For column B8, $\lambda_{XZ} = 14.32 < \lambda_{limit} = 34.25$ and $\lambda_{YZ} = 22.92 < \lambda_{limit} = 34.25$ so the column is not slender.

For column A8, $\lambda_{XZ} = 14.32 < \lambda_{limit} = 43.66$ and $\lambda_{YZ} = 22.92 < \lambda_{limit} = 43.66$ so the column is not slender.

3.5.2.3. Design of steel reinforcement for columns

The design axial forces (N_{Ed}) and moment (M_{Ed}) for columns B8 and A8 is given in table 3.18

Table 3. 18. The design moment of columns

Columns	N_{Ed}(kN)	M_{01} (kN.m)	M_{02} (kN.m)	e_i(mm)	e_0(mm)	M_{Ed} (kN.m)
B8	989	- 2.3	14.5	4.13	20	18.59
A8	806.5	-11.6	26.4	4.13	20	29.73

Using the design chart $\frac{d'}{h} = 0.1$, the area of steel reinforcement provided is in between $A_{s,min}$ and $A_{s,max}$ the summary of the design of steel reinforcement is available in table 3.19 for column B8 and column A8.

Table 3. 19. Steel reinforcement for columns B8 and A8

Columns	N_{Ed} (kN)	M_{Ed} (kN m)	$N_{Ed}/(bhf_{ck})$	$M_{Ed}/(b^2hf_{ck})$	$A_{s,prov}$ (mm^2)	$A_{s,min}$ (mm^2)	$A_{s,max}$ (mm^2)
B8	989	18.59	0.70	0.032	923.62	252	4000
A8	806.5	29.73	0.57	0.052	923.62	252	4000

Using M-N interaction diagram. Assuming a symmetrical reinforcement, 6⌀14 are provided in the section. The M – N interaction diagram presented in figure 3.37 shown that the design axial force and the design moment for both column B8 and A8 are lying inside the M – N interaction diagram, hence the steel reinforcement provided 6⌀14 is good.

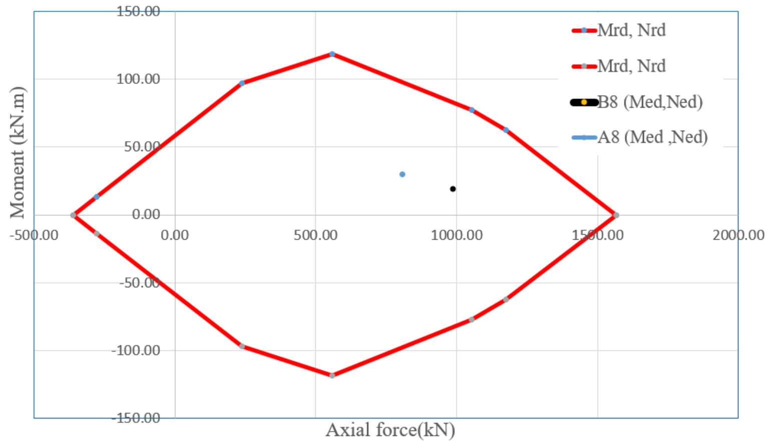


Figure 3. 37. M – N interaction diagram

The summary of the design of shear under columns B8 and A8 are presented in table3.20.

Table 3. 20. Shear verification on the columns

Columns	$\mathbf{V_{Ed}}$**(kN)**	$\mathbf{V_{rdc}}$**(kN)**	$\mathbf{V_{rdc} \geq V_{Ed}}$
B8	10.2	51.7	Yes, shear reinforcement to resist shear force is not required
A8	19.6	51.7	Yes, shear reinforcement to resist shear force is not required

The diameter of stirrups $\varnothing_{\text{link}} = \max\left(6\text{mm}, \frac{1}{4}\varnothing_{\text{long}}\right) = 6$ mm

The maximum spacing between stirrups $S_{\max} = \min(\ 20\varnothing_{\text{long}},\ 250,\ 400\ mm)$ =280 mm

3.5.3. Structural design of the staircase

3.5.3.1. Preliminary design of the staircase

The structural design of the staircase is for the one located in part A of the building as illustrated in figure 3.38. The stair has a riser height (R) of 17 cm and a going (G) of 30 cm.

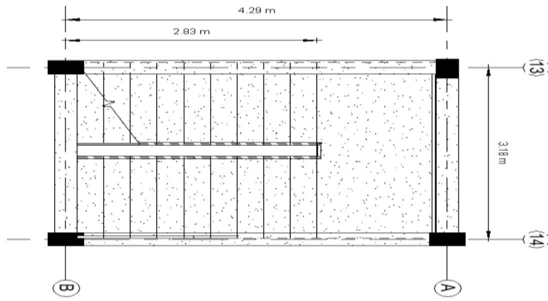


Figure 3. 38. 2D view of the staircase

The effective span length is 4.3m, a suitable value of waist h is 170 mm and the average thickness t is 281 mm.

3.4.6.2. Design of staircase

a. Structural analysis

The loads on flight and landing are permanent loads and live loads, a summary of loads and load combinations acting on the flight and landing is provided in table 3.21.

Table 3. 21. Summary of loads acting on the flight and landing

Slab self-weight ($g_{1,\text{flight}}$) on flight	$g_{1,\text{flight}} = 25 * t = 7.02\ kN/m^2$
Slab self-weight ($g_{1,\text{landing}}$) on landing	$g_{1,\text{landing}} = 25 * h = 4.25\ kN/m^2$
Non-structural permanent load (g_2)	$g_2 = 1kN/m^2$
Total permanent load (G_{flight})	$G_{flight} = 1m * (1 + 7.02)kN/m^2 = 8.02kN/m$
Total permanent load (G_{flight})	$G_{landing} = 1m * (1 + 4.25)kN/m^2 = 5.25kN/m$
Variable load on landing and flight (Q)	$Q = 1m * 3\ kN/m^2 = 3kN/m$
Load combination at ULS	$1.3(G_{flight}+G_{landing}) + 1.5Q$
Load combination at SLS	$(G_{flight}+G_{landing}) + Q$

Bending moments and shear forces are determined using the static schemes presented in figure 3.39 where case (a) provide maximum bending moment in span and case (b) provide maximum bending moment at supports.

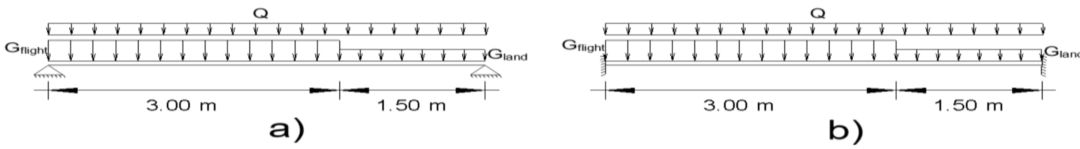


Figure 3. 39. Static scheme for structural analysis of the staircase

The bending moment diagram for static scheme (a) and the bending moment diagram for static scheme (b) are presented respectively in figure 3.40 and figure 3.41.

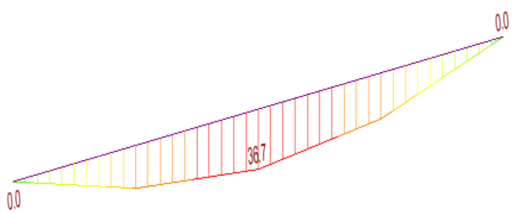


Figure 3.40. Maximum bending moment in span

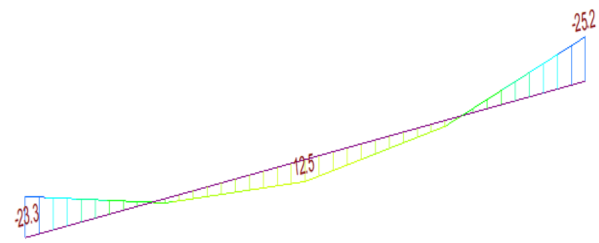


Figure 3.41. Maximum bending moment at supports

b. Structural design at ultimate limit state

The structural design consist of performing the design for bending moment and shear forces

i. Design of flexural reinforcement

The summary of the design of the maximum bending moment on the staircase is provided in table 3.22.

Table 3. 22. The summary of the design of steel reinforcement for the staircase

Designation	Symbols	Values
Width	b	1000 mm
Height	h	170 mm
Effective depth	d	139 mm
Ultimate moment	M_{Ed}	36.8 kN.m
Normalized moment	K	0.075

Verification	$K < K_{bal} = 0.167$	yes, no compressive reinforcement required
Reinforcement area required	$A_{sl,req}$	724.25 mm^2/m
Arrangement of bars	(7⌀12-125 mm)/m	
Reinforcement area provided	$A_{sl,prov}$	791.68 mm^2/m
Design of secondary reinforcement (transversal reinforcement)		
Reinforcement area required	$A_{st,req} = 20\% A_{sl,req}$	144.8 mm^2/m
Arrangement of bars Per	Transversal bar (⌀8-325mm)/m	
Reinforcement area provided	$A_{st,prov}$	150.78 mm^2/m

The maximum bending moment at support equal to 25.2kN.m, the area of reinforcement adopted is 549.7 mm^2/m and (7⌀10 -140 mm)/m is provided.

ii. Design of shear reinforcement

The summary of the design of shear reinforcement is presented in table 3.23.

Table 3. 23. The summary of the design of shear reinforcement on the staircase

Designation	**Symbols**	**Values**
Shear force	V_{Ed}	34 kN
Shear resistance without shear	V_{rdc}	84.53 kN
Verification	$V_{rdc} \geq V_{Ed}$	shear reinforcement is not required

c. Structural design at serviceability limit state

The structural design at serviceability limit state consists of performing the verification of deflection and cracking.

i. Deflection

The summary on the verification of the deflection is presented in table 3.24.

Table 3. 24. Verification of the deflection for the staircase

Designation	Symbols	Values
Percentage of reinforcement	ρ	0.43 %
Reference reinforcement ratio	ρ_0	0.5 %
structural system factor	k	1.3
Correction factor	$500\ A_{s,prov}/f_{yk}A_{s,req} \leq 1.5$	1.21
Allowable limit span/depth	$(l/d)_{allow}$	33.09
Actual limit span /depth	$(l/d)_{actual}$	32.37
Verification	$(l/d)_{actual} < (l/d)_{allow}$	Yes, deflection is verified

ii. Cracking

The slab thickness (h=170mm) of the stair is less than 200mm, the verification of cracking is performed by verifying the spacing between bars as recommended by code for a concrete slab. Table 3.25 present the result of cracking verification

Table 3. 25. The result of cracking verification for staircase

Main bar	Max allowable bar spacing	$S_{max,slab} = \min(3h, 400mm)$	400 mm
	Max actual bar spacing	$S_{long,max}$	150 mm
	Verification	$S_{long,max} \leq S_{max,slab}$	OK
Secondary bar	Max allowable bar spacing	$S_{max,slab} = \min(3.5h, 450\ mm)$	450 mm
	Max actual bar spacing	$S_{trans,max}$	325 mm
	Verification	$S_{trans,max} \leq S_{max,slab}$	Ok
Cracking is verified			

3.5.4. Structural design of foundations

The admissible soil constraint is 0.3 N/mm^2, shallow foundations consisting of pad footing, combined footing and strap footing are used. Figure 3.42 present the structural plan of the foundation.

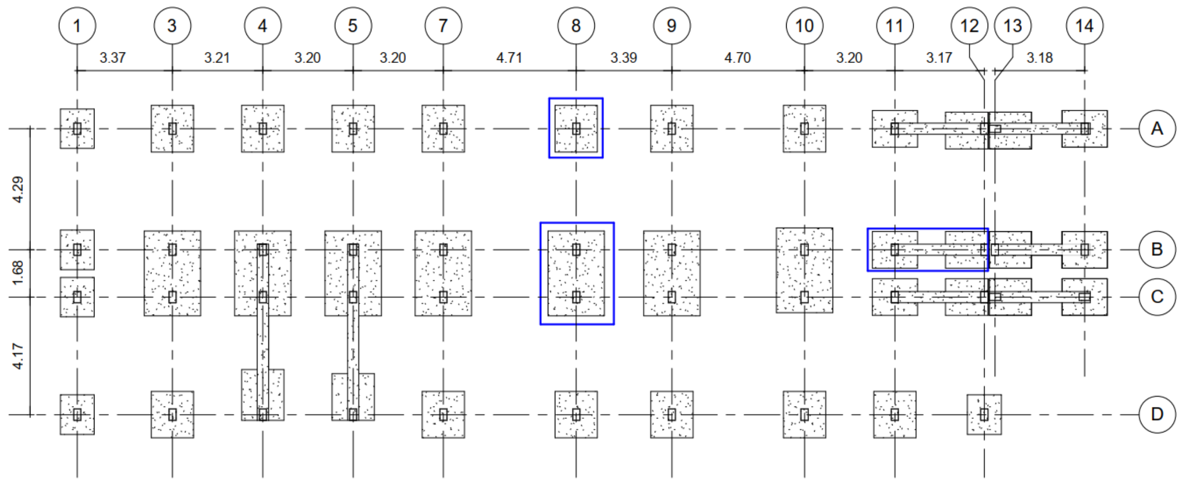


Figure 3.42. The structural plan of foundation.

3.6.5.1. Structural design of pad footing

The most loaded exterior pad footing is at bottom of column A8, the loading at SLS are $N_{sls} = 690.3$ KN and at ULS, $N_{uls} = 926.1$ kN, $M_{y,uls} = 0.8$ kN.m. Because the moment is negligible, pad footing is considered to be subjected to axial force only and there is a constant distribution of pressure at the base.

a. Geometrical size of the pad footing

The result of the preliminary design to obtain the geometrical size (B, C and H) of the pad footing (F1) under column A8is presented in table 3.26.

Table 3. 26. Summary of the preliminary design of pad footing

Designation	Symbols	Values
Admissible pressure	σ_{adm}	0.3 Mpa
Axial force at SLS	N_{sls}	690.3 kN
Column width	b	200 mm
Column length	c	400 mm
Overhand distance from the column	d_0	620 mm
Height of the footing	H	400 mm

Width of the footing	B	1500 mm
Length of the footing	C	1650 mm
Net pressure under the pad	σ_{net}	0.28 Mpa
Verification	$\sigma_{adm} \geq \sigma_{net}$	Yes, the section is adequate

The same procedure has been done for other pad footings, table 3.27 provide pad footing size dimensions under columns.

Table 3. 27. Pad footing size dimensions under columns

Columns	**Design axial load at SLS**	**Size of footings**			**Pad footing family**
		B	**C**	**H**	
D12, D11,D10,D9,D8,D3 A12,A11,A10,A9,A8,A6,A4,A3	690.3 kN	1.5m	1.65m	0.4m	F1
A1,B1,C1,D12,	486 kN	1.2m	1.4m	0.4m	F2

b. Verification of maximum punching shear resistance

The maximum punching resistance for footing under column B8 is $V_{Rd,max}$ = 1806.07 kN $\geq N_{uls}$ = 926.1 kN, hence punching shear resistance at column perimeter is verified.

c. Design of steel reinforcement

The design of steel reinforcement inside the footing under column A8 illustrated in figure 3.43 is summarized in table 3.28. The pressure at the base of the footing pressure is 0.37 N/mm^2 .

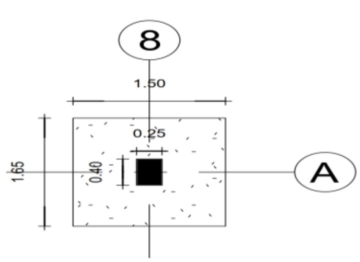


Figure 3. 43. 2D view of the footing

Table 3. 28. Design of steel reinforcement in both x and y direction of the footing F1

Design of steel reinforcement parallel to the width of the footing (x-direction)								
$M_{Ed,y}$	d (mm)	k	k_{bal}	Z (mm)	A_s(mm^2)	Bar	Number of bar	Spacing
120.58	363	0.022	0.167	344.85	893.17	14	7	250
Design of steel reinforcement parallel to the length of the footing (y-direction)								
$M_{Ed,x}$	d (mm)	k	k_{bal}	Z (mm)	A_s(mm^2)	Bar	Number of bar	Spacing
109.62	349	0.020	0.167	344.85	811.97	14	7	200

d. Verification on shear resistance

A summary of the results for the verification of shear resistance at distance d from column face and also the verification of punching shear resistance at distance 2d from column face is presented in table3.29 And table 3.30 respectively.

Table 3. 29. Verification of shear resistance at distance d from the column face

Planes considered		**XZ plane**	**YZ plane**
Pressure at distance d from column	P (Mpa)	0.37	0.37
Design shear force	V_{Ed}(kN)	214.95	188.70
Shear force resistance	V_{rdc}(kN)	236.1	219.54
Verification	V_{rdc}(kN) $\geq V_{Ed}$(kN)	Shear is verified	

Table 3. 30. Verification of punching shear resistance at distance 2d from the column face

Designation	**Symbols**	**Values**
Pressure at distance 2d from the	P	0.37 MPa
Punching shear force	V_{Ed}	513.54 kN
Punching shear resistance	V_{rdc}	859.75 kN
Verification	V_{rdc}(kN) $\geq V_{Ed}$(kN)	Punching shear is verified

3.6.5.2. Structural design of combined footing

Base on the preliminary design, isolated footings under columns B9, C9, B8, C8, B10, C10, B11and C11 are not adequate due to the overlapping areas of the isolated footings, so combined footing is provided under adjacent columns.

a. Geometrical sizes of combined footing

A preliminary design of the combined footing under the most loaded adjacent columns (B8, C8) is. The plan size of the combined footing is 2m width and 3 m length as illustrated in figure3.44.

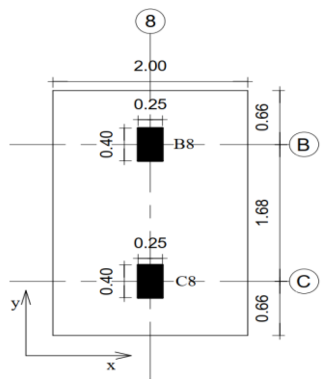


Figure 3.44. The plan size of the combined footing

b. Verification of punching shear resistance

Assuming a height of 40 cm, the summary for the verification of punching shear at column perimeter B8 and C8 is provided in table 3.31.

Table 3. 31. Verification of punching shear resistance at column perimeter

Columns	N_{ED}	$V_{Rd,max}$	$N_{ED} \leq V_{Rd,max}$
B8	1136.2 kN	1806.1 kN	Yes, punching shear is verified
C8	1114.1 KN	1806.1 kN	Yes, punching shear is verified

c. Design of steel reinforcement

The design of steel reinforcement in the y-direction is done using bending moment at the ultimate limit state obtained from the structural analysis of a beam with a length of 3m loaded with an upward pressure $w_u = B\sigma_u = 749.63$ kN/m as presented in figure 3.45.

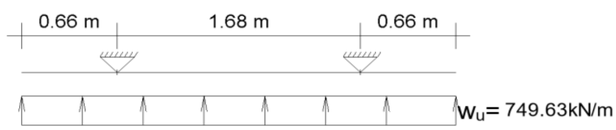


Figure 3.45. Static of the scheme the combined footing in the Y direction

The longitudinal bending moment and shear force diagram provided in figure3.46 and figure3.47 obtained after performing the structural analysis.

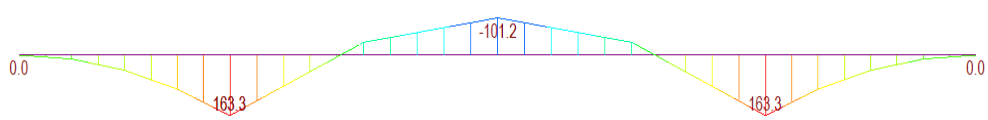


Figure 3.46. The longitudinal bending moment to design combined footing

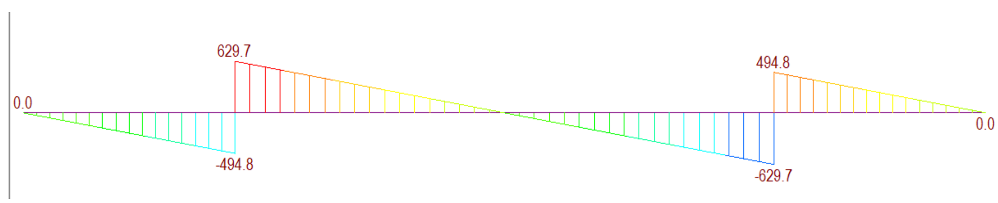


Figure 3.47. The longitudinal shear force diagram to design combined footing

The summary of the design of the longitudinal reinforcement is provided in table 3.32.

Table 3. 32. The summary of the design of the longitudinal and transversal reinforcement

Design of top steel reinforcement in the y-direction with $M_{Ed} = 101.2$kN.m						
d (mm)	k	k_{bal}	Z (mm)	A_s(mm^2)	$A_{s,min}$(mm^2)	Arrangement of bars
363	0.01	0.167	344.85	748.1	1075.91	9⌀14 - 230 mm

Design of bottom steel reinforcement in the y-direction M_{Ed} = 163.3kN.m						
d (mm)	k	k_{bal}	Z (mm)	A_s(mm^2)	$A_{s,min}$(mm^2)	Arrangement of bars
363	0.02	0.167	344.85	1209.56	1075.91	9⌀14 - 230 mm
Design of bottom steel reinforcement in the x-direction M_{Ed} = 187.56kN.m/m						
d (mm)	k	k_{bal}	Z (mm)	A_s(mm^2/	$A_{s,min}$(mm^2)	Arrangement of bars
363	0.05	0.167	344.85	1389.29	537.95 mm^2/m	(⌀16-130mm)/m
The top steel reinforcement provided in the x-direction is (⌀12-130mm)/m						

d. Design of shear

The punching shear cannot be checked because the distance 2d is outside of the base area

A summary of the results for the verification of shear resistance is provided in table 3.33.

Table 3. 33.verification of shear resistance

V_{ED} (kN)	**V_{rdc} (kN)**	**V_{rdc}> V_{ED}**
273.63	292.50	Yes, shear reinforcement is not needed

3.6.5.3. Structural design of strap footing

The strap footing considered to design is the one provided to connect the eccentrically loaded exterior pad footing under column B12 and the interior column footing under column B11 as illustrated in figure 3.48.

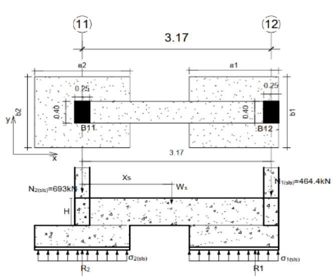


Figure 3.48. Strap footing with uniform contact pressure under each footing

a. Geometrical size of the strap footing

Assuming the distance $a_1$is equal to1500 mm, the eccentricity e is equal to 550 mm. the soils reactions and sizes of footings are provided in table 3.34.

Table 3. 34. The soils reactions and sizes of footings

Footing 1(under column B12)		Footing 2(under column B11)	
$N_{1(sls)}$	464.4 kN	$N_{2(sls)}$	693 kN
R_1	568.12kN	R_2	600.17kN
$\sigma_{1(sls)}$	378kN/m	$\sigma_{2(sls)}$	375kN/m
a_1	1.5m	a_2	1.6 m
b_1	1.3 m	b_2	1.3 m

b. Verification of punching shear resistance at column perimeter

Assuming a height of 50 cm, the summary for the verification of punching shear at column perimeter B12 and B11 is provided in table 3.35.

Table 3. 35. Verification of punching shear resistance at column perimeter

Columns	$\mathbf{N_{ED}}$	$\mathbf{V_{Rd,max}}$	$\mathbf{N_{ED} \leq V_{Rd,max}}$
B12	915kN	2303.62kN	Yes, punching shear is verified
B11	610 KN	2303.2kN	Yes, punching shear is verified

c. Design of longitudinal reinforcement

The design of longitudinal reinforcement for the strap beam and the steel reinforcement in the x-direction for footing1 and 2 is done using bending moment at ultimate limit state obtained from the structural analysis of a beam loaded with upward pressures σ_{u1} equal to 497.15 kN/m and σ_{u2} equal to 494.47 kN/m as presented in figure 3.49.

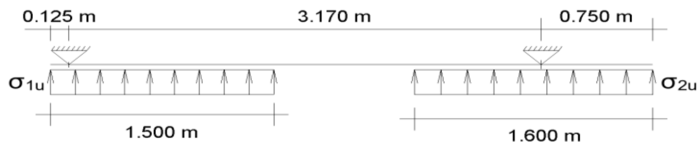


Figure 3.49. Static scheme model for analyzing the strap footing

The longitudinal bending moment and shear force obtained is provided in figure 3.50 and figure 3.51 respectively.

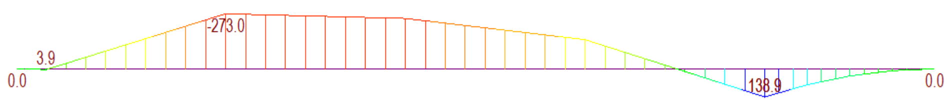


Figure 3. 50. The longitudinal bending moment obtained from the structural analysis

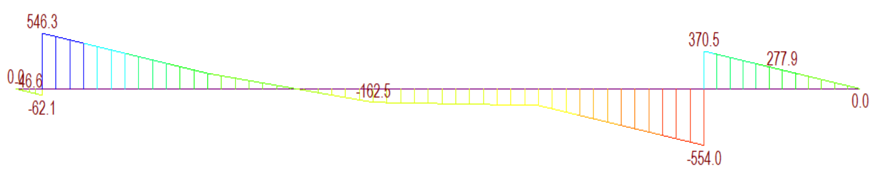


Figure 3.51. The shear force obtained from the structural analysis of the strap footing

The summary of the design of steel reinforcement for footings and strap beams is provided in table 3.36.

Table 3. 36. Design of steel reinforcement for the strap footing

Design of steel reinforcement for footing under column B11 with $M_{Ed} = 79$kN.m						
d (mm)	k	k_{bal}	Z (mm)	A_s(mm^2)	$A_{s,min}$(mm^2)	Arrangement of bars
362	0.01	0.167	343.9	587.4	806.93	8ø12 - 200 mm
Design of steel reinforcement in the x-direction for footing under column B12 with $M_{Ed} = 138.9$kN.m						
d (mm)	k	k_{bal}	Z (mm)	A_s(mm^2)	$A_{s,min}$(mm^2)	Arrangement of bars
362	0.02	0.167	343.9	983.1	697.41	8ø14 - 150mm

Design of steel reinforcement in y-direction for footing under column B12 with $M_{Ed} = 79$kN.m						
d (mm)	k	k_{bal}	Z (mm)	A_s(mm^2/	$A_{s,min}$(mm^2)	Arrangement of bars
362	0.01	0.167	343.9	587.4	806.93	8⌀12 - 200 mm
Design of top longitudinal steel reinforcement for the strap beam with $M_{Ed} = 273$ kN.m						
d (mm)	k	k_{bal}	Z (mm)	A_s(mm^2/	$A_{s,min}$(mm^2)	Arrangement of bars
461	0.13	0.167	401.5	1725.2	273.9	5⌀20 + 2⌀16
The bottom steel reinforcement provided for the strap beam is 4⌀12						

d. Design of shear reinforcement

The summary of the design of shear reinforcement is provided in table 3.37.

Table 3. 37. shear reinforcement design for the strap beam

V_{ED} (kN)	V_{rdc} (kN)	$V_{rdc} \leq V_{ED}$	$A_s/S(mm^2/mm)$	Arrangement of bars
162	109.9	Yes, shear reinforcement is needed	0.33	4legs - ⌀8 - 300 mm

3.6. Detailed report of the structural design

Based on the result of the structural design phase, a structural database is developed using the S-BIM tool Revit. This database consists of steel reinforcement drawings.

3.7.1. Steel reinforcement details of structural elements

The steel reinforcement details of structural elements consist of steel reinforcement for beams, columns, slabs, staircase and foundations elements.

3.7.1.1. Steel reinforcement of the principal beam

Structural reinforcement plan of beam located on axis 8 is provided through the longitudinal section, transversal section, bar bending details and 3D reinforcement view respectively in figure 3.52, figure 3.53 and figure 3.54.

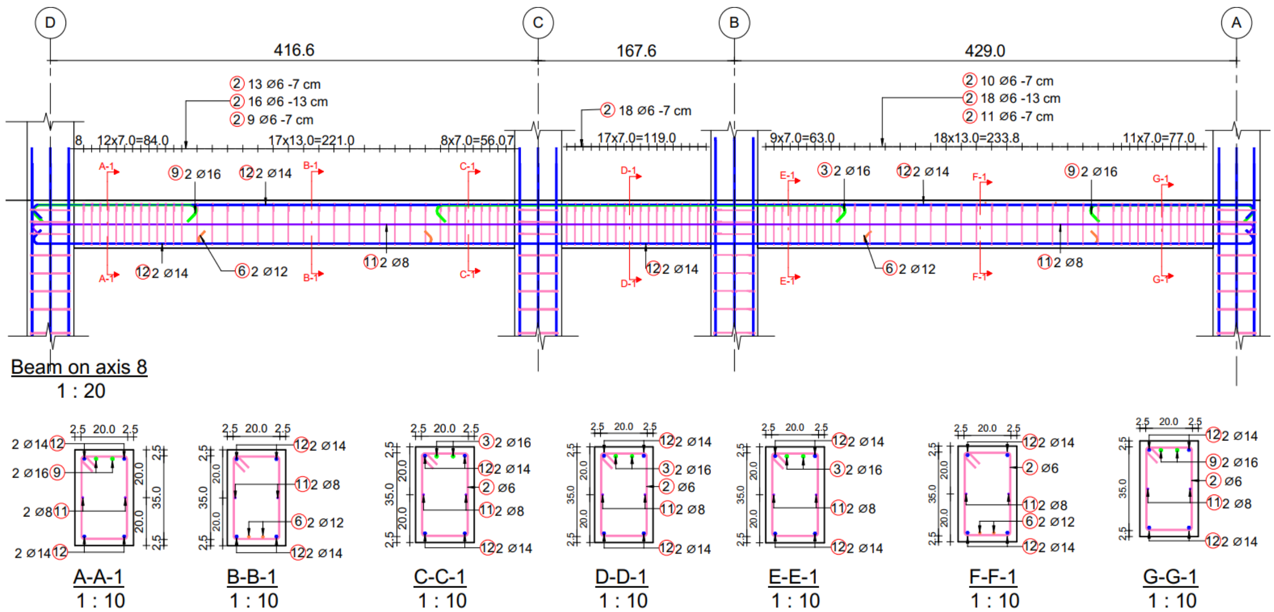


Figure 3.52. 2D steel reinforcement for the beam on axis 8

The bar bending detail presented in figure 3.53 illustrates the diameter of the bar, the shape of bending, the length of each bent and straight portion, and the number of each type of bar.

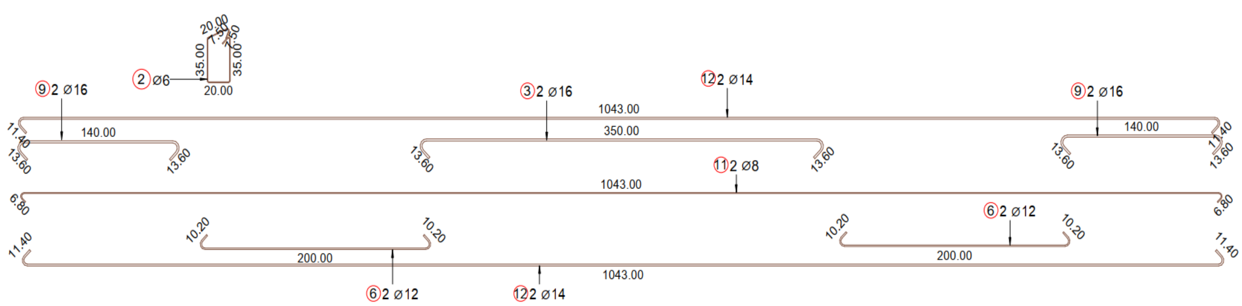


Figure 3.53. Bar bending detail for the beam

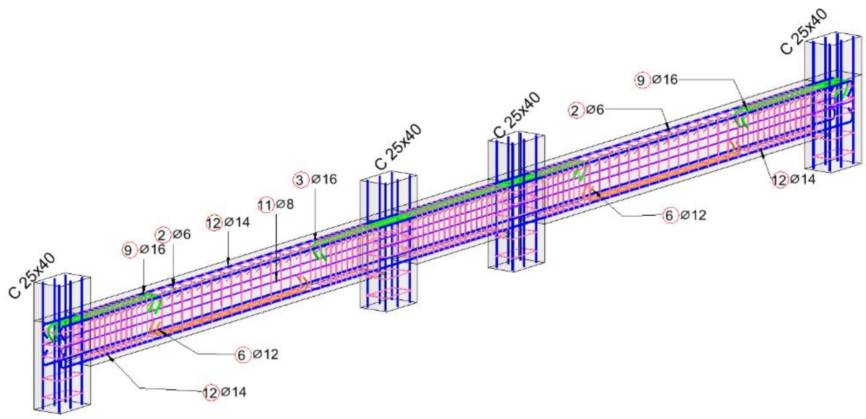


Figure 3.54. 3D steel reinforcement view for the beam on axis 8

The rebar schedule of steel reinforcement for beam on axis 8 is given in table 3.38. The number of rebars with a length of 11.5m for each of the diameters 6mm, 8mm, 12mm, 14mm and16mm are respectively 10 bars, 2 bars, 1bar, 4 bars and 1.5 bars.

Table 3. 38. Schedule of steel reinforcement for beam on axis 8 at the first floor

<Rebar Schedule for beam on axis 8>

A	B	C	D	E	F	G	H
Partition	Rebar Number	Bar Diameter	Quantity	Bar Length	Total Bar Length	Rebar Weight	Number of industrial Rebar
Beam on axis 8	2	6 mm	95	121.31 cm	11524.39 cm	25.58 kg	10.02
6 mm			95		11524.39 cm	25.58 kg	10.02
Beam on axis 8	11	8 mm	2	1056.71 cm	2113.42 cm	8.34 kg	1.84
8 mm			2		2113.42 cm	8.34 kg	1.84
Beam on axis 8	6	12 mm	4	220.56 cm	882.26 cm	7.83 kg	0.77
12 mm			4		882.26 cm	7.83 kg	0.77
Beam on axis 8	12	14 mm	4	1065.64 cm	4262.54 cm	51.51 kg	3.71
14 mm			4		4262.54 cm	51.51 kg	3.71
Beam on axis 8	3	16 mm	2	377.42 cm	754.84 cm	11.91 kg	0.66
Beam on axis 8	9	16 mm	4	167.42 cm	669.68 cm	10.57 kg	0.58
16 mm			6		1424.52 cm	22.48 kg	1.24

3.7.1.2. Steel reinforcement of the column

The structural reinforcement plan of column B8 is provided through sections and 3D view respectively in figure 3.56 and figure3.55.

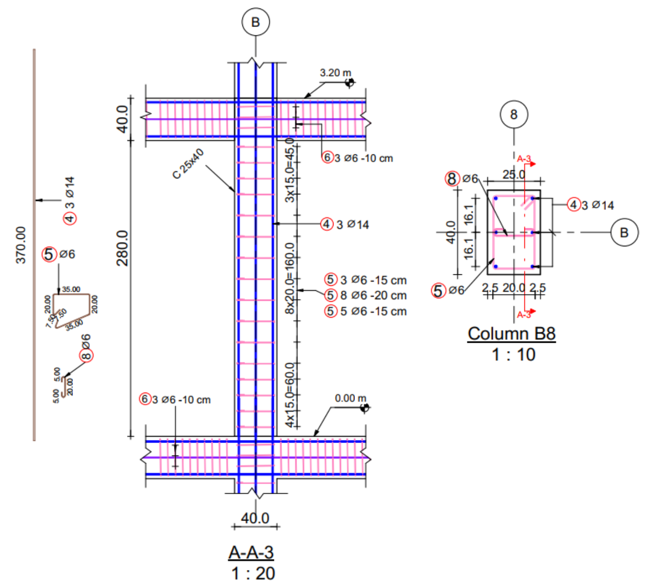


Figure 3.56. 2D steel reinforcement of column

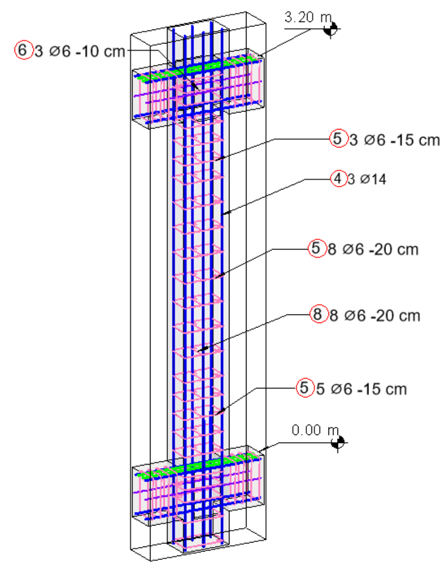


Figure 3.55. 3D steel reinforcement view of the column

The bar bending schedule of steel reinforcement for column B8 is provided in table 3.39.

Table 3. 39. Bar bending schedule for column B8

A	B	C	D	E	F	G	H
<Rebar schedule for Column B8>							
Partition	Rebar Number	Bar Diameter	Quantity	Bar Length	Total Bar Length	Number of industrial Rebar	Rebar weight
Column B8	5	6 mm	16	121.31 cm	1940.96 cm	1.69	4.31 kg
Column B8	6	6 mm	6	121.31 cm	727.86 cm	0.63	1.62 kg
Column B8	8	6 mm	16	32.22 cm	515.60 cm	0.45	1.14 kg
6 mm			38		3184.41 cm	2.77	7.07 kg
Column B8	4	14 mm	6	370.00 cm	2220.00 cm	1.93	26.83 kg
14 mm			6		2220.00 cm	1.93	26.83 kg

The number of rebars with a length of 11.5m for each of the diameters 6mm, and 14mm are respectively 7 bars and 2 bars.

3.7.1.3. Steel reinforcement of the ribbed slab

The longitudinal steel reinforcement plan along the ribs and also bar bending detail is provided in figure 3.57, figure 3.58 and figure 3.59.

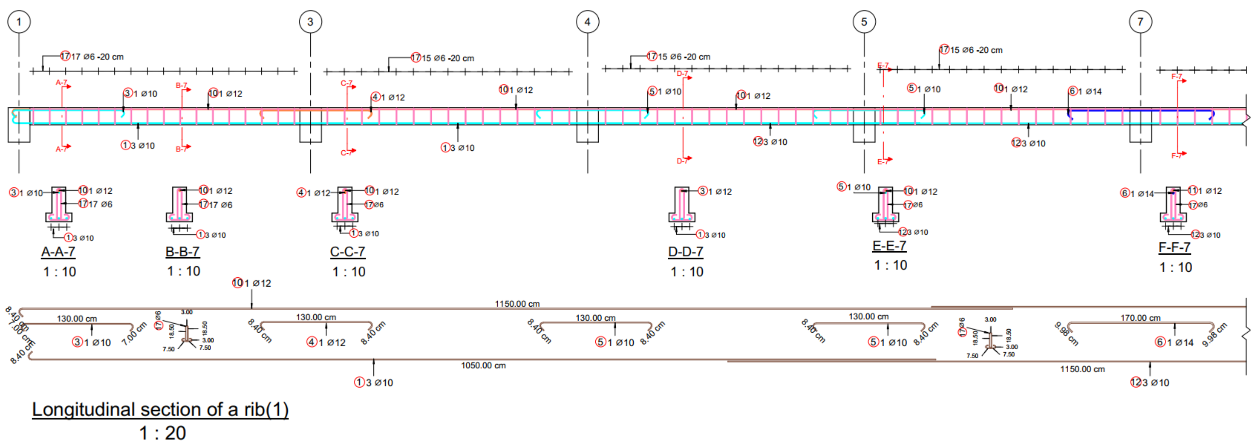


Figure 3.57. Longitudinal cross-section of the rib (part 1)

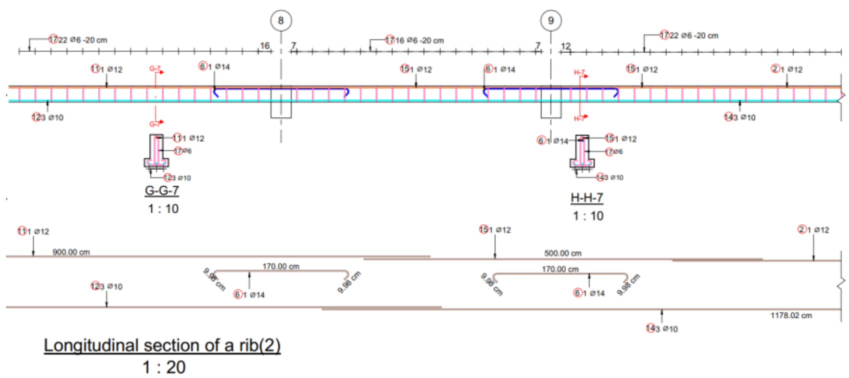


Figure 3.58. Longitudinal cross-section of the rib (part 2)

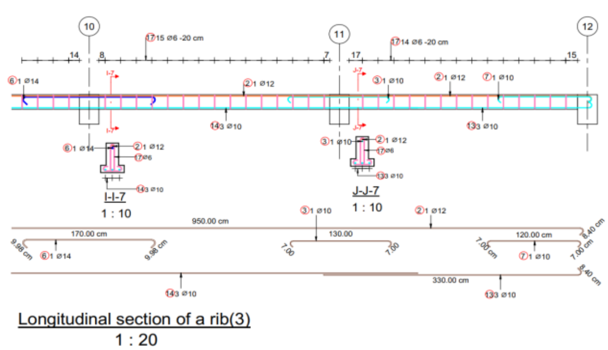


Figure 3. 59. Longitudinal cross-section of the rib (part 3)

The transversal cross-section of the ribbed slab is provided in figure 3.60.

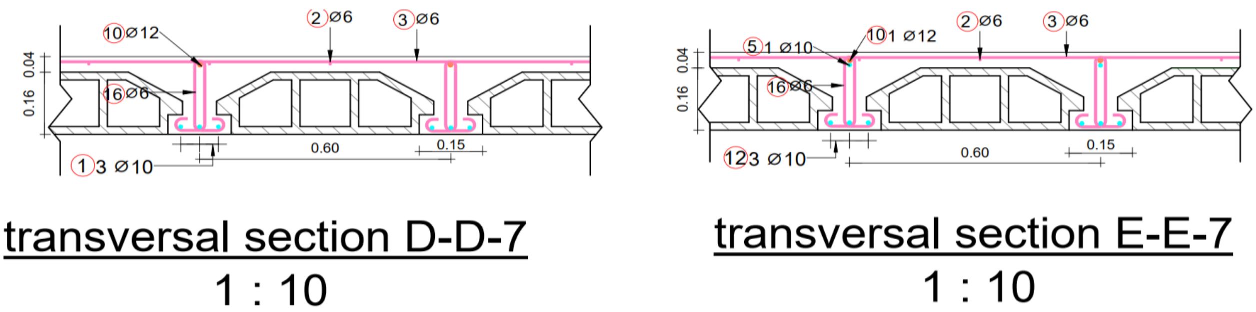


Figure 3. 60. Transversal cross-section of the ribbed slab

A 3D view of the steel reinforcement for a part of the ribbed hollow block slab is provided in figure 3.61.

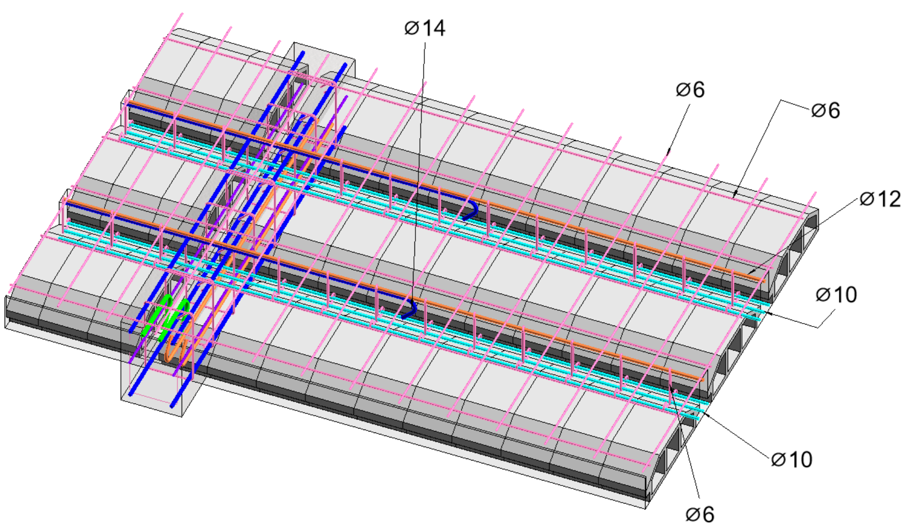


Figure 3. 61. 3D view of the steel reinforcement for the ribbed hollow block slab

The rebar schedule of steel reinforcement for ribs is given in table 3.40.

Table 3. 40. Rebar schedule of a rib

<Rebar Schedule of a rib>

A	B	C	D	E	F	G
Partition	Rebar Number	Bar Diameter	Bar Length	Total Bar Length	Rebar weight	Number of industrial rebars
6 mm						
ribs	17	6 mm	58.50 cm	11523.62 cm	25.58 kg	10.020536
				11523.62 cm	25.58 kg	10.020536
10 mm						
ribs	1	10 mm	1058.65 cm	3175.94 cm	19.58 kg	2.761687
ribs	3	10 mm	144.49 cm	577.97 cm	3.56 kg	0.502586
ribs	5	10 mm	147.29 cm	294.59 cm	1.82 kg	0.256162
ribs	7	10 mm	134.49 cm	268.99 cm	1.66 kg	0.233901
ribs	12	10 mm	1150.00 cm	3450.00 cm	21.27 kg	3
ribs	13	10 mm	338.65 cm	2031.88 cm	12.53 kg	1.766852
ribs	14	10 mm	1178.02 cm	3534.05 cm	21.79 kg	3.073086
				13333.42 cm	82.21 kg	11.594275
12 mm						
ribs	2	12 mm	958.48 cm	1916.96 cm	17.02 kg	1.666926
ribs	4	12 mm	146.96 cm	146.96 cm	1.30 kg	0.127795
ribs	10	12 mm	1158.48 cm	1158.48 cm	10.29 kg	1.007376
ribs	11	12 mm	900.00 cm	900.00 cm	7.99 kg	0.782609
ribs	15	12 mm	500.00 cm	500.00 cm	4.44 kg	0.434783
				4622.41 cm	41.04 kg	4.019488
14 mm						
ribs	6	14 mm	190.15 cm	1140.91 cm	13.79 kg	0.992098
				1140.91 cm	13.79 kg	0.992098
				30620.36 cm	162.61 kg	26.626397

The number of rebars with a length of 11.5m for each of the diameters 6mm, 10mm, 12mm, and 14mm are respectively 10 bars, 12 bars, 4bars and 1bar.

The rebar schedule of structural area reinforcement for the compressive slab is provided in table 3.41, the number of rebars with a length of 11.5m for diameter 6mm is 235 rebars

Table 3. 41. Structural rebar schedule for the compressive slab

<Structural Area Reinforcement Schedule for the Compressive Slab>

A	B	C	D	E	F	G	H
Bar Diameter	Top/Exterior Major Number Of Lines	Top/Exterior Major Spacing	Top/Exterior Minor Number Of Lines	Top/Exterior Minor Spacing	Reinforcement Volume	Rebar Weight	Number of Industrial Rebar
6 mm	161	200 mm	34	300 mm	75656.18 cm³	593.90 kg	234.74347

3.6.1.4. Steel reinforcement of the staircase

The plan view of the staircase and the longitudinal section of the landing is provided in figure 3.62.

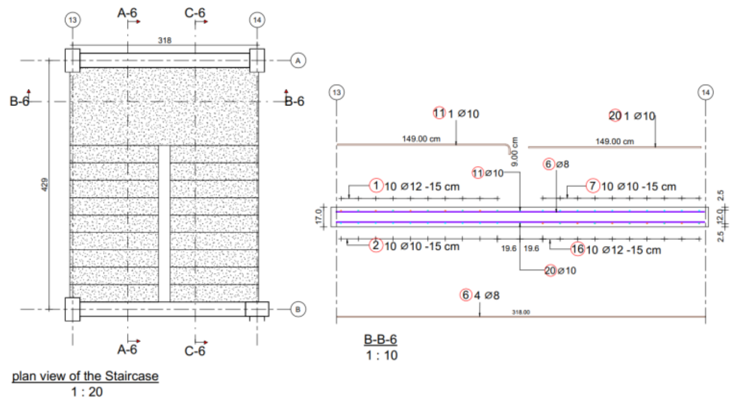
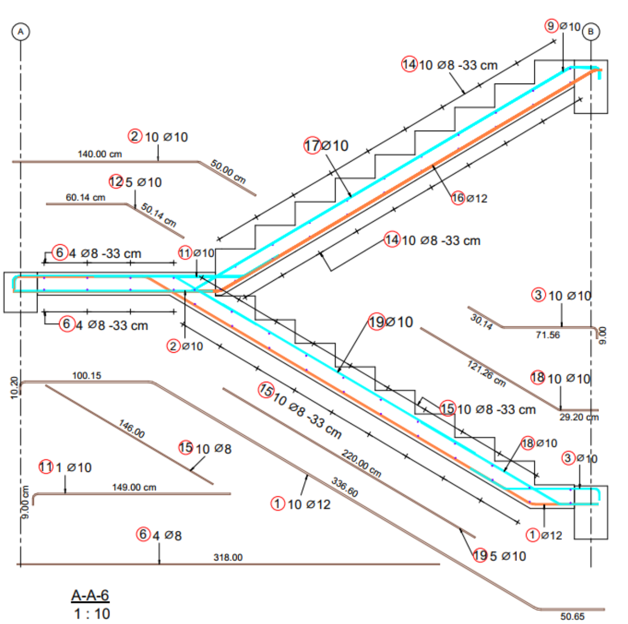


Figure 3. 62. Longitudinal cross-section of the landing

The Structural reinforcement plan throughout the longitudinal section of the staircase and the bar bending details consisting of the diameter of the bar, the shape of bending, the length of each bent and straight portion, and the number of each type of bar are provided in figure 3.63 and figure3.64.



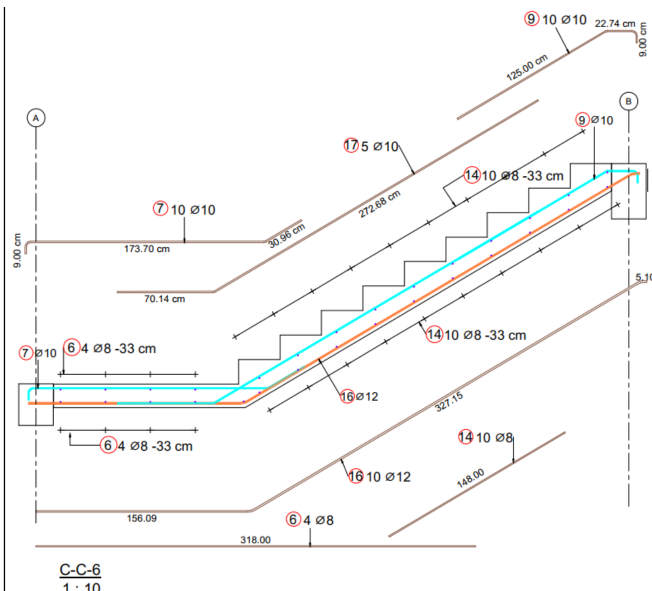


Figure 3. 64. Longitudinal cross-section of the staircase at section A-6

Figure 3. 63. Longitudinal cross-section of the staircase at section C-6

The 3D steel reinforcement view of the staircase is provided in figure 3.65.

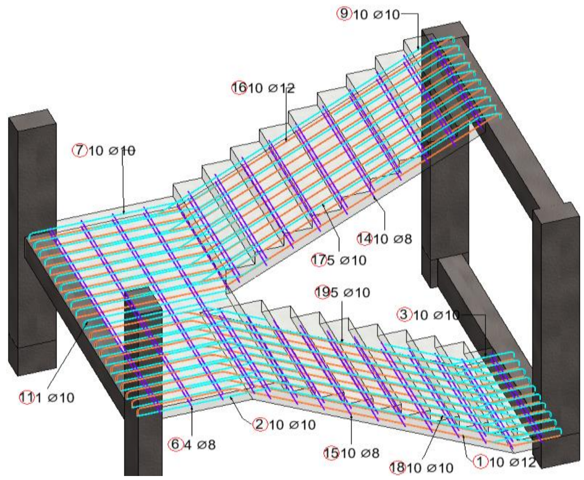


Figure 3. 65.3D steel reinforcement view of the staircase

The Rebar schedule of steel reinforcement for the staircase consisting of informations on bar diameters, the quantity of rebar, bar length, number of industrial rebar and rebar weight are provided in table 3.42.

Table 3. 42. Rebar schedule for the staircase

<Rebar Schedule of the Staircase>							
A	B	C	D	E	F	G	H
Partition	Rebar Number	Quantity	Bar Diameter	Bar Length	Total Bar Length	Number Of Industrial Rebar	Rebar Weight
Staircase	6	4	8 mm	318.00 cm	1272.00 cm	1.106087	5.02 kg
Staircase	13	1	8 mm	147.91 cm	295.81 cm	0.257229	1.17 kg
Staircase	14		8 mm	148.00 cm	3108.00 cm	2.702609	11.68 kg
Staircase	15	10	8 mm	146.00 cm	2920.00 cm	2.53913	11.52 kg
Staircase	21	4	8 mm	325.50 cm	1302.00 cm	1.132174	5.14 kg
8 mm					8897.81 cm	7.737229	34.53 kg
Staircase	2	10	10 mm	189.68 cm	1896.76 cm	1.649354	11.69 kg
Staircase	3	10	10 mm	107.88 cm	1078.79 cm	0.938078	6.65 kg
Staircase	7	10	10 mm	210.84 cm	2108.44 cm	1.833425	13.00 kg
Staircase	9	10	10 mm	153.92 cm	1539.20 cm	1.338438	9.49 kg
Staircase	11	1	10 mm	155.50 cm	155.50 cm	0.135215	0.96 kg
Staircase	12	5	10 mm	109.95 cm	549.77 cm	0.478062	3.39 kg
Staircase	17	5	10 mm	342.50 cm	1712.52 cm	1.489146	10.56 kg
Staircase	18	10	10 mm	150.14 cm	1501.44 cm	1.305599	9.26 kg
Staircase	19	5	10 mm	220.00 cm	1100.00 cm	0.956522	6.78 kg
Staircase	20	1	10 mm	149.00 cm	149.00 cm	0.129565	0.92 kg
10 mm					11791.42 cm	10.253405	72.70 kg
Staircase	1	10	12 mm	493.92 cm	4939.23 cm	4.294987	43.85 kg
Staircase	16	10	12 mm	487.41 cm	4874.06 cm	4.23831	43.27 kg
12 mm					9813.29 cm	8.533296	87.12 kg

The number of rebars with a length of 11.5m for each of the diameters 8mm, 10mm and 12mm, are respectively 8bars, 10.5 bars and 9 bars.

3.7.1.5. Steel reinforcement of a pad foundation

Structural reinforcement of the pad foundation is presented in figures 3.66 and 3.67.

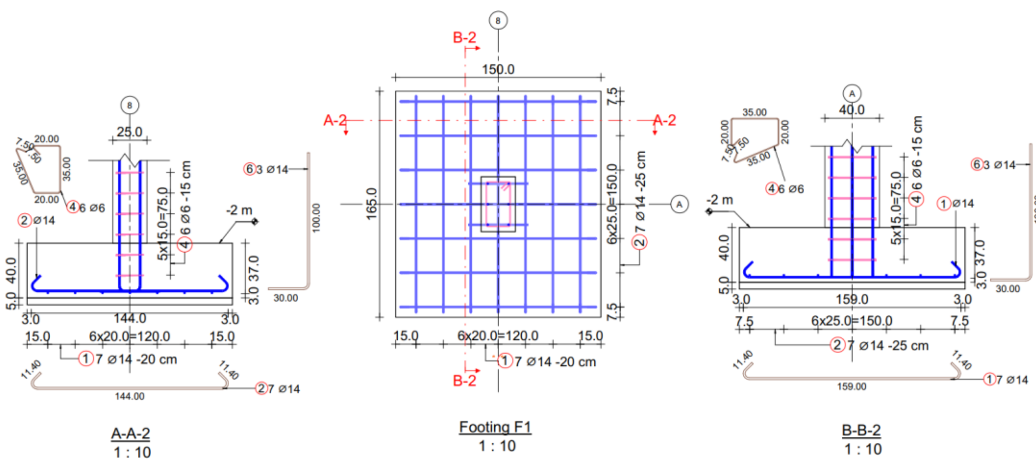


Figure 3. 66. 2D steel reinforcement of pad foundation

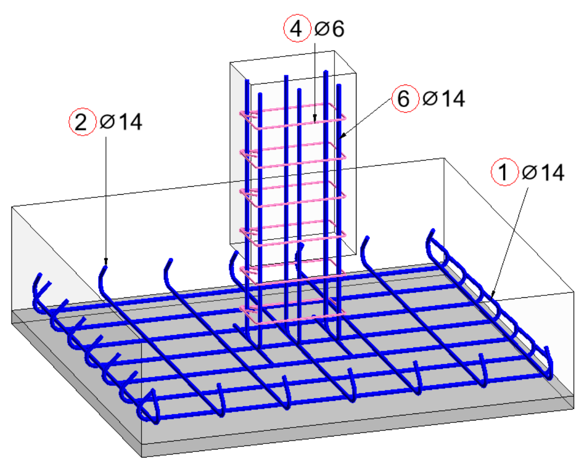


Figure 3. 67. 3D steel reinforcement of pad foundation

The rebar Schedule of steel reinforcement in pad foundation is given in table 3.43.

Table 3. 43. Rebar of schedule for the footing

<Rebar Schedule for Footing F1>							
A	B	C	D	E	F	G	H
Partition	Rebar Number	Bar Diameter	Quantity	Bar Length	Total Bar Length	Number of industrial Rebar	Rebar weight
Footing F1	1	14 mm	7	181.64 cm	1271.45 cm	1.11	15.36 kg
Footing F1	2	14 mm	7	166.64 cm	1166.45 cm	1.01	14.10 kg
14 mm			14		2437.90 cm	2.12	29.46 kg

The number of rebars with a length of 11.5m for a bar diameter of 14 mm is 2.5 bars

3.7.1.6. Steel reinforcement of a strap foundation

The horizontal steel reinforcement plan of the strap footing is presented in figure 3.68

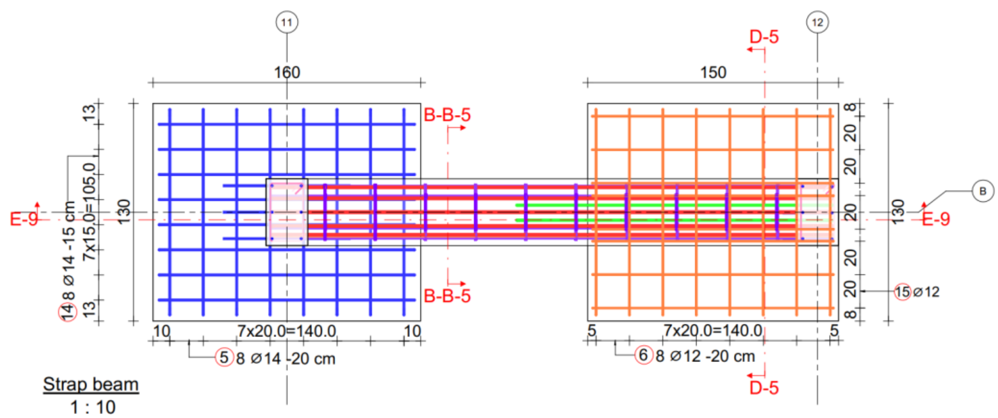


Figure 3. 68. Horizontal steel reinforcement plan of the strap footing

The longitudinal steel reinforcement plan of the strap footing is presented in figure 3.69 and the transversal steel reinforcement plan of the strap footing is presented in figure 3.70.

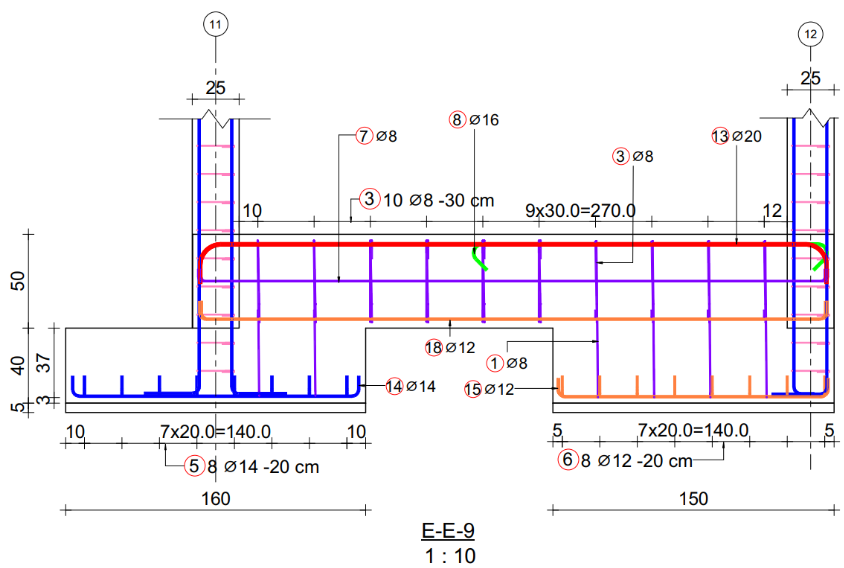


Figure 3. 69. Longitudinal steel reinforcement plan of the strap footing

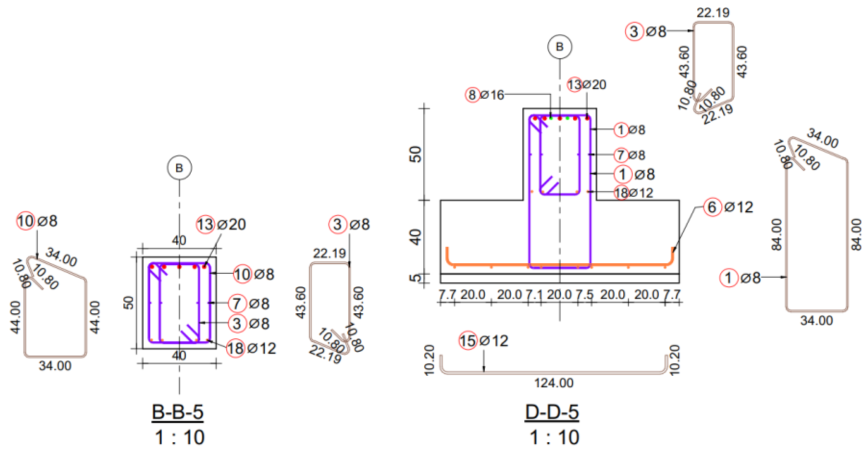


Figure 3. 70. Transversal steel reinforcement for strap foundation

The bar bending detail presented in figure 3.71 illustrates the diameter of the bar, the shape of bending, the length of each bent and straight portion, and the number of each type of bar. The 3D steel reinforcement of the strap foundation is provided in figure 3.72.

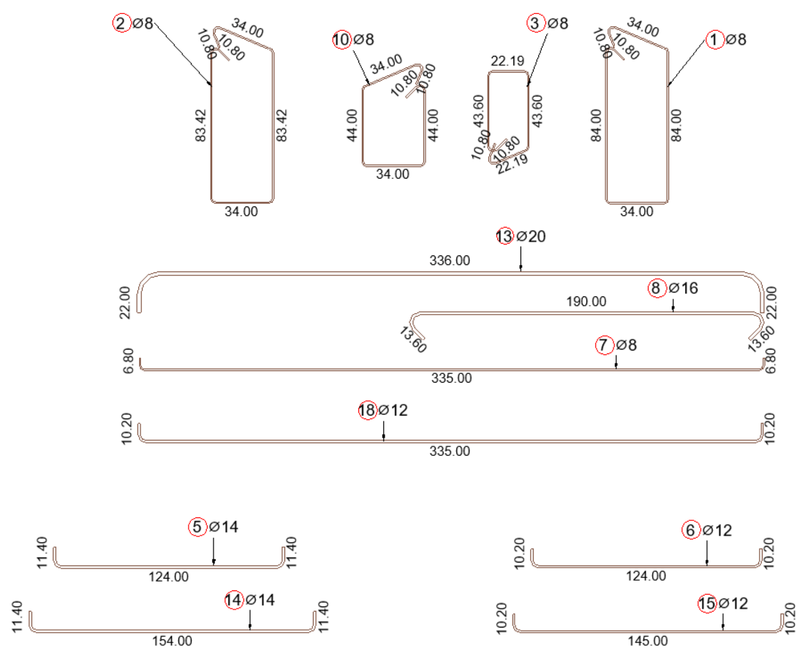


Figure 3. 71. Bar bending detail for the strap beam

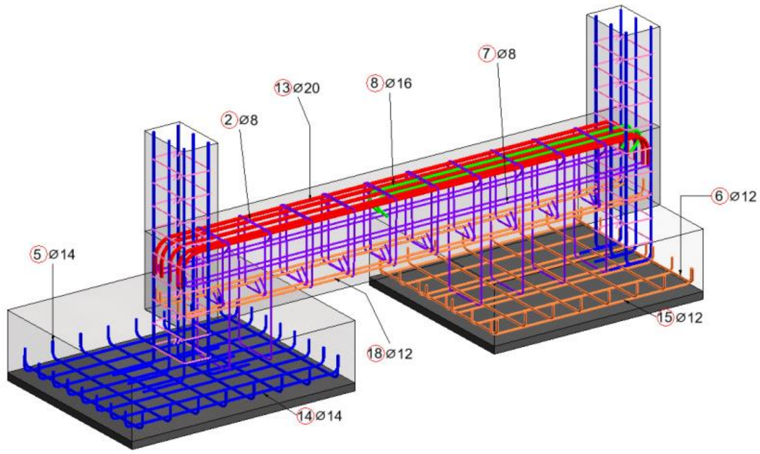


Figure 3. 72. 3D steel reinforcement of strap foundation

The bar diameter, the quantity, the total bar length, the number of bars and the rebar weight are provided in table 3.44.

Table 3. 44. Rebar schedule of steel reinforcement for the strap footing

<Rebar Schedule for strap footing>

A	B	C	D	E	F	G	H
Partition	Rebar Number	Bar Diameter	Quantity	Bar Length	Total Bar Length	Number of industrial rebar	Rebar weight
Strap fondation	1	8 mm	4	252.22 cm	1008.88 cm	0.877285	3.98 kg
Strap fondation	2	8 mm	2	251.05 cm	502.10 cm	0.436611	1.98 kg
Strap fondation	3	8 mm	10	147.79 cm	1477.90 cm	1.285132	5.83 kg
Strap fondation	7	8 mm	2	344.94 cm	689.88 cm	0.599895	2.72 kg
Strap fondation	7	8 mm	2	344.94 cm	689.88 cm	0.599895	2.72 kg
Strap fondation	10	8 mm	4	172.22 cm	688.88 cm	0.599024	2.72 kg
8 mm						4.397843	19.96 kg
Strap fondation	6	12 mm	8	138.91 cm	1111.28 cm	0.966329	9.87 kg
Strap fondation	15	12 mm	2	159.91 cm	319.82 cm	0.278104	2.84 kg
Strap fondation	15	12 mm	3	159.91 cm	479.73 cm	0.417156	4.26 kg
Strap fondation	15	12 mm	3	159.91 cm	479.73 cm	0.417156	4.26 kg
Strap fondation	18	12 mm	2	349.91 cm	699.82 cm	0.608539	6.21 kg
Strap fondation	18	12 mm	2	349.91 cm	699.82 cm	0.608539	6.21 kg
12 mm						3.295821	33.65 kg
Strap fondation	5	14 mm	8	140.82 cm	1126.59 cm	0.979644	13.61 kg
Strap fondation	14	14 mm	8	170.82 cm	1366.59 cm	1.18834	16.51 kg
14 mm						2.167985	30.13 kg
Strap fondation	8	16 mm	1	217.42 cm	217.42 cm	0.18906	3.43 kg
Strap fondation	8	16 mm	1	217.42 cm	217.42 cm	0.18906	3.43 kg
16 mm						0.378121	6.86 kg
Strap fondation	13	20 mm	2	366.56 cm	733.12 cm	0.637491	18.08 kg
Strap fondation	13	20 mm	3	366.56 cm	1099.67 cm	0.956237	27.12 kg
20 mm						1.593728	45.20 kg

From the bar bending schedule, the number of rebars with a length of 11.5m for each of the diameters 8mm, 12mm, 14mm,16mm and 20 mm are respectively 4.5bars, 3.5 bars, 2.5 bars, half of a bar and 2 bars.

3.7.1.7 Steel reinforcement of the combined foundation

The bottom and top horizontal plan for the reinforcement of the combined footing is presented in figure 3.73.

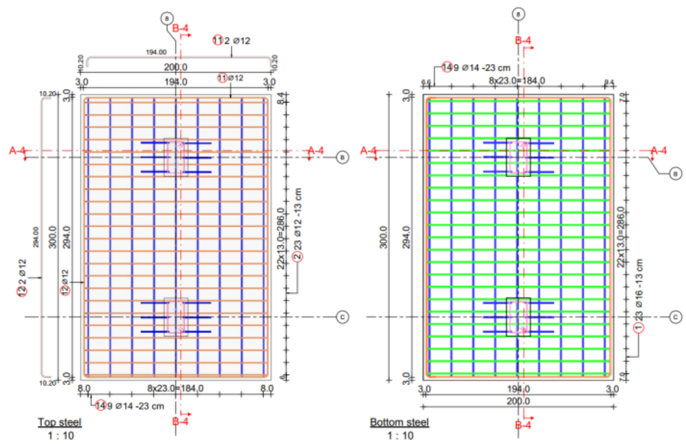


Figure 3. 73. Plan view of steel reinforcement of combined footing

The longitudinal steel reinforcement plan through section B-B-4 of combined footing with bar bending detail is provided in figure 3.74.

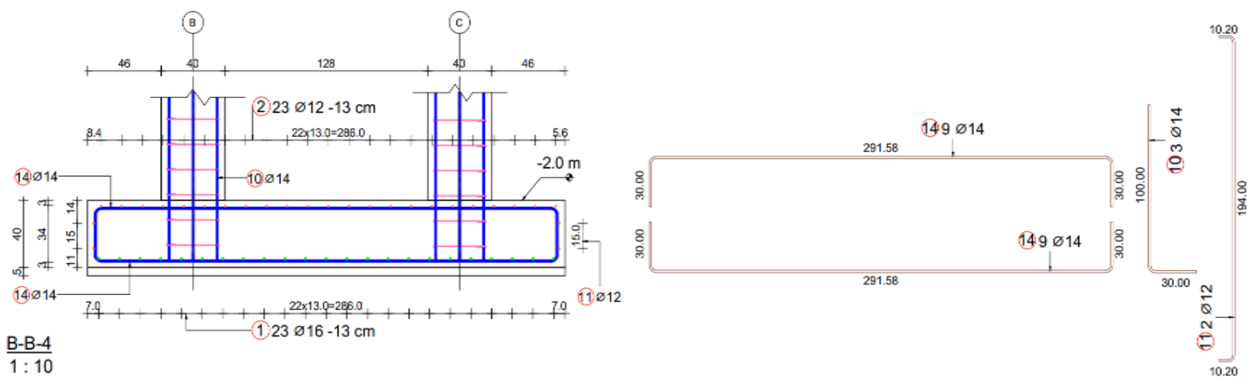


Figure 3. 74. Longitudinal steel reinforcement plan of the combined footing

The transversal steel reinforcement plan through section A-4 of combined footing with bar bending detail is provided in figure 3.75.

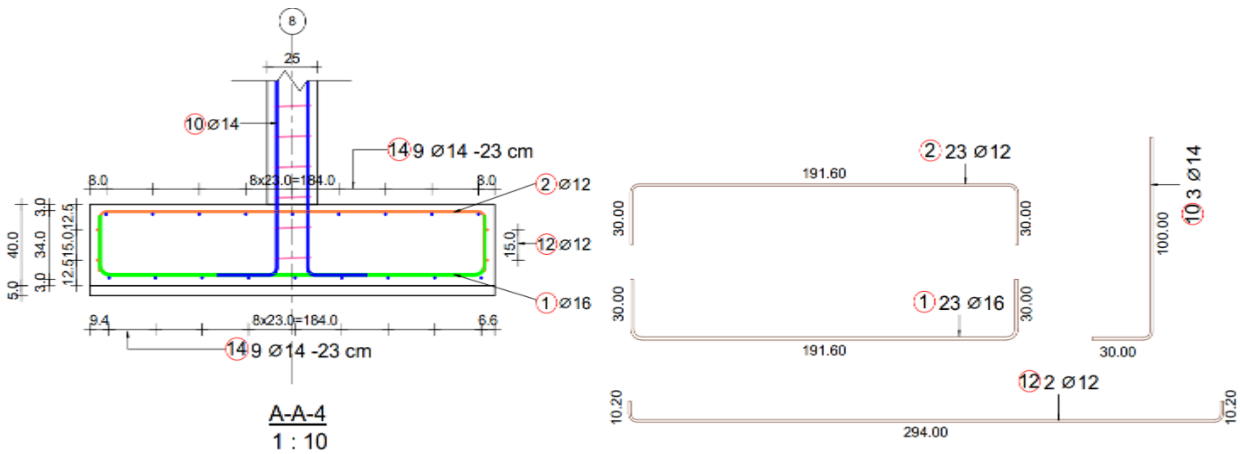


Figure 3. 75. Transversal steel reinforcement plan of the combined footing

The 3D steel reinforcement plan of the combined footing is provided in figure 3.76

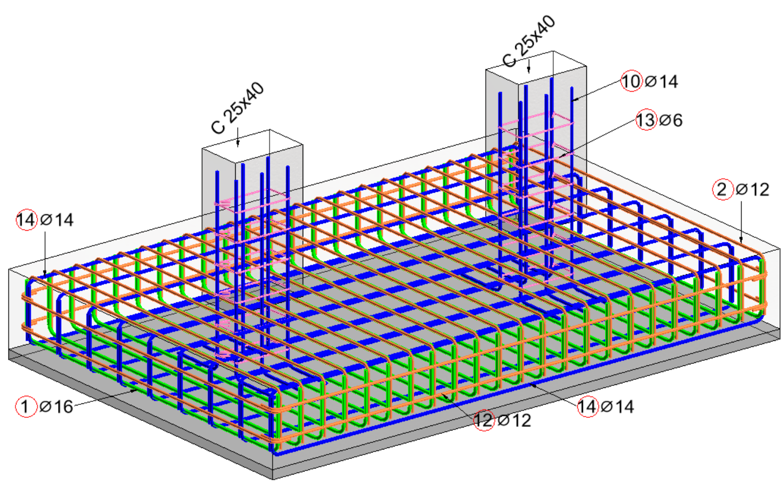


Figure 3. 76. 3D view of the steel reinforcement combined foundation

The schedule of steel reinforcement rebar in the combined footing is provided in table3.45.

Table 3. 45. Rebar schedule for the combined footing

<Rebar Schedule for a combined Footing>

A	B	C	D	E	F	G	H
Partition	Rebar Number	Bar Diameter	Quantity	Bar Length	Total Bar Length	Number of industrial rebar	rebar weight
Combined Footing	1	16 mm	23	244.27 cm	5618.32 cm	4.885498	88.68 kg
16 mm: 1			23		5618.32 cm	4.885498	88.68 kg
Combined Footing	14	14 mm	18	345.60 cm	6220.86 cm	5.409447	75.17 kg
14 mm: 2			18		6220.86 cm	5.409447	75.17 kg
Combined Footing	2	12 mm	23	246.11 cm	5660.52 cm	4.922195	50.25 kg
Combined Footing	11	12 mm	4	208.91 cm	835.64 cm	0.726643	7.42 kg
Combined Footing	12	12 mm	4	308.91 cm	1235.64 cm	1.074469	10.97 kg
12 mm: 5			31		7731.80 cm	6.723306	68.64 kg

From the bar bending schedule, the number of rebars with a length of 11.5m for each of the diameters 12mm 14mm and 16 mm are respectively 7 bars, 5.5 bars and 5 bars.

3.7.2. Quantity take-off of concrete material for structural elements

The aim is to provide the volume of concrete material for structural elements of parts A and B of the building such as beams, columns, concrete slab floor, and foundations.

3.7.2.1. Quantity take-off of concrete material for beams

The volume of hardened concrete material for beams at each level for parts A and B of the building is provided in table 3.46, the total volume of hardened concrete is 140.67 m^3.

Table 3. 46. Quantity take-off of concrete material for beams

<Quantity take-off of concrete material for beams>

A	B	C	D	E	F
Comments	Family	Structural Material	Type	Reference Level	Volume
PART A					
PART A	Concrete - Rectangular Beam	Concrete C25/30	B 25x40	Level 1str	1.91 m³
PART A	Concrete - Rectangular Beam	Concrete C25/30	B 25x40	Level 2str	1.91 m³
PART A	Concrete - Rectangular Beam	Concrete C25/30	B 25x40	Level 3str	1.91 m³
PART A	Concrete - Rectangular Beam	Concrete C25/30	B 25x30	Level 4str	1.65 m³
PART A	Concrete - Rectangular Beam	Concrete C25/30	B 25x30	Level 5str	1.65 m³
PART A	Concrete - Rectangular Beam	Concrete C25/30	B 25x40	Level 6str	1.91 m³
PART A					10.95 m³
PART B					
PART B	Concrete - Rectangular Beam	Concrete C25/30	B 25x40	Level 1str	20.89 m³
PART B	Concrete - Rectangular Beam	Concrete C25/30	B 25x40	Level 2str	20.89 m³
PART B	Concrete - Rectangular Beam	Concrete C25/30	B 25x40	Level 3str	20.89 m³
PART B	Concrete - Rectangular Beam	Concrete C25/30	B 25x40	Level 4str	20.89 m³
PART B	Concrete - Rectangular Beam	Concrete C25/30	B 25x40	Level 5str	20.89 m³
PART B	Concrete - Rectangular Beam	Concrete C25/30	B 25x40	Level 6str	20.89 m³
PART B	Concrete - Rectangular Beam	Concrete C25/30	B 25x40	Level 7str	4.36 m³
PART B					129.72 m³
Grand total: 447					140.67 m³

3.7.2.2.Quantity take-off of concrete material for columns

The volume of hardened concrete material for columns at each level of parts A and B of the building is provided in table 3.47, the total volume of hardened concrete is 74.70 m^3

Table 3. 47. Quantity take-off of concrete material for columns

<Quantity take-off of concrete material for columns>

A	B	C	D	E	F
Comments	Family	Structural Material	Type	Base Level	Volume
Level 0 str					
PART A	Concrete Rectangular	Concrete C25/30	C 25x40	Level 0 str	1.40 m³
PART B	Concrete Rectangular	Concrete C25/30	C 25x40	Level 0 str	8.00 m³
Level 1str					
PART A	Concrete Rectangular	Concrete C25/30	C 25x40	Level 1str	1.28 m³
PART B	Concrete Rectangular	Concrete C25/30	C 25x40	Level 1str	3.52 m³
Level 2str					
PART A	Concrete Rectangular	Concrete C25/30	C 25x40	Level 2str	1.92 m³
PART B	Concrete Rectangular	Concrete C25/30	C 25x40	Level 2str	12.80 m³
Level 3str					
PART A	Concrete Rectangular	Concrete C25/30	C 25x40	Level 3str	1.92 m³
PART B	Concrete Rectangular	Concrete C25/30	C 25x40	Level 3str	12.80 m³
Level 4str					
PART A	Concrete Rectangular	Concrete C25/30	C 25x40	Level 4str	1.80 m³
PART B	Concrete Rectangular	Concrete C25/30	C 25x40	Level 4str	12.16 m³
Level 5str					
PART A	Concrete Rectangular	Concrete C25/30	C 25x40	Level 5str	1.80 m³
PART B	Concrete Rectangular	Concrete C25/30	C 25x40	Level 5str	12.00 m³
Level 6str					
PART B	Concrete Rectangular	Concrete C25/30	C 25x40	Level 6str	3.30 m³
Grand total: 257					74.70 m³

3.7.2.3.Quantity take-off of concrete material for the ribbed hollow slab

The volume of hardened concrete material for the ribbed hollow sab (16+4) cm is provided through table 3.48 and table 3.49 where table 3.48 presents the volume of concrete for ribs at a floor level for parts A and B of the building and table 3.49 presents the volume of concrete materials for concrete slab over the ribs at a floor level.

Table 3. 48. Quantity take - off of concrete material for ribs

<Quantity take – off of concrete material for ribs>

A	B	C	D	E
Level	Comments	Mark	Length	Volume
Level 2str	PART A	ribs	8.38 m	0.198 m³
PART A			8.38 m	0.198 m³
Level 2str	PART B	ribs	546.62 m	14.507 m³
PART B			546.62 m	14.507 m³
Grand total			555.00 m	14.705 m³

Table 3. 49. Quantity take – off of concrete slab over the ribs

<Quantity take -off of concrete slab over the ribs>				
A	B	C	D	E
Level	Comments	Default Thickness	Area	Volume
Level 2str	concrete slab over the ribs	4.00 cm	284.76 m²	11.39 m³
			284.76 m²	11.39 m³

The total volume of hardened concrete material for the ribbed hollow slab block (16+4) cm for each floor level of parts A and B of the building is 26.1 m^3.

3.7.2.4. Quantity take-off of concrete material for the staircase

The volume of hardened concrete for the staircase located on part A is provided in table 3.50, the total volume of hardened concrete is 12.4 m^3.

Table 3. 50. Quantity take-off of concrete material for the staircase

<Quantity Take off for staircase>						
A	B	C	D	E	F	G
ub	Comments	Base Level	Top Level	Actual Number of Ri	Count	Material: Volume
Cast-In-Place Stair	PART A	Level 1str	Level 2str	20	1	3.1 m³
Cast-In-Place Stair	PART A	Level 2str	Level 3str	20	1	3.1 m³
Cast-In-Place Stair	PART A	Level 3str	Level 4str	20	1	3.1 m³
Cast-In-Place Stair	PART A	Level 4str	Level 5str	20	1	3.1 m³
						12.4 m³

3.7.2.5. Quantity take-off of concrete material for the foundations

The volume of hardened concrete material for Pad footings, combined footings and strap footings is provided in table 3.51, the total volume of hardened concrete is 50.21 m^3.

Table 3. 51. Quantity take-off of concrete material for structural foundations elements

<Quantity take -off of concrete material for structural fondations elements>							
A	B	C	D	E	F	G	H
Level	Comments	Family	Mark	Structural Material	Type	Count	Volume
CF (200x300x40)							
Level 0 str	PART B	Footing-Rectangular	Combined Footing	Concrete C25/30	CF (200x300x40)	5	12.00 m³
F1 (150x165x40)							
Level 0 str	PART B	Footing-Rectangular	Pad footing	Concrete C25/30	F1 (150x165x40)	14	13.86 m³
F2 (120x140x40)							
Level 0 str	PART B	Footing-Rectangular	Pad Footing	Concrete C25/30	F2 (120x140x40)	5	3.36 m³
SF2 (300x200x40)							
Level 0 str	PART B	Footing-Rectangular	Strap Footing	Concrete C25/30	SF2 (300x200x40)	2	4.80 m³
Strap beam 1							
Level 0 str	PART B	Ground Beam	Strap Footing	Concrete C25/30	Strap beam 1	3	1.86 m³
strap beam 2							
Level 0 str	PART B	Footing-Rectangular	Strap Footing	Concrete C25/30	strap beam 2	1	1.08 m³
Strap beam 3							
Level 0 str	PART B	Ground Beam	Strap Footing	Concrete C25/30	Strap beam 3	3	1.75 m³
Strap beam 4							
Level 0 str	PART B	Ground Beam	Strap Footing	Concrete C25/30	Strap beam 4	2	1.83 m³
strap footing F1(150x130x40)							
Level 0 str	PART A	Footing-Rectangular	Strap footing	Concrete C25/30	strap footing F1(150x130x40)	3	2.34 m³
Level 0 str	PART B	Footing-Rectangular	Strap Footing	Concrete C25/30	strap footing F1(150x130x40)	3	2.34 m³
strap footing F2(160x130x40)							
Level 0 str	PART A	Footing-Rectangular	Strap Footing	Concrete C25/30	strap footing F2(160x130x40)	3	2.50 m³
Level 0 str	PART B	Footing-Rectangular	Strap Footing	Concrete C25/30	strap footing F2(160x130x40)	3	2.50 m³
						47	50.21 m³

The volume of hardened blinding concrete under foundations elements is provided in table 3.52, the total volume of hardened blinding concrete is 5.47 m^3.

Table 3. 52. Quantity take-off of concrete material for blinding concrete

<Quantity take -off of concrete material for blinding concrete>					
A	B	C	D	E	F
Level	Type	Comments	Height Offset From Level	Default Thickness	Volume
Level 0 str	Blinding concrete	PART A	-0.40	0.05	0.60 m³
Level 0 str	Blinding concrete	PART B	-0.40	0.05	4.86 m³
Grand total					5.47 m³

Conclusion

This chapter aimed to present the case study, then to perform the structural analysis and design of structural elements using a BIM process and produce construction document consisting of steel reinforcement plans associated with rebar schedule and quantity take-off of concrete materials. Base on the geometrics of the building, a structural BIM model of the building is developed with the software Revit. This model is transfered to the sofware Midas Gen using the pluguins “ Midas link for revit structure” then a structural analysis is performed, internal forces abtained are use to perform the structural design of structural elements with Excel spreadsheet. Base on the results of the structural design, the steel reinforcement drawings of structural elements associated with bar bending schedule and rebar schedule are provided using the software Revit. Also the volume of concrete for each types of structural elements are obtained with a total volume of concrete equal to 388 m^3. It can be noticed that BIM facilitates the sharing of structural information inside the structural design process leading to minimise the manual re-entering of data then minimising the probability of errors and enable the re-use of data and also provides an Accurate evaluation of bill of quantities extracted from the structural BIM model.

GENERAL CONCLUSION

The main objective of this work was to perform the structural design of a reinforced concrete building using the BIM process, by providing structural design deliverables such as quantity take-off of reinforced concrete elements, 2D and 3D steel reinforcement drawings associated with bar bending details and bar bending schedule of structural elements. To achieve this objective a literature review gave the presentation of concrete and steel materials together with the knowledge on reinforced concrete buildings and basic concepts of Building Information Modelling. This followed a methodology to perform the structural analysis and design of a reinforced concrete multi-storey building using the BIM process. With this methodology, an R+5 multi-storey building consisting of 3 parts (Part A, B and C) separated by rupture joints were studied. The frame structure of the building is made up of beams with sections 25x40cm, columns of sections 25x40cm whose concrete class is C25/30 is modelled using Revit software. Then, the structural model was transferred from Revit to Midas Gen software, the loads and load combinations are added to the model. The static analysis of the structure leads to find internal forces such as axial forces, moments and shear forces used to determine steel reinforcement area of structural elements using an Excel spreadsheet conceived. The structural design of the ribbed hollow block floor (16+4) cm and for the staircase are also performed.

The result obtained from the structural design of Part A and B of the building shows that the total volume of concrete for beams, columns, ribbed hollow block slab , staircases, structural foundation elements and binding concrete is respectively 140.67 m^3, 74.70 m^3, 104.4 m^3, 12.4 m^3, 50.21 m^3 and 5.47 m^3. Also, the number of steel reinforcement rebars with a length of 11.5m for each bar diameter for a structural element is available. Hence for a principal beam, the number of rebars of diameters 6mm, 8mm, 12mm, 14mm and16mm are respectively $5\frac{1}{2}$, 2, 1, 4 and $1\frac{1}{2}$. For a column, the number of rebars of diameters 6mm, and 14mm is respectively $2\frac{1}{2}$, and 2. Concerning the ribbed hollow block slab, the number of rebars of diameters 6mm, 10mm, 12mm, and 14mm for a rib are respectively 10 , 12, 4 and 1. The number of rebars of diameter 6mm for the concrete slab over the ribs is 235. For the staircase, the number of rebars of diameters 8mm, 10mm and

12mm, are respectively 8, $10\frac{1}{2}$, and 9. Concerning the foundation, the number of rebars of diameter 14 mm for each pad footing F1 is $2\frac{1}{2}$. The number of rebars for diameters 8mm, 12mm, 14mm,16mm and 20 mm for the strap beam under columns B12 and B1 are respectively $4\frac{1}{2}$, $3\frac{1}{2}$, $2\frac{1}{2}$, $\frac{1}{2}$, and 2. The number of rebars for diameter 12mm, 14mm and 16 mm for the combined footing are respectively 7, $5\frac{1}{2}$ and 5.The following conclusion were deduced from the analysis results:

- BIM provides a flexible environment for interoperability and collaboration facilitating the sharing of structural information.
- BIM provides an accurate evaluation of bill of quantities extracted from the structural BIM model.
- The 3D steel reinforcement drawings eliminates any misunderstanding of the design intent.
- The bar bending details facilitate execution and reduces construction time.

The limit of this work was that seismic actions and soil-structure interaction was not considered for the structural design of the building. The perspectives could be as followed:

- Study the clash detection and coordination of structural building elements with architectural and HVAC elements.
- Study the structural design of steel building or timber building using the BIM process.

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ANNEXES

ANNEXE A: Imposed loads and partial factors on a building

Table 1: Imposed loads on in the building (EN1990)

Categories of loaded areas	q_k [kN/m^2]	Q_k [kN]
Category A		
- **Floors**	1,5 to2,0	2,0 to 3,0
- **Stairs**	2,0 to4,0	2,0 to 4,0
- **Balconies**	2,5 to 4,0	2,0 to 3,0
Category B	2,0 to 3,0	1,5 to 4,5
Category C		
- C1	2,0 to 3,0	3,0 to 4,0
- C2	3,0 to 4,0	2,5 to 7,0 (4,0)
- C3	3,0 to 5,0	4,0 to 7,0
- C4	4,5 to 5,0	3,5 to 7,0
- C5	5,0 to 7,5	3,5 to 4,5
category D		
- D1	4,0 to 5,0	3,5 to 7,0 (4,0)
- D2	4,0 to 5,0	3,5 to 7,0

Table 2: partial factors of imposed loads (EN1990)

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991-1-1)			
Category A : domestic, residential areas	0,7	0,5	0,3
Category B : office areas	0,7	0,5	0,3
Category C : congregation areas	0,7	0,7	0,6
Category D : shopping areas	0,7	0,7	0,6
Category E : storage areas	1,0	0,9	0,8
Category F : traffic area, vehicle weight ≤ 30kN	0,7	0,7	0,6
Category G : traffic area, 30kN < vehicle weight ≤ 160kN	0,7	0,5	0,3
Category H : roofs	0	0	0
Snow loads on buildings (see EN 1991-1-3)*			
Finland, Iceland, Norway, Sweden	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l.	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H ≤ 1000 m a.s.l.	0,50	0,20	0
Wind loads on buildings (see EN 1991-1-4)	0,6	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0
NOTE The ψ values may be set by the National annex. * For countries not mentioned below, see relevant local conditions.			

ANNEXE B: Exposure class and strength class

Corrosion										
	Carbonation-induced corrosion				Chloride-induced corrosion			Chloride-induced corrosion from sea-water		
	XC1	XC2	XC3	XC4	XD1	XD2	XD3	XS1	XS2	XS3
Indicative Strength Class	C20/25	C25/30	C30/37		C30/37		C35/45	C30/37	C35/45	

Damage to Concrete							
	No risk	Freeze/Thaw Attack			Chemical Attack		
	X0	XF1	XF2	XF3	XA1	XA2	XA3
Indicative Strength Class	C12/15	C30/37	C25/30	C30/37	C30/37		C35/45

ANNEXE C: Maximum spacing between bars and bar maximum bar diameter

Table 1. Maximum spacing between bars (EN1992-1-1)

Steel stress[2] [MPa]	Maximum bar spacing [mm] w_k=0,4 mm	w_k=0,3 mm	w_k=0,2 mm
160	300	300	200
200	300	250	150
240	250	200	100
280	200	150	50
320	150	100	-
360	100	50	-

Table 2. Maximum bar diameter (EN1992-1-1)

Steel stress[2] [MPa]	Maximum bar size [mm] w_k= 0,4 mm	w_k= 0,3 mm	w_k= 0,2 mm
160	40	32	25
200	32	25	16
240	20	16	12
280	16	12	8
320	12	10	6
360	10	8	5
400	8	6	4
450	6	5	-

ANNEXE D: tabulated value of the limit/span ratio (l/d) and structural factor K

Structural System	K	Concrete highly stressed ρ = 1,5%	Concrete lightly stressed ρ = 0,5%
Simply supported beam, one- or two-way spanning simply supported slab	1,0	14	20
End span of continuous beam or one-way continuous slab or two-way spanning slab continuous over one long side	1,3	18	26
Interior span of beam or one-way or two-way spanning slab	1,5	20	30
Slab supported on columns without beams (flat slab) (based on longer span)	1,2	17	24
Cantilever	0,4	6	8

Note 1: The values given have been chosen to be generally conservative and calculation may frequently show that thinner members are possible.
Note 2: For 2-way spanning slabs, the check should be carried out on the basis of the shorter span. For flat slabs the longer span should be taken.
Note 3: The limits given for flat slabs correspond to a less severe limitation than a mid-span deflection of span/250 relative to the columns. Experience has shown this to be satisfactory.