REPUBLIC DU CAMEROUN

Paix-Travail-Patrie

****

****

DEPARTEMENT DE GENIE CIVIL

****

DEPARTMENT OF CIVIL ENGINEERING

****

MINISTRY OF HIGHER EDUCATION

Peace-Work-Fatherland

****

****

DEPARTMENT OF CIVIL, ARCHITECTURAL AND ENVIRONMENTAL ENGINEERING

****

DEVELOPING A FORENSIC ENGINEERING STRUCTURAL FRAMEWORK FOR FAILURE EVIDENCES DURING THE STRUCTURAL REMODELLING OF REINFORCED CONCRETE STRUCTURES: CASE STUDY AN R+1 RC INDUSTRIAL BUILDING IN NSIMALEN, YAOUNDE.

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:

NGOMBEH Rene

Matricule: **15TP20907**

Supervisors:

Prof. Eng. Carmelo MAJORANA

Co-supervisors:

Eng. Giuseppe CARDILLO

Dr. Eng. Guillaume POH'SIE

Academic Year: 2019/2020

Research Question: How does a **forensic engineering framework** help to show failures evidences during the **structural remodelling** of a **reinforced concrete structure**?

DEDICATIONS

I dedicate this work to my beloved wife NGOMBEH Carreen Sheltche for taking care of me from the beginning to the completion of my studies for a Master's Degree in Civil Engineering.

ACKNOWLEDGEMENTS

Before entering into the juicy bit of this work, my acknowledgement goes to:

- God Almighty for the strength He gave me to do this work.
- Professor…………………………………… for the honour he gave me in being the president of the jury in my thesis presentation.
- Professor…………………………………… for being the examiner of my work.
- Professor Carmelo MAJORANA and Professor ESOH Elame High authorities responsible for the partnership ENSTP- UNIPD;
- Professor NKENG George ELAMBO, Director of ENSTP Yaounde;
- Professor Carmelo MAJORANA, Eng. Giuseppe CARDILLO and Dr. Eng. Guillaume POH'SIE supervisors of this thesis for continuous encouragement, correction and cooperation in doing my research work.
- Professor MBESSA Michel, Head of Civil Engineering Department ENSTP;
- The board of lecturers of the University of Padova and of ENSTP Yaounde for the knowledge they transmitted to us during the project;
- The administrative personnel of ENSTP Yaounde for the unquenchable support;
- My promotion colleagues for the boost and energy transmitted every day;
- My children Praise, Awesome, Flourish and Fortress for standing by me throughout my studies;
- My elder brother Dr. ACHO Alphonse for his relentless support and encouragement and equally for the corrections he did on this thesis.
- The entire PIZOH family, for the moral, financial, and spiritual support. Without you I wouldn't have worked on this thesis;
- My friends and all those who helped me, let them have my profound gratitude.

.

LIST OF SIGNS AND ABBREVIATIONS

RC	Reinforced concrete
ULS	Ultimate Limit State
SLS	Serviceability Limit State
t	Index for beam span
G_0	Structural permanent loads
G_1	Non-structural permanent loads
Q_k	Variable load
N_{Ed}	Design axial compression force
N_{Rd}	Resisting axial force
f_{yd}	Design yield strength
A_c	Area of the concrete cross section
G_1T_t	Structural permanent loads on the span t
G_2T_t	Non-structural permanent loads on the span t
Q_kT_t	Variable loads on the span t
A_s	Cross-sectional area of steel reinforcement
A_c	Area of the concrete cross section
$A_{s, max}$	Maximum cross-sectional area of steel reinforcement
$A_{sw, max}$	Maximum shear reinforcement cross section
$A_{s, min}$	Minimum cross-sectional area of steel reinforcement
f_{cd}	Cylindrical concrete compressive strength
f_{yd}	Design yield strength of reinforcement
b_t	Mean width of the tension zone
d	Effective depth of a concrete section
c	Concrete cover
f_{ctm}	Tensile strength of concrete
N_{ed}	Design axial force
M_{ed}	Design moment
M_{rd}	Resisting moment
d	Effective depth of the section

b	Width of concrete section
β_1	Correction factor equal to 0.81
β_2	Correction factor equal to 0.41
As_{real}	Effective area of the steel sections
V_{Ed}	Acting shear
V_{Rdc}	Allowable shear without shear reinforcement
f_{ck}	Characteristic strength of reinforcement
f_{ywd}	Design yield strength of the shear reinforcement
v_1	Reduction factor for concrete cracked in shear
α_{cw}	Coefficient taking account of the state of stress in compression cord
S	Spacing of stirrups
A_{sw}	Cross sectional area of the shear reinforcement with a maximum value
$\emptyset_l$	Diameter of the longitudinal bars
i	Radius of gyration of uncracked concrete
l_o	Effective length of a column element
φ_{ef}	Effective creep ratio
w	Mechanical reinforcement ratio
n	Relative normal force
σ_s	Maximum work stress
σ_{allow}	Allowable bearing capacity of the soil
$A_{s,y}$	Footing reinforcement along the y direction
$A_{s,x}$	Footing reinforcement along the x direction
S.F	Safety factor
Ly	Footing with along y
Lx	Footing width along x
N/A	Not applicable

ABSTRACT

The main objective of this work focuses on the development of a forensic framework to remodel an existing structure in order to avoid failures leading to the collapse of part or all of the structure. The variability and properties of concrete as a material makes reinforced concrete structures susceptible to failure or collapse of part or all of the structure especially when care is not taken in its composition and use (David Moore P.E, 2004). This leads to enormous financial losses and sometimes, human lives are lost. It is therefore inevitable to investigate failure which brings about the field of forensic engineering. In the context of structural engineering, forensic engineering is taken to be the application of engineering principles to investigate and determine the causes of deficiencies in structural performance, the collapse of a structure or its inability to perform the services for which it was constructed. The methodology used permitted the general site recognition, site visit, data collection; physical modelling of building, calculation parameters definition, structural analysis and failure characterisation for the evaluation of an existing building that is to be remodelled. The failure mechanism was simulated taking into consideration the soil structure interaction. In conclusion, the existing load bearing elements were found not applicable for the remodelling. They need to be reinforced to bear the new loads for failure to be avoided, because the safety factors of these load bearing elements were below the required value of 1.

Keywords: Concrete, Reinforced concrete structures, Failure mechanism, Forensic engineering, Soil-Structure Interaction.

RESUME

L'objectif principal de cette étude est le développement d'un cadre forensique pour remodeler une structure existante afin d'éviter les défaillances conduisant à l'effondrement d'une partie ou de la totalité de la structure. La variabilité ainsi que les propriétés du béton comme matériaux rend les structures en béton armé sensible à l'effondrement de toute la structure ou à la défaillance d'une partie de la structure surtout lorsque la composition et la mise en œuvre ne sont pas bien fait (David Moore P.E, 2004). Cela entraîne d'énormes pertes financières et parfois des pertes en vies humaines. Il est donc inévitable que lorsqu'une défaillance se produit, une enquête soit menée. Ceci relève du domaine de l'ingénierie forensique, qui est l'enquête des causes d'effondrement d'une structure ou son incapacité à effectuer les services pour lesquels elle a été construite. La méthodologie utilisée a permis la reconnaissance générale du site, visite du site, collecte des données, modélisation physique du bâtiment, paramétrages des calculs, analyse dynamique et statique sur le logiciel, caractérisation des modes de ruines pour l'évaluation du bâtiment existant à remodelé. Le mécanisme de ruine possible a été simulé en tenant compte de l'interaction sol-structure. Les poteaux, poutres et fondations existant ne sont pas adapter pour les nouvelles charges que vont subir le bâtiment remodelé. Ils doivent être renforcés pour que la défaillance ou l'effondrement soient évités car les facteurs de sécurité de ces éléments porteurs étaient inférieurs à la valeur requise de 1.

Mots-clés : Béton, Structure en béton armé, Mécanismes de ruine, Ingénierie forensique, Interaction Sol-Structure.

LIST OF FIGURES

LIST OF TABLES

TABLE OF CONTENTS

GENERAL INTRODUCTION

Forensic engineering can be defined as the application of engineering sciences to the investigation of failures and/or performance issues. Therefore, forensic engineering deals not only with technical expertise but also with knowledge of the legal procedures. In this context, forensic structural and/or civil engineers have the role of identifying the technical causes that induce failures and responsibilities that cause these failures. The development of a forensic engineering framework for failure evidences arises from a deep problem well known in the Republic of Cameroon.

The problem originates from the fact that very often, homeowners have their homes designed or not by an engineer. Thereafter, they want to make their homes more comfortable and valuable by changing the home's design, or increasing living space without allowing an engineer to carry out an investigation to see if the structure can support the structural remodelling. Such operations cannot be done abruptly because the elements were designed for a specific resistance and there can be overloading leading to failures that will finally cause the collapse of the structure.

The limit of science in the domain of forensic engineering is the availability of technical codes and standards, impartiality, multidisciplinary working and vocational qualifications.

The main objective of this document is to establish a forensic framework that will help engineers to sort out the failure evidences during the structural remodelling of reinforced concrete structures and explain to home owners for correct responsibilities to be taken before the engagement in remodelling activities.

The chapter one presents concrete as a material, reinforced concrete, reinforced concrete structures, failures, structural remodelling and forensic engineering. Chapter two gives the forensic engineering structural framework methodology taking care of the different stages: preliminary stage, evidence collection stage, analysis stage and failure characterisation. Chapter three presents the results and interpretation showing the output of the different analysis and the failure characterised by safety factors of load-bearing elements and foundations. The conclusion and the responsibility assigning phase is given in the general conclusion and perspectives in order to alert structural engineers or forensic engineers on the importance of the forensic engineering structural framework for reinforced concrete buildings remodelling.

CHAPTER 1. LITERATURE REVIEW

Introduction

Structural remodelling helps to increase or change the exploitation of a Reinforced concrete structure. Remodelling needs to be done following a forensic framework to avoid undesirable events such as the failure of structural elements or the collapse of the structure. The objective of this chapter is to review the literature surrounding the topic "developing a forensic engineering structural framework for the structural remodelling of reinforced concrete structures". Firstly, concrete as a material is recalled followed by reinforced concrete. Furthermore, the notion of reinforced concrete structures is explained followed by the remodelling concept applied to structural engineering. Finally, a detailed explanation of forensic engineering is given.

1.1. Concrete

Concrete is the most commonly used man-made material on earth. It is an important construction material used extensively in buildings, bridges, roads and dams. Its uses range from structural applications, to pavious, kerbs, pipes and drains. Concrete is a composite material, consisting mainly of Portland cement, water and aggregate (gravel, sand or rock). When these materials are mixed together, they form a workable paste which then gradually hardens over time. The characteristics of concrete are determined by the aggregate or cement used, or by the method that is used to produce it. The water-to-cement ratio is the determining factor in ordinary structural concrete with a lower water content resulting in a stronger concrete. The benefits of concrete are numerous: It is a relatively cheap material, and has a relatively long life span with few maintenance requirements. It is strong in compression. Before it hardens it is a very pliable substance that can easily be shaped. It is non-combustible. (David Moore P.E, 2004).The limitations of concrete include: relatively low tensile strength when compared to other building materials, low ductility, and low strength-to-weight ratio. It is susceptible to cracking. Figure 1.1 illustrates concrete used for buildings.

Figure 1.1. Concrete construction material
(Source: m.wikipedia.org/concrete)

1.2. Reinforced concrete

Reinforced concrete (RC), also called reinforced cement concrete (RCC), is a composite material in which concrete's relatively low tensile strength and ductility are compensated for by the inclusion of reinforcement having higher tensile strength or ductility. The reinforcement is usually, though not necessarily, steel bars (rebar) and is usually embedded passively in the concrete before the concrete sets. Reinforcing schemes are generally designed to resist tensile stresses in particular regions of the concrete that might cause unacceptable cracking and/or structural failure. Modern reinforced concrete can contain varied reinforcing materials made of steel, polymers or alternate composite material in conjunction with rebar or not. Reinforced concrete may also be permanently stressed (concrete in compression, reinforcement in tension), to improve the behaviour of the final structure under working loads. For a strong, ductile and durable construction the reinforcement needs to have the following minimum properties: high relative strength, high toleration of tensile strain, good bond to the concrete, irrespective of pH, moisture, and similar factors thermal compatibility, not causing unacceptable stresses (such as expansion or contraction) in response to changing temperatures. Durability in the concrete environment, irrespective of corrosion or sustained stress for example. (Ariana. Z, 2016). The most used type of reinforcement is steel in the form of reinforcement bars. Figure 1.2 indicates reinforced concrete with reinforcement bars.

Figure 1.2. Reinforced concrete construction material
(Source: m.wikipedia.org/reinforcedconcrete)

1.3. Reinforced concrete structures

Reinforced concrete structures are structures that use the most universal construction material which is reinforced concrete. They constitute many important infrastructures and projects such as buildings, bridges, stadiums, industrial, and geotechnical infrastructures. Some examples of reinforced concrete structures are foundations, walls, cores, slabs, columns, and beams. Figure 1.3 shows an example of building made of reinforced concrete structures.

Figure 1.3. Reinforced concrete building made of reinforced concrete elements such as beams, pillars. (Source: m.wikipedia.org/reinforcedconcretestructures)

1.4. Failures

To build a structure that meets safety and strength requirements, it is necessary to execute construction process according to the applicable codes and specifications. Failure is often stated as the steppingstone to success, but there is a high price to pay in terms of energy, time, and money. Nobody wants a failure yet they occur. Lessons from failures are everlasting, revealing and often shocking. We define failure as the absence of a derived function, goal or objective, mission, task or purpose. The main factors that cause failure of concrete structures are: incorrect selection of materials, errors in design calculations and detailing, improper construction techniques and insufficient quality control and supervision, chemical attacks on concrete structures, external mechanical factors.

1.4.1 Structural failures

Structural failures refer to the absence of its desired / designed / intended performance, behaviour, response under all expected environmental conditions (loads, forces, etc.). There are tension, compression, shear, flexure and torsion failures, occurring singly or in a combined state. The classical notions of factors of safety have undergone tremendous changes giving rise to partial safety factors and limit state factors. Soil and concrete media have their own unique failure mechanisms. Some examples of structural failures include: flexural-compressional brittle failures, concrete crushing and rebar buckling, overall buckling of thin walls, splice failures, soft floor(irregularities), lack of concrete confinement, deficient reinforcement detailing, shear failure, walls too slender, heavy loaded walls, (Carlos Videla C, 2012)

1.4.1.1.Flexural-Compressional brittle failures

Flexural or compressive brittle failure occurs when the imposed load exceeds the flexural capacity of the materials of the beam. It begins with the crushing of concrete at compression side followed by yielding of steel at tension side of the beam. It occurs when the beam is over-reinforced which means the beam reinforcement ratio is greater than balanced reinforcement ratio. This type of failure is sudden and does not provide warning i.e. brittle, (Wang C. K., 1983), Figure 1.4 indicates flexural-compressional brittle failures on shear wall.

Figure 1.4. Flexural-compressional brittle failures on shear walls
(Carlos Videla, 2012)

1.4.1.2. Concrete crushing and rebar buckling:

These are two types of phenomena observed in columns when axial loads are applied in the downward direction. Crushing means breaking and failure of short columns and when subjected to high compressive stress and buckling is the failure of long column structure when subjected to high buckling stress. Figure 1.5 indicates concrete crushing and rebar buckling on a column element.

Figure 1.5. Concrete crushing and rebar buckling on a column element. (Carlos Videla, 2012)

1.4.1.3. Overall buckling of thin walls

Overall buckling is characterised by a distorted or buckled, longitudinal axis of the member. It is mostly observed in thin walls. Figure 1.6 indicates overall buckling of thin walls of an RC building.

Figure 1.6. Overall buckling of thin RC building walls (Carlos Videla, 2012)

1.4.1.4. Splice failures

Reinforced concrete structures are designed to behave monolithically. Properly designed splices of individual reinforcing bars are a key element in transmitting forces through the structure and creating a load path. A lap splice is the predominant method used for splicing reinforcement bars. Failure to slice correctly the bars leads to splice failure. Figure 1.7 indicates splice failure on a reinforced concrete building.

Figure 1.7. Splice failure on an RC building
(Carlos Videla, 2012)

1.4.1.5. Soft floor (irregularities)

This refers to one level of a building that is significantly more flexible or weak in lateral load resistance than the stories above it and the floors or foundation below it. This condition can occur in any construction type and is typically associated with large openings in the walls or exceptionally tall story height in comparison to the adjacent stories. These soft stories can present a very serious risk in the event of an earthquake, both in human safety and financial liability. Figure 1.8 indicates soft floor failure on a reinforced concrete building.

Figure 1.8. Soft floor failure on a reinforced concrete building.
(Carlos Videla, 2012)

1.4.1.6. Lack of concrete confinement bars

On the first storey column and shear walls of multi-storey buildings, heavy damages mostly occur because of a lack of sufficient transverse reinforcement. This failure also occurs due to insufficient confinement of concrete at beam-column joint. Figure 1.9 indicates failure on a shear wall due to lack of concrete confinement bars.

Figure 1.9. Failure of shear wall due to lack of confinement bars
(Carlos Videla, 2012)

1.4.1.7. Deficient reinforcement detailing

Reinforcement detailing is the drawing of a reinforced concrete structure, which includes showing the size, location, type, placement, splices, and termination of the reinforcement. Deficiency in the reinforcement detailing can cause structural failures. Figure 1.10 indicates failure of RC column due to poor detailing.

Figure 1.10. Failure of RC column due to poor detailing.
(Carlos Videla, 2012)

1.4.1.8. Shear failure

Shear failure occurs when the beam has shear resistance lower than flexural strength and the shear force exceeds the shear capacity of different materials of the beam. A shear load is a force that tends to produce a sliding failure on a material along a plane that is parallel to the direction of the force. Since shear failure is usually sudden with little or no advanced warning, the design for shear must ensure that the shear strength for every member in the structure exceeds the flexural strength. Providing proper shear reinforcement along the beam will reduce the possibility of shear failure along the beam. Figure 1.11 indicates shear failure of an RC wall, (*Madeh Izat Hamakareem , 2001)*

Figure 1.11. Shear failure of an RC wall
(Carlos Videla, 2012)

1.4.1.9. Walls too slender

A wall whose H/t ratio is greater than 30 can be considered to be a slender wall. Compression tests on small walls elements indicate that these walls without any cross ties or boundary zones reinforcement may have a compression strain as low as 0.001.Concrete crushing occurs very suddenly with little or no prior damage. Providing nominal cross ties like those provided in gravity-load columns transform the failure mode to a more gradual one and increases the compression strain capacity of concrete walls to traditional assumed value of 0.003 for unconfined concrete. Figure 1.12 indicates failure of a RC shear wall due to the slenderness.

Figure 1.12. Failure of a Reinforced concrete shear wall due to slenderness (Carlos Videla, 2012)

1.4.1.10. Heavy loaded walls

This is the inability of the wall to support a designed structural load without breaking. This means that the material that makes up the wall is stressed beyond its strength limit causing fracture or excessive deformation. Figure 1.13 indicates failure of a RC shear wall due to heavy loads. (Jones D.R.H, 2001).

Figure 1.13. Indicate failure of a RC shear wall due to heavy loads (Carlos Videla, 2012)

1.4.2. Geotechnical failures

There are several causes of geotechnical failures. Many disasters are related to geotechnical failures. Amongst the many causes of geotechnical failures, we can mention the following: an exceptional load and an exceptionally low strength or a combination of the two in the foreseen failure mechanism, calculation errors from a well-qualified engineer, unknown or unforeseen failure mechanisms or lack of scientific knowledge and lastly the lack of available knowledge or willingness at the designing stage. According to (Khan, 2005), geotechnical failures can result from foundation factors such as change in water table, progressive soil excavation activities, burrowing by animals, liquefaction of soil, and natural disasters such as earthquakes and landslides. (Roy E. Hunt, 2007),

1.4.2.1. General shear failure

This involves total rupture of the underlying soil. There is a continuous shear failure of the soil from below the footing to the ground surface. When the load is plotted versus the settlement of the footing, there is a distinct load at which the foundation fails, and there is a designated Q_{ult} .The value of Q_{ult} divided by the width B and the length L of the footing is considered to be the ultimate bearing capacity (q_{ult}) of the footing. The ultimate bearing capacity has been defined as the stress that causes a sudden catastrophic failure of the foundation. Figure 1.14 shows that a general failure ruptures and pushes up the soil on both sides of the footing. For actual failure in the field, the soil is often pushed up on only one

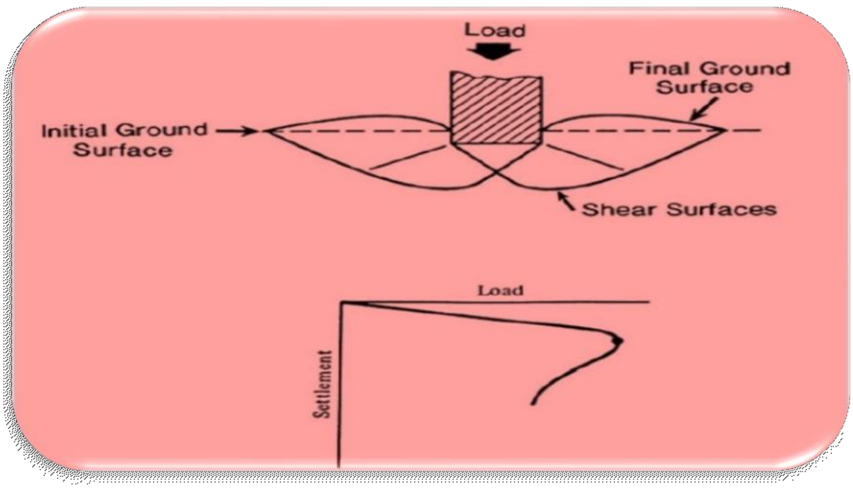


Figure 1.14. General shear failure
(Source: civilblog.org)

1.4.2.2. Local shear failure

Local shear failure involves rupture of the soil only immediately below the footing. There is soil bulging on both sides of the footing, but the bulging is not as significant as in general shear failure. Local shear can be considered as transitional between general shear and punching shear. Because of the transitional nature of local shear failure, the bearing capacity could be defined as the first major nonlinearity in the load-settlement curve (open circle) or at the point where the settlement rapidly increases (solid circle). A local shear occurs for soils that are in a medium dense or firm state. Figure 1.15 shows local shear failure.

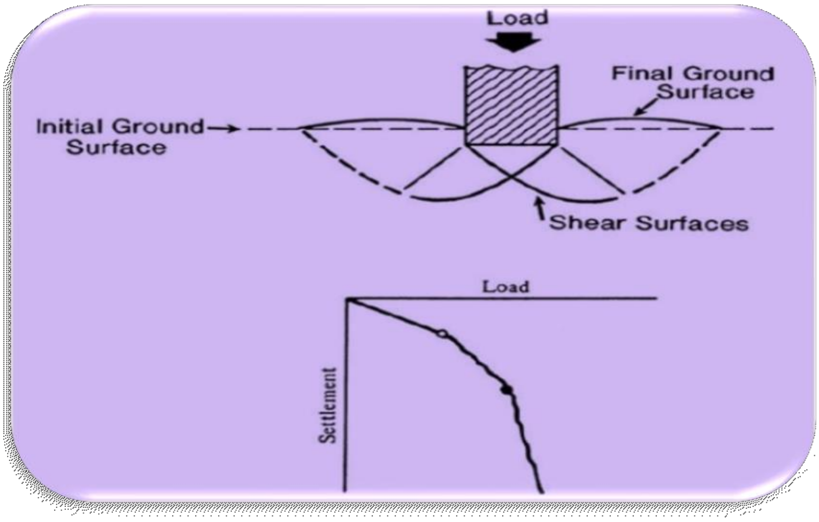


Figure1.15. Local shear failure
(Source: civilblog.org)

1.4.2.3. Punching shear failure

As shown on Figure 1.7, a punching shear failure does not develop the distinct shear surfaces associated with general shear failure .For punching shear, the soil outside the loaded area remains relatively uninvolved and there is minimal movement of soil on both sides of the footing. The process of deformation of the footing involves compression of soil directly below the footing as well as the vertical shearing of soil around the footing perimeter. The load-settlement curve in Figure 1.16 does not have a dramatic break and for punching shear, the bearing capacity is often defined as the first major non linearity in the load – settlement curve (open circle). A punching failure occurs for soils that are in a loose or soft state.

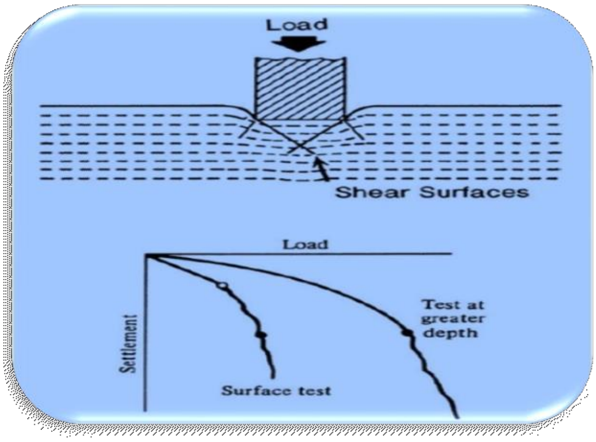


Figure 1.16. Punching shear failure
(Source: civilblog.org)

1.4.2.4. Slope stability failures

The process of sliding down large soil mass along a plane or a curved surface with respect to the remaining mass is known as slope failure. There are different types of slope failure which are translational failure, rotational failure, wedge failure and compound failure. Generally, slope failure occurs mainly for two reasons: firstly, an increase in shear force due to an increase in the slope of the soil mass or due to a sudden dynamic force applied to soil mass like an earthquake or due an external load applied and secondly a decrease in shear strength of soil which can happen due to an increase in pore water pressure. Fig 1.17 illustrates such failure.

Figure 1.17. Slope stabilty failure
(Source: civilblog.org)

1.5. Structural remodelling

Structural remodelling refers to residential remodelling that involves fixing, changing, removing, or adding any load bearing elements. A load could either refer to weight or pressure. These elements could include posts, beams, columns, and of course, the home's walls and foundation. Home owners may consider a structural remodel for many reasons. Sometimes, people need to repair or maintain their homes because of damage caused by a storm or aging. Very often, the homeowners simply want to make their homes more comfortable and valuable by adding windows, changing the home's design, or increasing living space. (Dan Bawden, 2019).

1.6. Forensic engineering

Forensic engineering could be considered as a fact-finding expertise for identifying responsibilities-related failures. (Noon R. K., 2000). It is the application of engineering sciences to the investigations of failures and/or performance issues. A Forensic Engineering Structural framework enables engineers in conducting forensic investigations for buildings. (Ratay R. T., 2017). An effective forensic framework should be simple and straightforward, represent all causes of failure in reinforced concrete structures and include corresponding legal responsibilities. The suggested forensic engineering structural framework comprises 5 stages: Preliminary stage, Evidence Collection Stage, Failure Hypothesis and Analysis Stage, Conclusion Stage and Responsibility assigning stage (Brown E, 2006)

1.6.1. Preliminary stage

During the preliminary stage, the necessary information and data related to the building is collected and all related documents are reviewed. Further, the preliminary stage includes setting the plan of the investigation of the failure.

1.6.2. Evidence collection stage

The second stage comprises collection of evidences. The investigators should conduct site visits as early as possible to eliminate any disturbance to the evidence. In turn, the site visit involves three components, as shown in Figure 1.18, namely visual inspection, eyewitness information and sample collection. Efficient visual inspection and availability of possible eyewitness' information would ease the process of collection of the samples. Through visual inspection, investigators are able to observe the failure scene, thereby providing the main evidence that may report about how the failure occurred. On the other hand, investigators while communicating to eyewitnesses on site seek to understand the actual modes and sequences of failure because eyewitnesses would often provide valuable evidence to investigators. Collecting samples relevant to the failure is also a significant step because it may reveal important evidence. The data collectively obtained at the site visit may shed light on the initial failure hypothesis to be examined at the third stage.

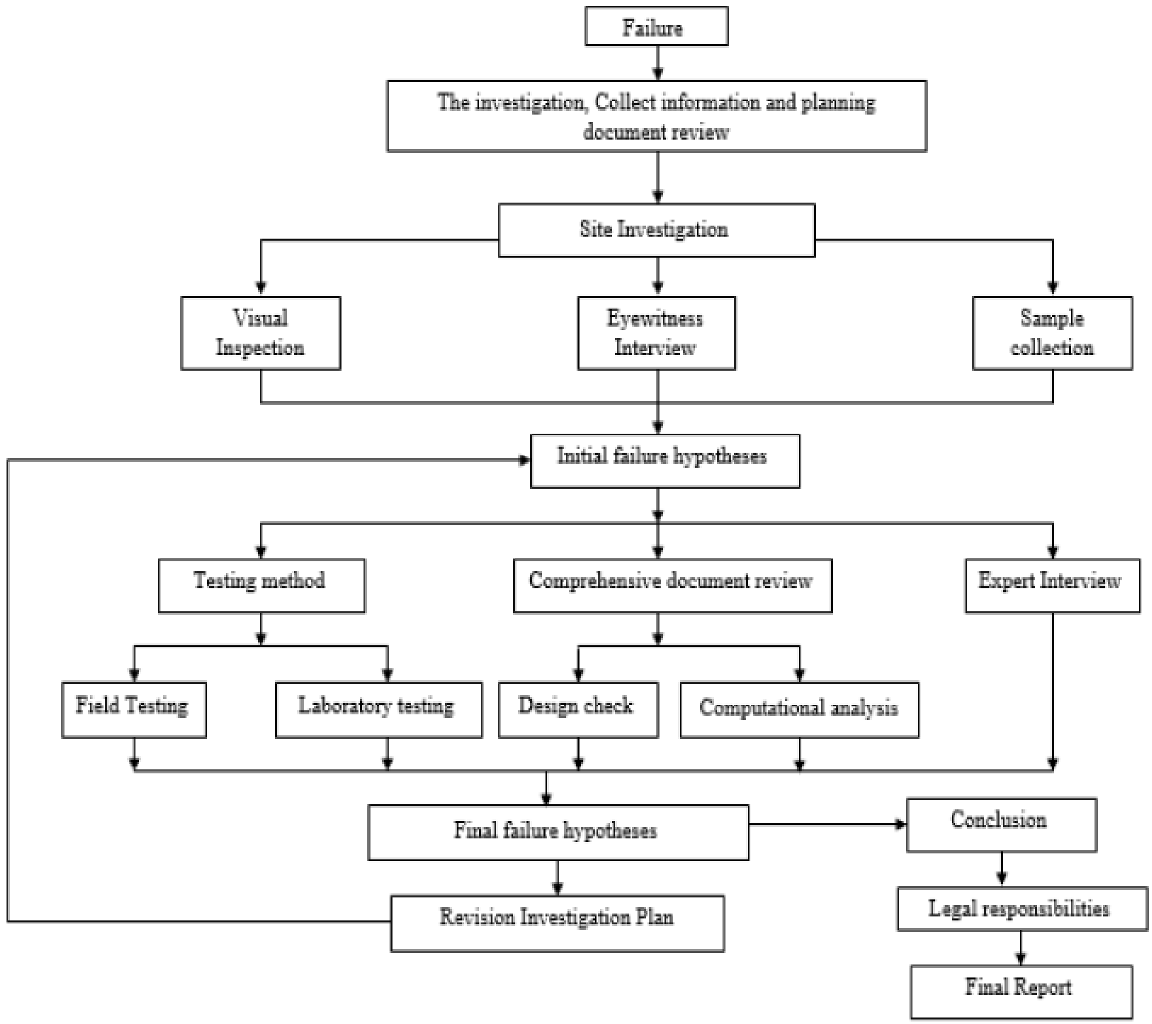


Figure 1.18. A developed forensic engineering structural framework
(Source: www.erpublication.org)

1.6.3. Failure hypothesis and analysis stage

The third stage, failure hypotheses and their analysis, discusses and approves the data obtained previously. It comprises three approaches: carrying out testing methods, a critical review of relevant documents and, lastly, conducting depth interviews, as shown in figure 1.18. The testing methods are categorized as field and laboratory assessments, involving a series of non-destructive and destructive that will be carried out on site. The key purpose is to check the actual mechanism of concrete structure. Laboratory, on the other hand, involves specific tests that are commonly destructive in an attempt to examine capacity and mechanism of certain components of concrete structure. It may also involve chemical analysis, loading tests and other associated testing.

The review of documents involves also 'Design check' and 'computational analyses. The former includes the review of relevant documents related to the failure. By reviewing the documents, the investigators will be more familiar with the case and any discrepancies that will be detected. Computational analysis is a recommended procedure using relevant software packages to analyse the concrete structure. A supplementary approach is therefore adopted in an effort to prove the 'failure hypotheses, for example using semi-structured interviews. In addition, an expert's opinion may also help prove the 'failure hypothesis', hence offering valuable explanations to the investigators towards understanding the cause of the failure. Upon completion of all analysis, work could be undertaken to test the failure hypotheses.

1.6.4. Conclusion stage

The fourth stage is the conclusion stage in which specific interpretations are drawn, namely from the findings derived from the evidences obtained which in turn lay the template for the causes of failure.

1.6.5. Responsibilities assigning stage

The final stage is the responsibilities assigning stage during which the major and minor responsibilities are assigned to the relevant parties, i.e., the contractor, engineers and owner. It is recommended that specific civil responsibilities law, local or international, should be considered during this stage. For example, the Egyptian law states that the major responsibility of failures and/or errors occurred in the design are assigned at the designer. However, during construction, major responsibility is assigned at the contractor and minor responsibility is assigned at supervision engineer. (Kardon J. B. ,2012)

Conclusion

The main objective of this chapter was to propose a literature review surrounding the key words of the topic: developing a forensic engineering structural framework for the structural remodelling of reinforced concrete structures. Firstly, concrete as material was defined followed by the notion of reinforced concrete. Furthermore, the notion of reinforced concrete structures was explained and structural failures on reinforced concrete structures illustrated. Also, structural modelling was explained briefly followed by a detailed explanation of forensic engineering and the forensic engineering structural framework. What can be drawn from this chapter is that the structural remodelling requires a forensic engineering structural framework in order to illustrate and resolve the failure hypothesis that may lead to the collapse of the structure.

CHAPTER 2: METHODOLOGY

Introduction

In order to move out of generality, this work was applied to a specific case study that is an R+1 industrial building in the locality of Nsimalen, precisely at Maetur Nkolnda used for agricultural exploitation, beverage production and the hosting of employees in the centre region of Cameroon. This chapter will show the methodology used following a forensic engineering structural framework for structural remodelling of the reinforced concrete industrial building to avoid failures. Firstly, the project site will be recognised. Secondly the project data will be collected that is architectural data, material characteristics, geotechnical data and structural data. Furthermore, the finite element modelling of the R+1 industrial building will be described, followed by calculation hypothesis that permit the dynamic and static analysis procedures using CSI Sap 2000 software. Finally, failure will be characterised in order to conclude on the possibility of remodelling or not.

2.1. General site recognition

The recognition of the site was done from documentary research whose essential objective is to know the physical parameters of the site of the case study that is the location of the site, the climate, the relief, the hydrology and the economic activities of the site.

2.2. Site visit

During the site visit some pictures of the project site were taken using a phone camera. The activity on the building and surrounding area was obtained by questioning the surrounding population. Some of the visible structural elements were inspected by eye inspection and measured using a simple meter.

2.2.1. Observation

The building's superstructure was observed followed by the relief of the site and the surrounding environment. Observation pictures were taken using a phone camera. Furthermore, the structural constitution that is the steel positioning in the pillars and beams were observed and noted.

2.2.2. Questionnaire

A questionnaire was addressed to the direct project impact zone that is the population around the new exchanger on the Nsimalen highway, 200m from Maetur Nkolnda concerning the climate in the area and the economic activities. The proprietor of the project was also questioned concerning the structural remodelling of the building and the utility such an investment.

2.3. Data collection

The data collected for the investigation were regrouped into 4 categories which are: architectural data, material characteristics, geotechnical data and structural data.

2.3.1. Architectural data

The architectural data collected from the construction company realising the work include:

- The foundation plans
- The distribution plan of the ground floor
- The distribution plan of the floor 1
- The 3d model before remodelling
- The distribution plan of the floor 2 after remodelling.
- The roof plan of the added floor.
- 3d render after remodelling

2.3.2. Material characteristics

The materials characteristics for the project include:

- Longitudinal reinforcement steel traction resistance.
- Confinement bars traction resistance
- Concrete compressional resistance.
- Concrete dosage
- Concrete cover

2.3.3. Geotechnical data

The geotechnical data collected for the investigation include:

- Allowable bearing capacity of soil.
- Anchor depth of footings
- Foundation type

2.3.4. Structural data

The structural data collected for the investigation include:

- Beams reinforcement distribution
- Pillar reinforcement distribution
- Footing's reinforcement distribution
- Slab span and reinforcement

2.4. Physical modelling of the building.

The industrial building's physical modelling was done using CSI Sap 2000. The superstructure consists of beams, and pillars. The substructure consists of strip footings. Firstly, the superstructure and substructure materials were defined to match with the physical model materials consisting of steel (Reinforcement bars) and concrete. Secondly, the beams and columns were defined using frame sections. The footings were modelled using the shell elements, links, frame elements and assigned thickness. Thirdly, the defined materials (steel, concrete, rebar) were assigned for each frame or shell element following the correspondence. The grid was defined to facilitate structural modelling and the structure modelled using the frame sections, shell elements and links to represents the soil-structure interaction. The boundary conditions were initiated, that is the application of diaphragm constrains on the slabs to represent the actual characteristics of support and continuity. The mass source was defined for the consideration of variable loads during modal analysis.

2.5. Calculation parameters

The necessary aspects taken into consideration during the structural remodelling verification using the forensic engineering framework are the codes and norms, the load patterns, load combinations and failure type.

2.5.1. Codes and norms

The codes used for the verification of the building's superstructure and substructure are Eurocodes 1, 2 and 7.

2.5.2. Load patterns

The load patterns assumed for the verification of the remodelled industrial building are the permanent and the variable loads derived from Eurocode 1. Table 2.1 illustrates the permanent load patterns used for the verification.

Table 2.1. Permanent load patterns used for the building verification

Permanent loads	**Value**
Slab	2.50 kN/m^2
Partition wall	1.20 kN/m^2
Screed	0.80 kN/m^2
Tiles	0.60 kN/m^2
Coated under slab (1.5cm thickness)	0.38 kN/m^2
Total	**5.48 kN/m^2**

The variable load pattern depends on the category of use of the building. Table 2.2 indicates the category, specific use and example of buildings.

Table 2.2. Building categories for variable load patterns evaluation

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, cafes, 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.

(Source: EN1991-1-1_E_2002, Table 6.1, page 21)

The load pattern values following the building category are defined in the table 2.3.

Table 2.3. Variable load pattern values used to follow the building category

Table 6.2 – Imposed loads on floors, balconies and stairs in buildings		
Categories of loaded areas	**q_k [kN/m^2]**	**Q_k [kN]**
Category A		
- Floors	1,5 to 2,0	2,0 to 3,0
- Stairs	2,0 to 4,0	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

(Source : EN1991-1-1_E_2002, Table 6.2, page 21)

The building is a C4 category with the last floor used for dancing and sporting activities. A variable load value (q_k) of 4.75kN/m^2 was taken for the verification.

2.5.3. Load combinations

Manual load combinations were used for the static analysis of load bearing elements of the structure at ultimate limit state and serviceability limit state to simulate loading cases of the industrial building.

2.5.3.1. Ultimate Limit State load combinations

ULS Load combinations were done following the slabs span and the static scheme. The mathematical model used for the combinations at ultimate limit states is defined by equation 2.1.

$$\sum 1.3G_1T_t + 1.3\ G_2T_t + 1.5Q_kT_t \qquad (2.1)$$

Where:

G_1T_t represents structural permanent loads on the span t

G_2T_t represents non-structural permanent loads on the span t

Q_kT_t represents variable loads on the span t

10 load combinations plus the envelop combination were defined applying the mathematical model on the SAP 2000 software respecting the directions of the slab span and influence areas for the application of loads to verify the main beams as illustrated by figure 2.1**.**

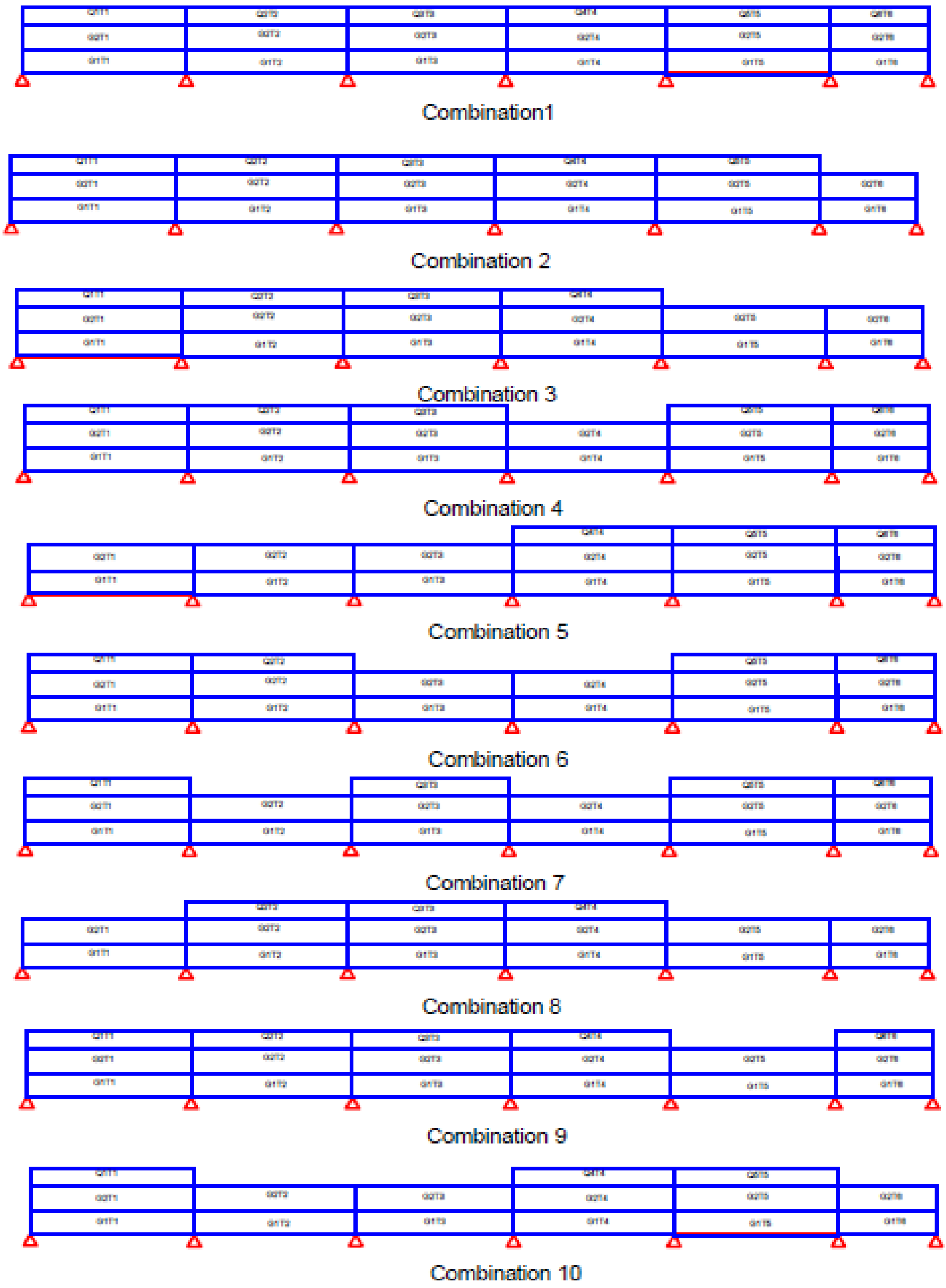


Figure 2.1. Load combinations used for the loads lowering on the structure.

2.5.3.2. Serviceability Limit State load combinations

SLS Load combinations were done following the slabs span and the static scheme. The mathematical model used for the combinations at serviceability limit states is defined by equation 2.2.

$$\sum G_1 T_t + G_2 T_t + Q_k T_t \qquad (2.2)$$

Where:

$G_{1,t}$ represents structural permanent loads on the span t

$G_{2,t}$ represents non-structural permanent loads on the span t

$Q_{k,t}$ represents non-structural variable loads on the span t

10 load combinations plus the envelop combination were defined applying the mathematical model (equation 2.2) on the SAP 2000 software respecting the directions of the slab span and influence areas for the application of loads to verify the main beams.

2.6. Structural Analysis

The methods of structural analysis performed for the investigation of failures during the structural remodelling of the Nsimalen R+1 industrial building are the dynamic modal analysis and the static structural analysis.

2.6.1. Dynamic modal analysis

Dynamics analysis is a type of structural analysis which covers the behaviour of a structure subjected to dynamic loading (actions having acceleration). Dynamic analysis applied to structures reflects the latest application of the structural dynamics theory to produce more optimal and economical structural designs. Dynamic modal analysis is used to control (reduce or avoid) the vibration of a structure when submitted to an effort. After physical modelling of the industrial building, a modal analysis was run in order to calibrate the numerical model. Numerical calibration makes the finite element model behaviour and response to match the physical model behaviour and response.

2.6.2. Static structural analysis

Static structural analysis determines the displacements, stresses, strains, forces, support reactions and stabilities in structures or components caused by loads that do not induce significant inertia and damping effects. The results of the analysis are used to verify the structure's fitness for use, often precluding physical tests. Structural analysis is thus a key part of the engineering design of structures, (C. K. Wang, 1983). Structural analysis was done for load bearing elements such as beams, columns and footings using CSI SAP 2000 software and hand calculation.

2.6.2.1. Beams

The beams were verified at ultimate limit state. The ultimate limit state is an agreed computational condition that must be fulfilled among other additional criteria, in order to comply with engineering demands for strength and stability under design loads. The verification performed on the beam elements at ultimate limit state are the reinforcement verifications, moment verification, shear verification.

a. Reinforcement verification

Knowing the solicitation curve, the steel reinforcement is computed for a rectangular section with the height h, the width b and the effective depth d as defined by the formulas below. Figure 2.2 illustrates the example of a rectangular section used to illustrate the calculation.

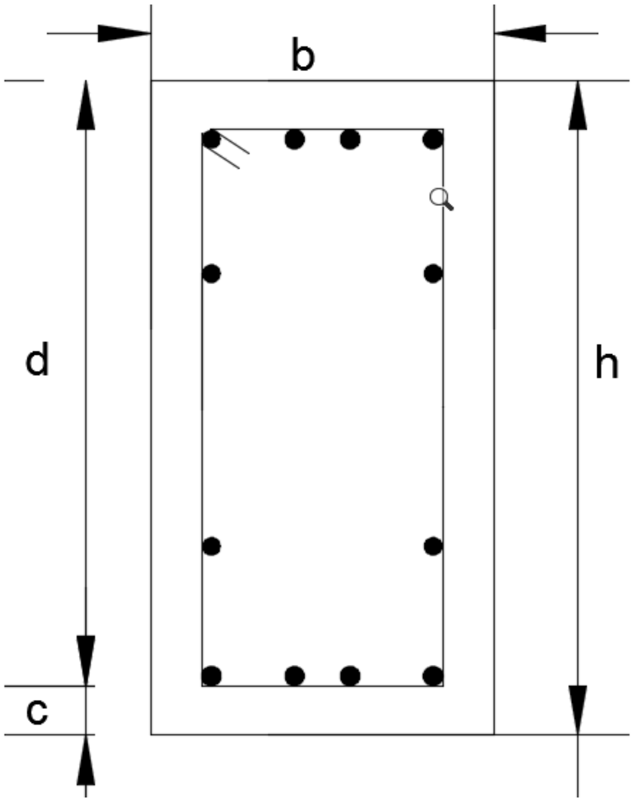


Figure 2.2. Transversal beam section with longitudinal reinforcement

The section of steel at each point of the beams is estimated using the formula defined by equation 2.3.

$$A_s = \frac{M_{Ed}}{0.9d.f_{yd}} \quad (2.3)$$

The section obtained has to verify the detailing of beams prescribed by the Eurocode2 which defines the minimum and the maximum reinforcement areas by the equation 2.4 and 2.5 respectively.

$$A_{S,min} = \max(0.26\frac{f_{ctm}}{f_{yk}}b_t d; 0.0013 b_t d) \quad (2.4)$$

$$A_{S,max} = 0.004\, A_c \quad (2.5)$$

Where:

b_t is the mean width of the tension zone

d is the effective depth of the section

f_{ctm} *is* the tensile strength of the concrete

b. Moment verifications

Having defined the steel reinforcement section, the effective area of the steel reinforcement is obtained by computing the number of bars necessary and the corresponding area. The moment verification of the section is done by calculating the resisting bending moment using the position of the neutral axis in the section. Figure 2.3 illustrates the neutral axis and the different parts (tension or compression).

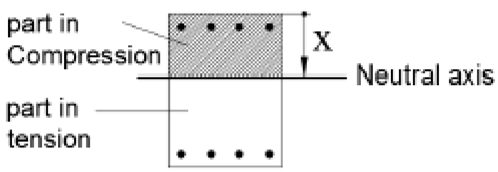


Figure 2.3. Neutral axis position in the section

The neutral axis is obtained from equation 2.6.

$$x = \frac{d}{2\beta_2} - \sqrt{\left(\frac{d}{2\beta_2}\right)^2 - \frac{M_{Ed}}{\beta_1\beta_2.b.f_{cd}}} \quad (2.6)$$

Where:

d is the effective depth of the section

b is the width of the section

f_{cd} is the design compressive strength of the concrete

β_1 is the correction factor equal to 0.81

β_2 is the correction factor equal to 0.41

The resisting moment is calculated by the relation defined by equation 2.7.

$$M_{Rd} = A_{s_{real}} . f_{yd} . (d - \beta_2 . x) \quad (2.7)$$

Where:

As_{real} is the effective area of the steel sections

f_{yd} is the design yielding strength of the steel

For the moment verification the action moment M_{ed} resulting from the envelop moment diagram has to be less than the resisting bending moment as defined by equation 2.8.

$$M_{Rd} \geq M_{ed} \quad (2.8)$$

a. Shear verification

Vertical action on the beam produces shear and in order to withstand the shear forces the beam has to be verified for shear. The transversal reinforcement on a beam is illustrated by figure 2.4.

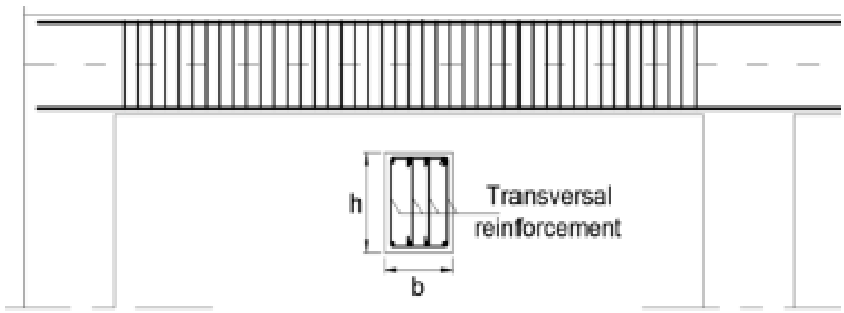


Figure 2.4. Longitudinal and transversal beam section illustrating transversal reinforcement

From the envelope curve of the shear solicitation, the necessity of the shear reinforcement is verified by comparing the acting shear V_{Ed} to the allowable shear of the member without shear reinforcement $V_{Rd,C}$ which is defined by equation 2.8.

$$V_{Rd,C} = \max\left\{\left[C_{Rd,c}k(100\rho_l f_{ck})^{\frac{1}{3}} + k_1\sigma_{cp}\right]b_w d;\ (V_{min} + k_1\sigma_{cp})b_w d\right\} \qquad (2.8)$$

Where:

f_{ck} is the characteristic strength of the reinforcement

d is the effective depth of the section

b_w is the smallest width of the cross section in the tensile area

σ_{cp} $=\frac{N_{Ed}}{A_c} < 0.2f_{cd}$ $[N/mm^2]$]

N_{ed} is the axial force in the cross section due to loading or pre-stressing

A_c is the area of the concrete cross section

$k = 1 + \sqrt{\frac{200}{d}} \leq 2.0$ With d in mm

$\rho_l = \frac{A_{sl}}{b_w d} \leq 0.02$

If no design shear reinforcement is required, the minimum shear reinforcement is applied according to the detailing of that member.

For members where the design shear reinforcement is required, the shear resistance is the minimum of V_{rds} and V_{rdmax} defined by the equations 2.9 and 2.10.

$$V_{Rd,max} = \alpha_{cw}b_w z v_1 f_{cd}/(cot\theta + tan\theta) \qquad (2.9)$$

$$V_{Rd,S} = \frac{A_{sw}}{S} z f_{ywd} cot\theta \qquad (2.10)$$

Where:

f_{ywd} is the design yield strength of the shear reinforcement

v_1 is a reduction factor for concrete cracked in shear

α_{cw} is a coefficient taking account of the state of stress in the compression cord

S is the spacing of the stirrups

A_{sw} is the cross-sectional area of the shear reinforcement with a maximum value given by the relation defined by equation 2.11.

$$\frac{A_{sw,max} f_{ywd}}{b_w S} \leq \frac{1}{2} \alpha_{cw} v_1 f_{cd} \tag{2.11}$$

The design shear reinforcement obtained must verify the detailing of members. In case of the beam, it defines the maximum longitudinal spacing of the shear assembly, maximum transversal spacing of the legs is a series of shear links and the minimum shear reinforcement ratio as shown by the figure 2.5.

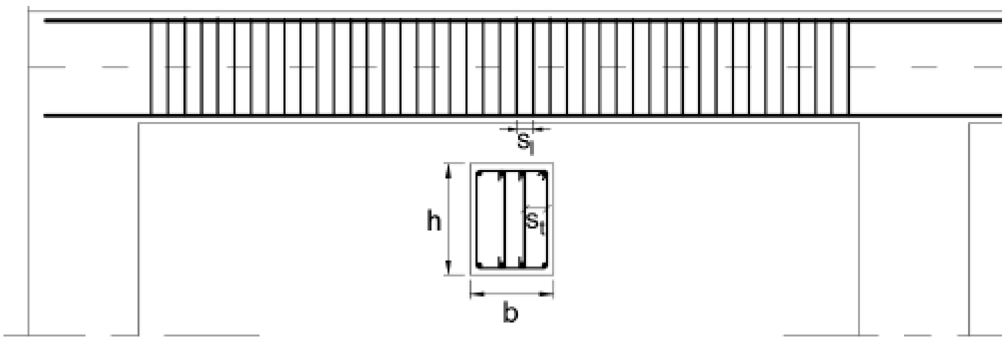


Figure 2.5. Illustration of the maximum longitudinal spacing and maximum transversal spacing.

The limitation values are calculated by equation 2.12 and 2.13.

$$s_{l,max} = 0.75d(1 + cot\alpha) \tag{2.12}$$

$$s_{t,max} = 0.75d \leq 600mm \tag{2.13}$$

With the shear reinforcement ratio equal to: $\rho_w = A_{sw} / (s.\, bw.\, sin\alpha)$

2.6.2.2. Columns

The verifications performed on the columns are reinforcement verification, followed by moment-axial force, shear verification and slenderness verifications.

a. Reinforcement verification

The steel reinforcement of the column is considered taking into account the limits of the Eurocode 2 that are defined by equation 2.14 and 2.15.

$$A_{s,min} = \max(\frac{0.10N_{Ed}}{f_{yd}}; 0.002A_c) \qquad (2.14)$$

$$A_{s,max} = 0.04A_c \qquad (2.15)$$

Where:

N_{Ed} is the design axial compression force

f_{yd} is the design yield strength of the longitudinal reinforcement

b. Moment-axial force verification

After obtaining the envelope of the bending moment and the axial force solicitations, the design is done through the M-N interaction diagram. For each level, the maximum M-N solicitation should belong to the M-N interaction diagram of the section considered. The interaction diagram is plotted following 6 points as defined in the following section.

i. First point

The section is completely subjected to tension; hence, the concrete is not reacting. We impose $\varepsilon_s = \varepsilon_{su}$, $\varepsilon_{s'} = \varepsilon_{syd}$ then the stress inside the element corresponds to the design yielding strength of the steel reinforcement and the limit axial force and bending moment are obtained from the equations 2.16 and 2.17.

$$N_{Rd} = f_{yd}.A_s + f_{yd}.A_s' \qquad (2.16)$$

$$M_{Rd} = f_{yd}.A_s.\left(\frac{h}{2} - d'\right) - f_{yd}.A_s'\left(\frac{h}{2} - d'\right) \qquad (2.17)$$

ii. Second point

The section is completely subjected to tension. The condition impose is $\varepsilon_s = \varepsilon_{su}$, $\varepsilon_c = 0$. We should verify if the upper steel is yielded or not by determining the strain $\varepsilon_{s'}$. The limit axial force and bending moment are obtained from the equations 2.17 and 2.18.

iii. Third point

The condition impose is that the failure is due to concrete and the lower reinforcement is yielded. It is assumed that $\varepsilon_s \geq \varepsilon_{syd}$, $\varepsilon_c = \varepsilon_{cu2}$ and the neutral axis position is determined. Furthermore, the evaluation consists in verifying if the upper steel is yielded or not by determining the strain $\varepsilon_{s'}$. To determine the corresponding stress, the limit axial force and bending moment are obtained from the equations 2.19 and 2.20.

$$N_{Rd} = -\beta_1.b.x.f_{cd} + f_{yd}.A_s - f_{yd}.A_s' \tag{2.19}$$

$$M_{Rd} = f_{yd}.A_s'.\left(\frac{h}{2} - d'\right) + f_{yd}.A_s.\left(\frac{h}{2} - d'\right) + \beta_1.b.x.f_{cd}\left(\frac{h}{2} - \beta_2.x\right) \tag{2.20}$$

iv. Fourth point

The condition impose is that the failure is due to concrete and the lower reinforcement reaches exactly $\varepsilon_s = \varepsilon_{syd}$. As for the previous point, we determine the neutral axis position and the strain $\varepsilon_{s'}$. The limit value of the axial force and the bending moment are determined using the equations 2.19 and 2.20.

v. Fifth point

The condition imposed is that the failure is due to concrete and the lower reinforcement reaches exactly $\varepsilon_s = 0$ then the neutral axis position is equal to the effective depth of the section. The limit axial force and bending moment are obtained from the equations 2.21 and 2.22.

$$N_{Rd} = -\beta_1.b.x.f_{cd} - f_{yd}.A_s' \tag{2.21}$$

$$M_{Rd} = f_{yd}.A_s'.\left(\frac{h}{2} - d'\right) + \beta_1.b.d.f_{cd}\left(\frac{h}{2} - \beta_2.x\right) \tag{2.22}$$

vi. Sixth point

The evaluation imposes that the section is uniformly compressed. We assume $\varepsilon_s = \varepsilon_c \geq \varepsilon_{c2}$.

The limit axial force and bending moment are obtained from the equations 2.23 and 2.24.

$$N_{Rd} = -b.h.f_{cd} - f_{yd}.A_s' - f_{yd}.A_s \tag{2.23}$$

$$M_{Rd} = f_{yd}.A_s'.\left(\frac{h}{2} - d'\right) - f_{yd}.A_s.\left(\frac{h}{2} - d'\right) \tag{2.24}$$

An example of M-N diagram is presented in figure 2.6. The blue point represents a couple of solicitation M_{Ed} and N_{Ed} which lies internally to the diagram hence the section is considered safe for those actions.

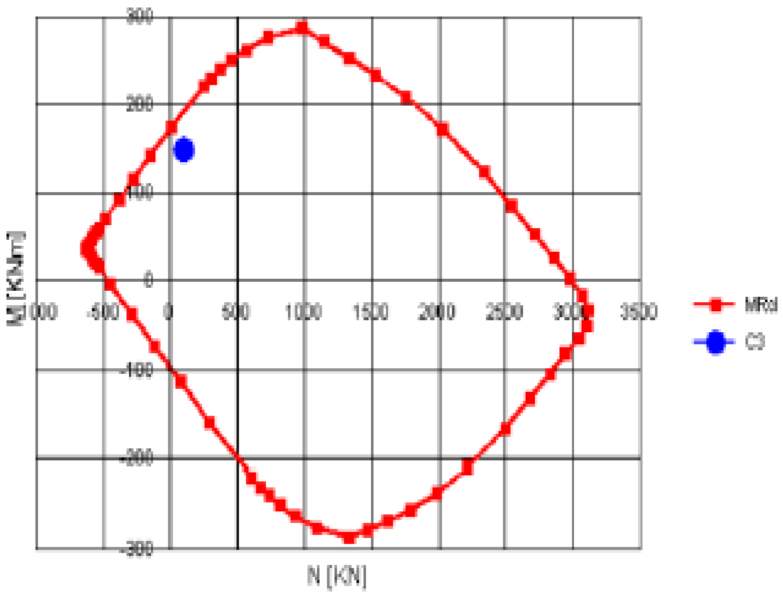


Figure 2.6. Example of M-N diagram (D'Antinio et al, 2016)

c. Shear verification

The verification procedure is the same for the beam. The detailing of members prescribed by the Eurocode 2 imposed a minimum diameter of 6 mm or one quarter the maximum diameter of the longitudinal bars. The maximum spacing of the transverse reinforcement is given by the equation 2.25.

$$S_{cl,max} = \min(20\emptyset_{l,min}; b; 400mm) \tag{2.25}$$

Where:

$\emptyset_l$, is the diameter of the longitudinal bars

b is the smaller dimension of the column

This maximum spacing has to be reduced by a factor 0.6 in sections within a distance equal to the larger dimension of the column cross-section above or below the beam.

d. Slenderness verification

The slenderness verification permits to know if the second order effect should be considered or not. It consists in verifying if the slenderness of the element is below a limit value defined by the Eurocode 2. The limit value is expressed by equation 2.26.

$$\lambda_{lim} = 20.A.B.C/\sqrt{n} \qquad (2.26)$$

Where:

$A = \frac{1}{1+0.2\varphi_{ef}}$ (φ_{ef} is the effective creep ratio; A= 0.7 if φ_{ef} is not known)

$B = \sqrt{1+2\omega}$ ($\omega = A_s f_{yd}/(A_c f_{cd})$: is the mechanical reinforcement ratio;

$C = 1.7 - r_m$ ($r_m = M_{01}/M_{02}$: is the moment ratio; equal to 1 for unbraced system)

n $= N_{Ed}/(A_c f_{cd})$: relative normal force

The slenderness of an element is evaluated by the formula defined by equation 2.27.

$$\lambda = l_o/i \qquad (2.27)$$

Where:

i is the gyration radius of the uncracked concrete section

l_o is the effective length of the element ($l_o = 0.7l$)

The gyration radius of the uncracked section is given by equation 2.28.

$$i = \sqrt{\frac{I}{A}} \qquad (2.28)$$

2.6.2.3. Footings

The verifications were performed for the footing element considering the serviceability state envelop load combination. The verifications performed are the allowable stress verification and reinforcement verification.

a. Allowable soil stress verification

This verification is done by comparing the maximum work stress (σ_{stress}) of the building on the soil to the allowable bearing capacity of the soil (σ_{allow}). The maximum work stress is obtained from the software under the option display then soil pressure while the allowable bearing capacity of soil was obtained from the dynamic penetrometer test on site. The verification is passed when the maximum work stress (σ_s) is less than the allowable bearing capacity of the soil (σ_{adm}) as defined by equation 2.29.

$$\sigma_s < \sigma_{adm} \qquad (2.29)$$

b. Reinforcement verification

For the reinforcement verification the footing reinforcement is obtained and compared to the minimum and maximum values prescribed by the design standards. The reinforcement is obtained in the two directions, firstly in the y-direction, $A_{s,\,y}$ and secondly in the x-direction $A_{s,\,x}$.

2.7. Failure caractérisation

The prediction of failure during the remodelling of the Nsimalen R+1 industrial building was done by analysing what was done on the site and comparing them with the results of the analysis also by assessing structural analysis results from the software Sap 2000. The safety factors resulting from the analysis of load bearing elements were calculated, the deformed shapes presented.

2.7.1. Safety factors

The safety factor resulting from the verification of each load bearing element was calculated, some comments made and the failure evidence highlighted according to the table 2.4.

Table 2.4. Building categories for variable load patterns evaluation

Element	Verification	Actual	units	Theoritical	Safety factors	Valid for remodeling	Failure
Beam	Reinforcement verification						
	Moment verification						
	Shear verification						
Element	Verification	Actual	units	Theoritical	Safety factors	Valid for remodeling	Failure
Column	Reinforcement verification						
	Moment-axial force						
	Shear verification						
	Slenderness verification						
Footing	Allowable stress verification						
	Verification	Actual	units	Theoritical	Safety factors	Valid for remodelng	Failure
	Reinforcement verification						

2.7.2. Deformed shape

The deformed shape was obtained from software by analysis under the different load combinations to identify possible failure modes due to the lowering of loads without excluding the analysis under soil pressure to check the possible failure due to the fact that the allowable bearing capacity verification was not verified.

Conclusion

The main objective of this chapter was to define the work methodology that permits the application of a forensic engineering structural framework for the structural remodelling of a specific reinforced concrete project found in Nsimalen, Yaounde. Firstly, the project site was recognised from documentary research while site visit was made for observations on the structure and a questionnaire for the building proprietor and the surrounding populations. The data collected for the investigation were architectural data, material characteristics, geotechnical data and structural data. Furthermore, physical modelling of the building was done using CSI Sap 2000 software, calculation hypothesis established, dynamic and static structural analysis done using the software CSI SAP 2000 and finally failure was characterised for the prediction of structural instabilities that might cause the collapse of the structure if the remodelling is maintained. What can be drawn from this work methodology is that the numerical simulation for the remodelling of a structure is an important investment and every parameter of the structure needs to be taken into account and verified with an important safety factor to avoid undesirable events. Chapter 3 displays the results of the following work methodology showing the importance of analysing a structure before undertaking any remodelling activity.

CHAPTER 3: PRESENTATION OF RESULTS AND INTERPRETATION

Introduction

In this chapter, The Nsimalen building will be analysed under static loading to assess the safety factors of load bearing elements and foundations for structural remodelling. Firstly, the project site is presented and described. Furthermore, the architectural, material characteristics, geotechnical and structural data are presented followed by the building's physical model. The lowering of loads is presented followed by the dynamic and static analysis results. Finally, failure will be characterised by giving the deformed shapes and safety factors of the structural elements.

3.1. General presentation of project site

After documentary research, the Nsimalen industrial building's site characteristics were obtained such as the location, climate, relief, population, hydrology and economic activities.

3.1.1. Location

The industrial building is found in the Center Region of Cameroon, in Yaounde 3°52'N 11°31'E Altitude-726m**.** The project site is near the exchanger, 200m from Maetur Nkolnda. Nsimalen is found in the Mefou et Akono division of the Center Region of Cameroon. Figure 3.1 shows the Nsimalen industrial building location on the Cameroon map.

Figure 3.1. Nsimalen industrial building project location on the Cameroon map (source: www.cvuc-uccc.com)

3.1.2. Climate

The project site features a tropical wet and dry climate, with constant temperatures throughout the year. However, primarily due to the altitude, temperatures are not quite as hot as one would expect in a city located near the equator. A lengthy wet season is experienced covering a ten-month span between March and November. However, there is a noticeable decrease in precipitation within the wet season, seen during the months of July and August, almost giving the city the appearance of having two separate rainy seasons. **(www.discovercameroon.com)**

3.1.3. Relief

The industrial building is found in the center region of Cameroon precisely at Yaoundé which is a hilly site broken down into 3 topographic units inscribed in bedrock of Precambrian gneiss: the inselberg barrier to the northwest dominated by the Mbam Minkom mountains (1295m).

3.1.4. Hydrology

The area covered by the project, has a fairly high rainfall (1500 to 5000 mm) and a fairly hilly terrain, which allows a good runoff of surface water, increasing the flow of rivers. These waters flow from the mainland to the coastal zone, constituting the "Atlantic basin". The main water collector is the Sanaga river which has its source in the Adamaoua region and empties into the Atlantic Ocean. It is approximately 918 km long. It is influenced by precipitation that starts along its course in April and stops around November, annually. The groundwater regime depends on regional hydrogeology, influenced by the nature of the soil in place.

3.1.5. Socio-economic parameters

The socio-economic parameters described in the following section are the population and the economic activities.

3.1.5.1. Population

The indirect impact zone of the project takes into account the whole center region on an area of 30 400 ha and a population estimated at 4.1 million inhabitants.

3.1.5.2. Economic activities

The main economic activities of the project area are linked to the primary sector (Agriculture, forestry exploitation, breeding), and also the secondary sector and tourism.

a. Agriculture

Local residents engage in urban agriculture. The populations surrounding the project area grow cassava, cocoyam, tomatoes, plantains and maize as main food crops.

b. Breeding

The herd in the project area consist mainly of pigs and chickens. The region is estimated to have 50 000 pigs and over a million chickens.

c. Industry

There are many industries in the region and near the project area. The major industries include tobacco, dairy products, beer, clay, glass goods and timber. The Indirect impact zone that is Yaoundé is a regional distributor centre for coffee, cocoa, copra, sugar cane and rubber.

d. Commerce and services

Services are at the origin of a set of transport flows. The main flows in the project area are: flow of people, flow of manufactured goods, flow of building materials, forest products. Apart from timber, the flow in the area constitutes cassava, tomatoes, sanitary drinks, and beans.

3.2. Physical description of the project site

The industrial building is an R+1 that will be used for the production of fruit drinks, in a 1000 m^2 piece of land. The building has a total length of 30.26m and total width of 13.56m. Initially the building was designed for a ground level and one floor, but with time to exploit the roof floor of the building the proprietor wants to remodel the building by adding a new floor and use the floor for dancing and sporting activities. During the site visit the sections of the load bearing elements were measured and the detailing observed for the elements such as pillars. To carry out the investigation the company in charge of the construction of the industrial building provided the floor plans and the reinforcement of the load bearing elements of the building. Figure 3.2 gives us an image of the building.

Figure 3.2. Nsimalen industrial building

The roof before remodeling was captured using a phone camera as illustrated by the figure 3.3.

Figure 3.3. Industrial building roof plan

3.3. General presentation of project

The data collected are regrouped into 4 categories which are: Architectural data, material characteristics, geotechnical data and structural data.

3.3.1. Architectural data

The architectural data required for the investigation of the industrial building are the foundation plan, The distribution plan of the ground floor, the distribution plan of the floor 1, The roof plan (floor 2) before remodelling, 3d model before remodelling, the distribution plan of the roof floor after remodelling, 3d render after remodelling.

3.3.1.1. Foundation plan

The foundation plan constituted of isolated footings for the industrial building before remodelling as illustrated by figure 3.4.

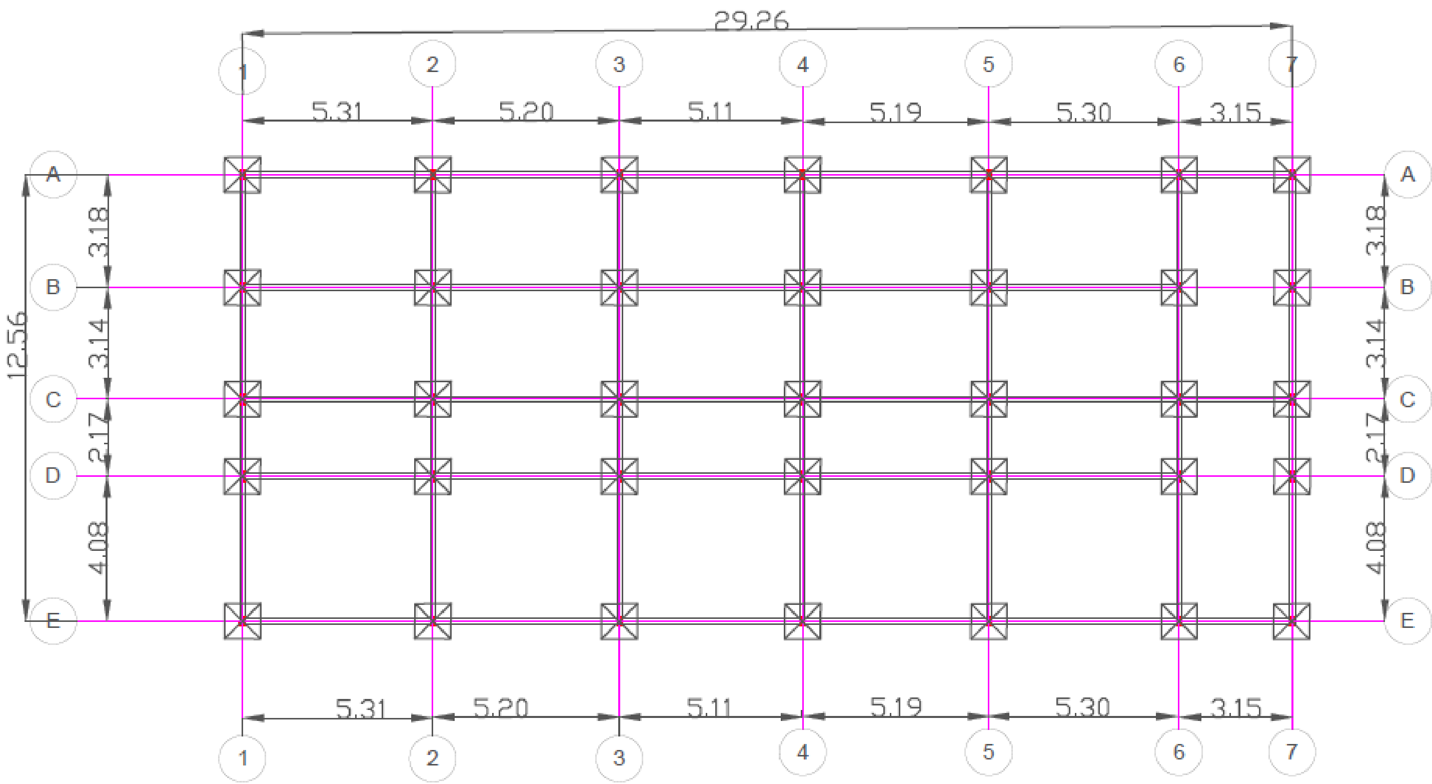


Figure 3.4. Nsimalen industrial building foundation plan

3.3.1.2. Ground floor distribution plan

The Nsimalen industrial building ground floor distribution plan before remodelling is illustrated by figure 3.5.

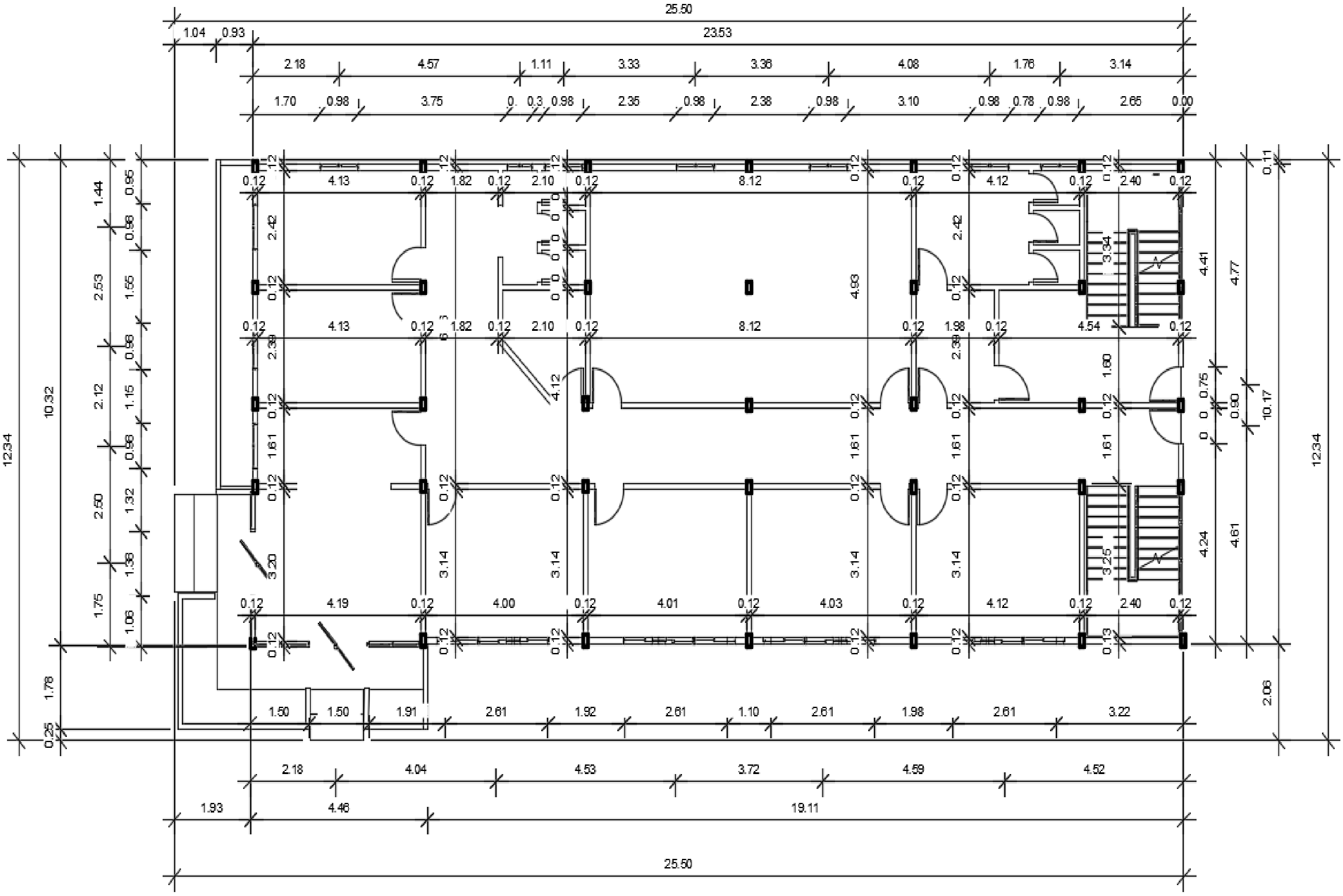


Figure 3.5. Nsimalen industrial building ground floor plan

3.3.1.3. Floor 1 distribution plan

The Nsimalen industrial building floor 1 distribution plan before remodelling is illustrated by figure 3.6.

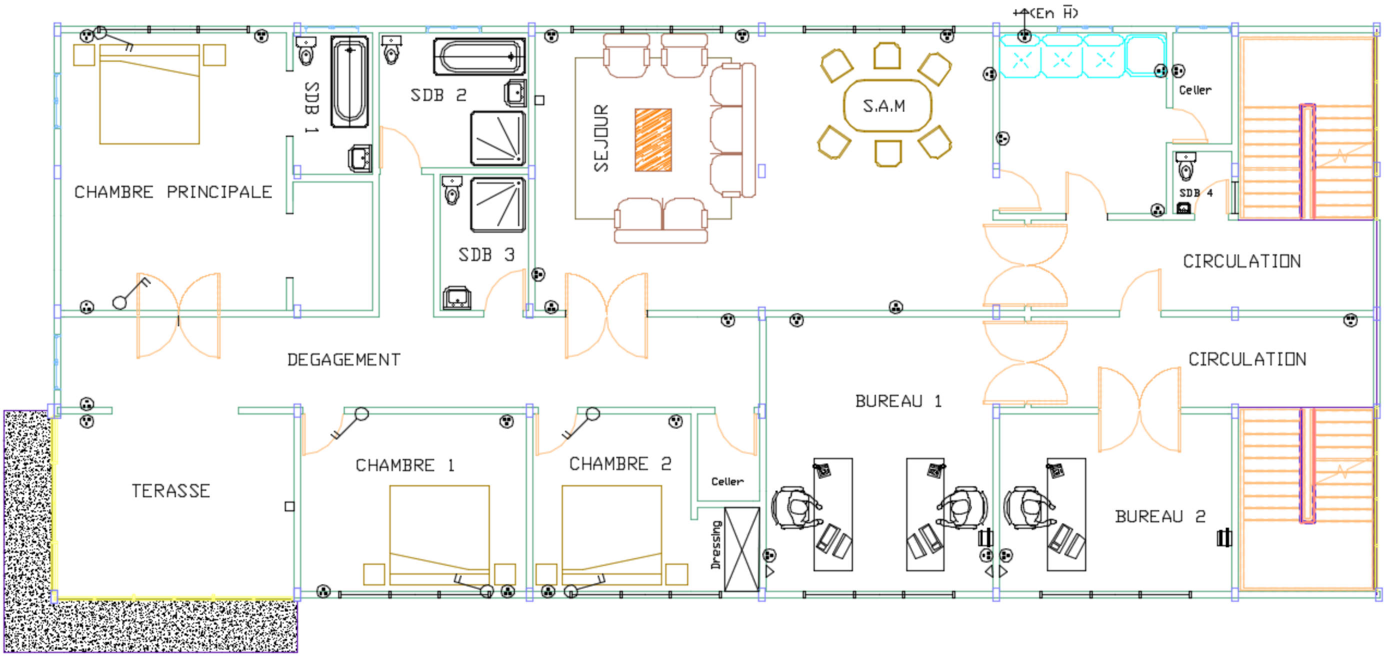


Figure 3.6. Nsimalen industrial building floor 1 distribution plan

3.3.1.4. 3d model before remodelling

The Nsimalen industrial building 3d model before remodelling is illustrated by figure 3.7.

Figure 3.7. Nsimalen industrial building 3d model before remodelling

3.3.1.5. Roof plan after remodelling

The Nsimalen industrial building roof plan after remodelling is illustrated by figure 3.8.

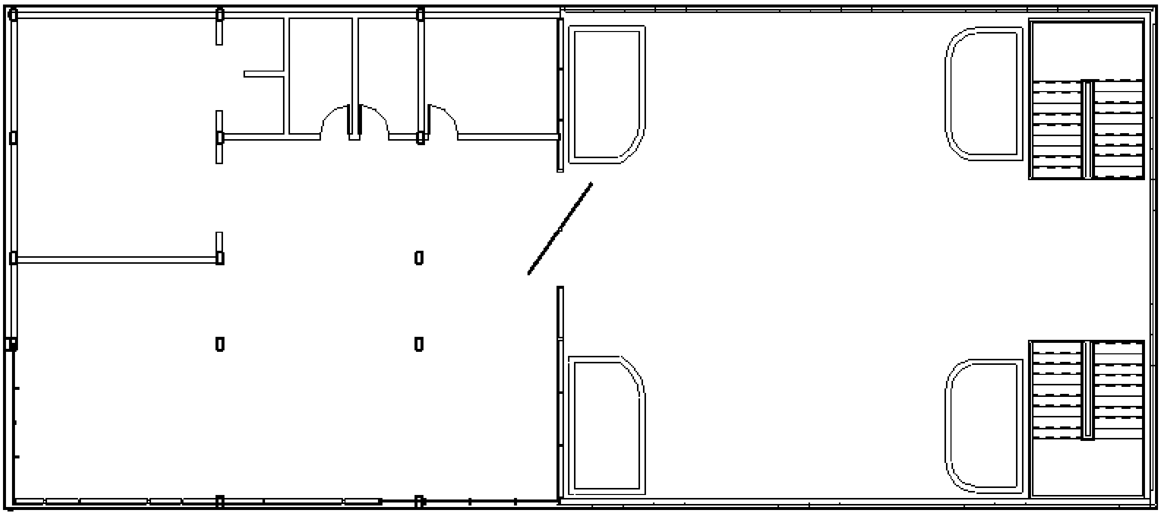


Figure 3.8. Nsimalen roof floor distribution plan after remodelling

3.3.1.6. 3d model after remodelling

The Nsimalen industrial building 3d model after remodelling is illustrated by figure 3.9.

Figure 3.9. Nsimalen industrial building 3d model after remodeling

3.3.2. Material characteristics

The materials characteristics for the project collected are steel reinforcing bars traction resistance, confinement bars traction resistance, concrete class, concrete dosage, concrete cover.

- Steel reinforcing bar: Fe400
- Confinement bars traction resistance RL235
- Concrete class C25/30
- Concrete dosage 350kg/m3
- Concrete cover (Beams and pillars) c = 3cm
- Foundation concrete cover c = 5cm

3.3.2. Geotechnical data

The geotechnical data collected for the investigation produced are defined by the table 3.1.

Table 3.1. Geotechnical data for the investigation

Property	Value
Allowable bearing capacity of soil	2 bars
Anchor depth of footings	1.5m
Foundation type	Isolated footings

3.3.4. Structural data

The structural data collected for the investigation are the beams reinforcement distribution, pillar reinforcement distribution, footings reinforcement distribution and slab span.

3.3.4.1. Structure reinforcement distribution

The beam distribution of the Nsimalen industrial building is illustrated by the figure 3.10.

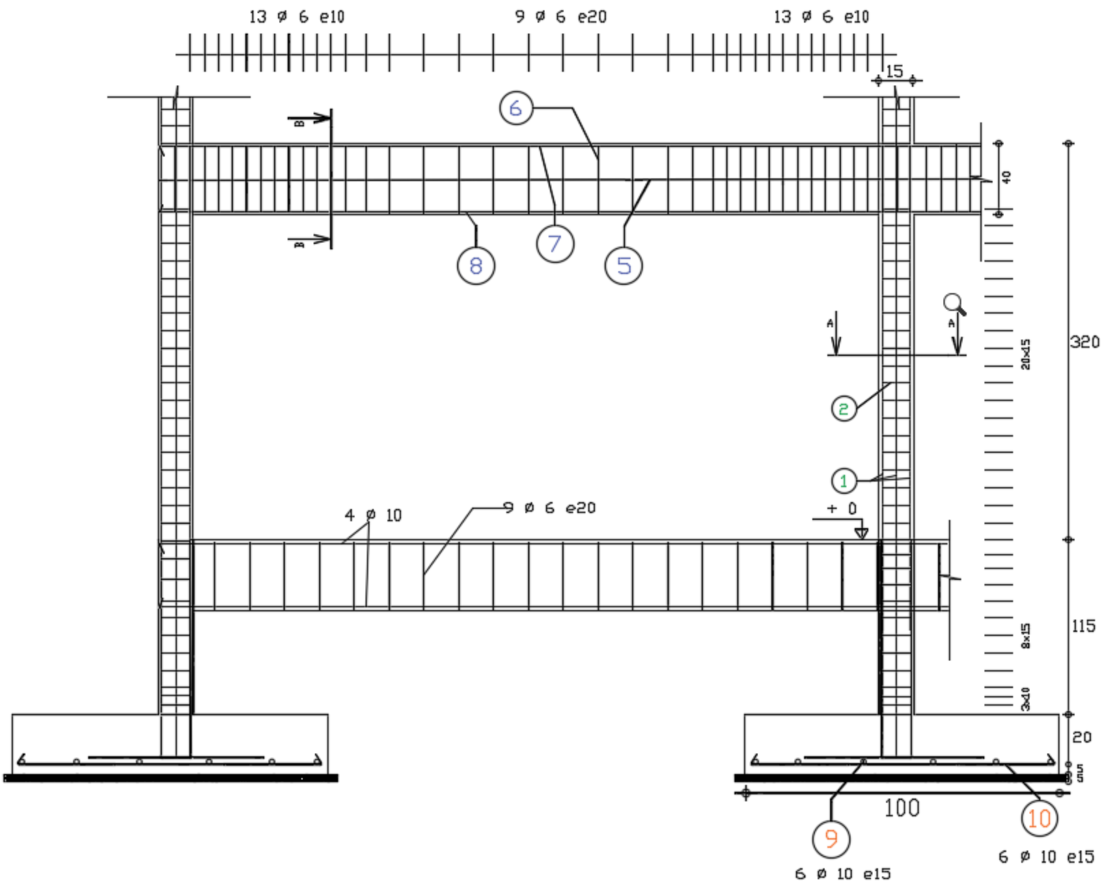
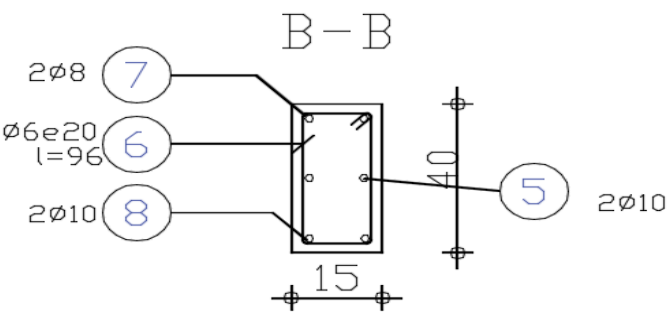


Figure 3.10. Structure reinforcement distribution.

3.3.4.2. Beam reinforcement distribution

The cut B-B is extracted from the figure 3.10 structure reinforcement distribution. The details of the beam are explained by the figure 3.11.

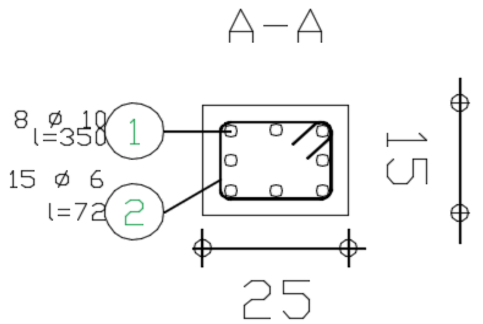


Position	Reinforcement		Shape	Steel
5	2ø10	l=560	10	HA 400
6	ø6e20	l=102	8	RL 235
7	2ø10	l=560	10	HA 400
8	2ø10	l=560	10	HA 400

Figure 3.11. Beam reinforcement detailing.

3.3.4.3. Column reinforcement distribution

The cut A-A is extracted from the figure 3.10, structure reinforcement distribution. The details of the column are explained by the figure 3.12.



Position	Reinforcement		shape	steel
1	8 ø 10	l=350	350	HA 400
2	15 ø 6	l=72	9 8 19	RL 235

Figure 3.12. Column reinforcement detailing.

3.3.4.4. Footing's reinforcement distribution

The footing reinforcement distribution is illustrated by the figure 3.13.

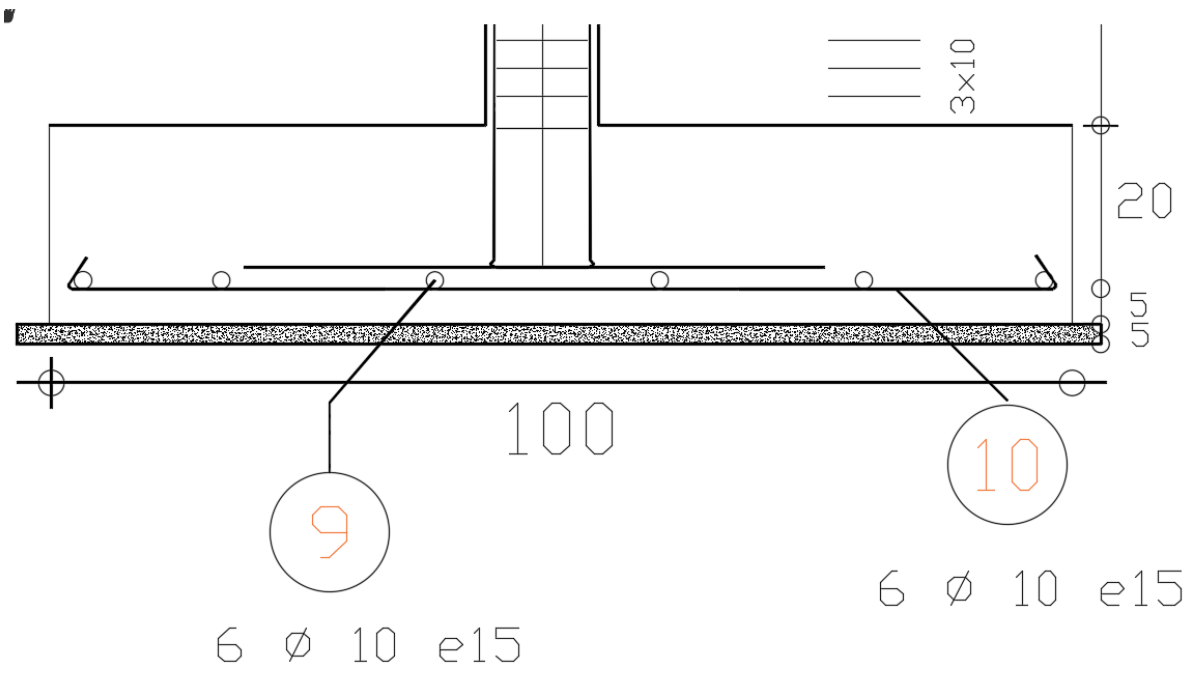


Figure 3.13. Footing's reinforcement detailing.

3.3.4.5. Slab reinforcement

The slab reinforcement is illustrated by figure 3.14

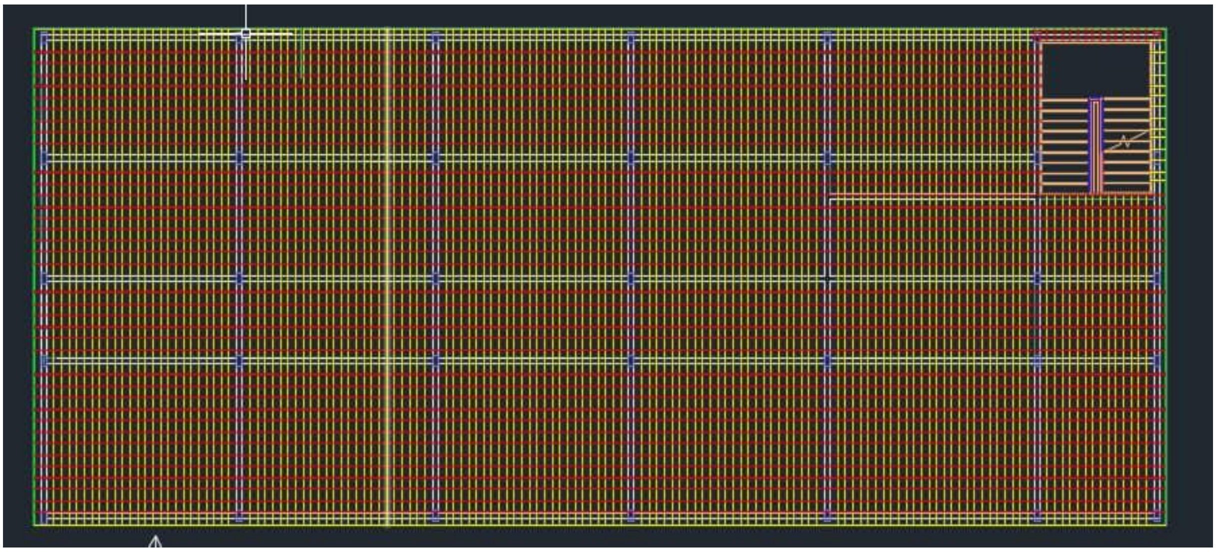


Figure 3.14. Slab reinforcement.

3.4. Numerical simulation of the Nsimalen R+1 building

After physical modelling using CSI Sap 2000 software the following numerical model was obtained representing the simulation of the Nsimalen industrial building after remodelling in order to carry out the dynamic analysis followed by static analysis and investigate if the existing building can support the newly added floor and activities. Figure 3.15 illustrates the 3d finite element model obtained.

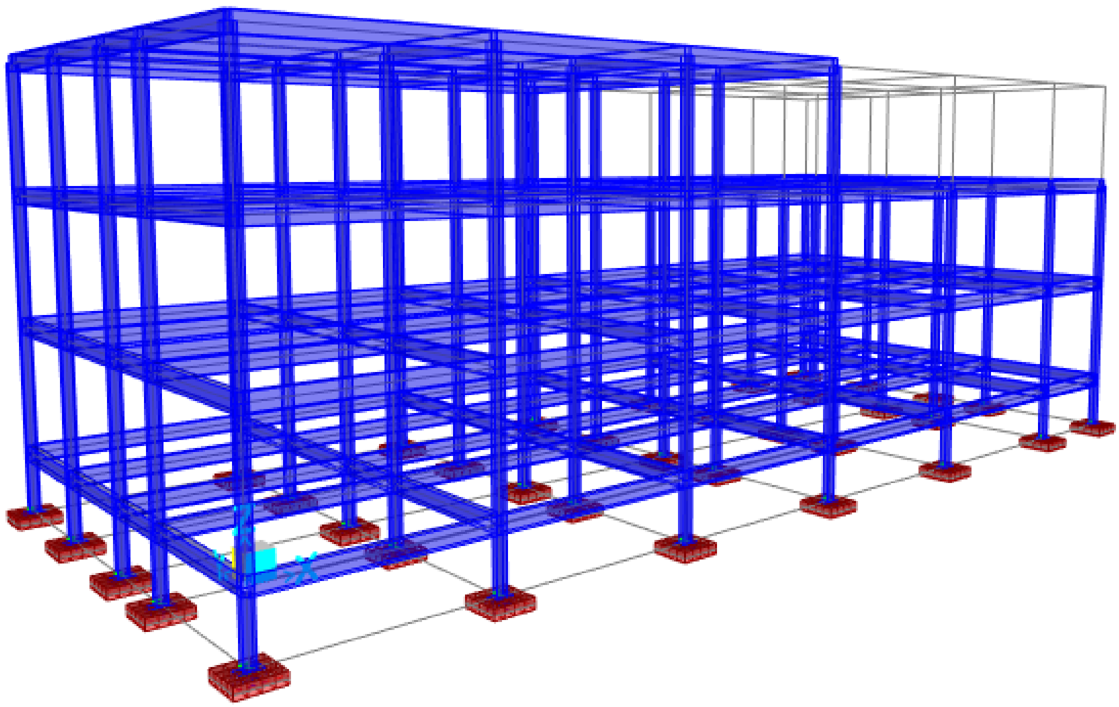


Figure 3.15. Finite element model used for analysis

3.5. Lowering of loads

After defining the 10 load combinations and an envelope combination at ultimate limit state and serviceability limit state in the software to verify the resistance of load bearing elements in terms of the newly added floor giving maximum loads for the static analysis verifications. The stresses on the foundations were also obtained for the verification of the footings. Figure 3.16 illustrates the 3d finite element model deformed shape after loads lowering due to the envelop combination at ultimate limit state.

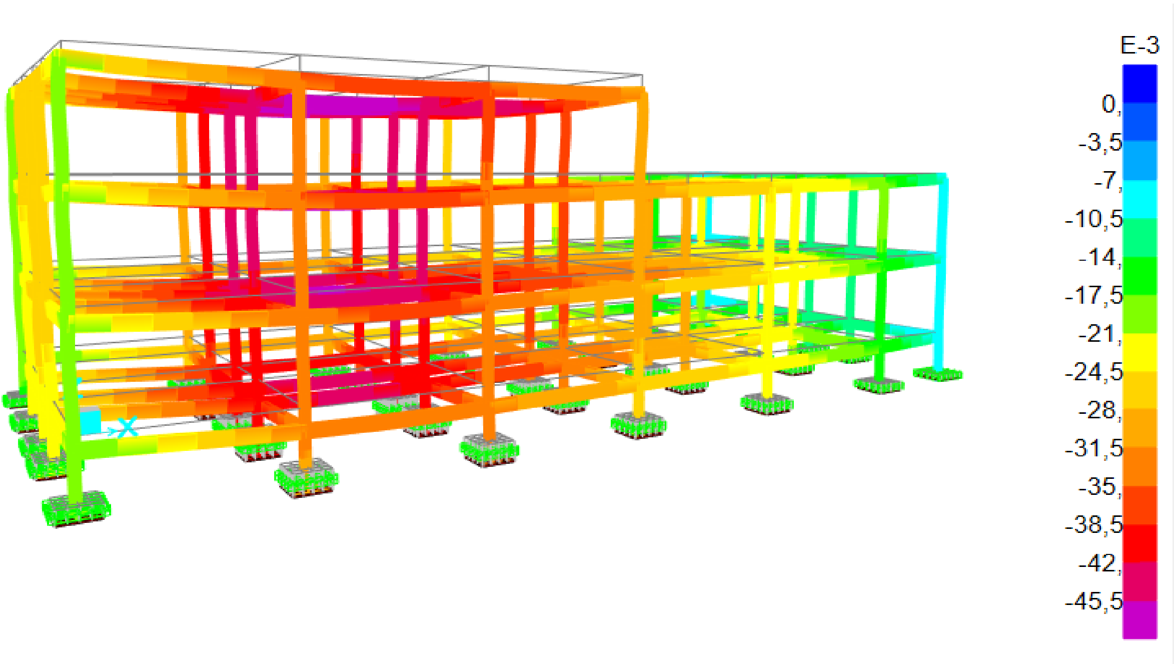


Figure 3.16. Finite element model after load lowering under the envelop combination at ultimate limit state

The colour variation indicates the variation of contour object indicating the maximum and minimum displacements of the meshed elements in the z-direction.

The beam selected for the investigation receiving the greatest loads characterised by the greatest bending moment under the envelop combination at ULS is highlighted by the rectangle as illustrated by the figure 3.17.

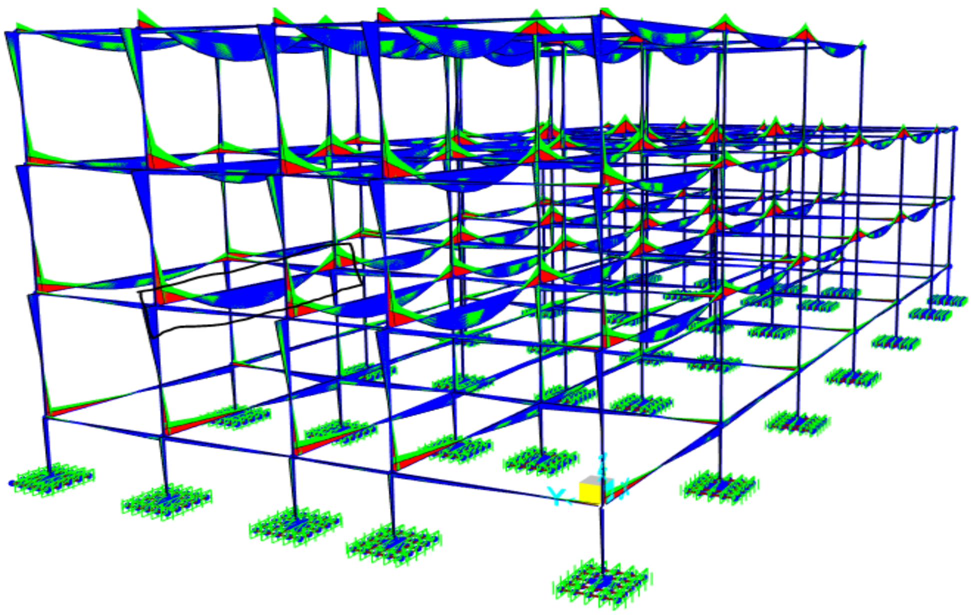


Figure 3.17. Maximum moment beam for the investigation

The column selected for the investigation receiving the greatest loads characterised by the greatest axial force under the envelop combination at ULS is highlighted by the rectangle as illustrated by the figure 3.18.

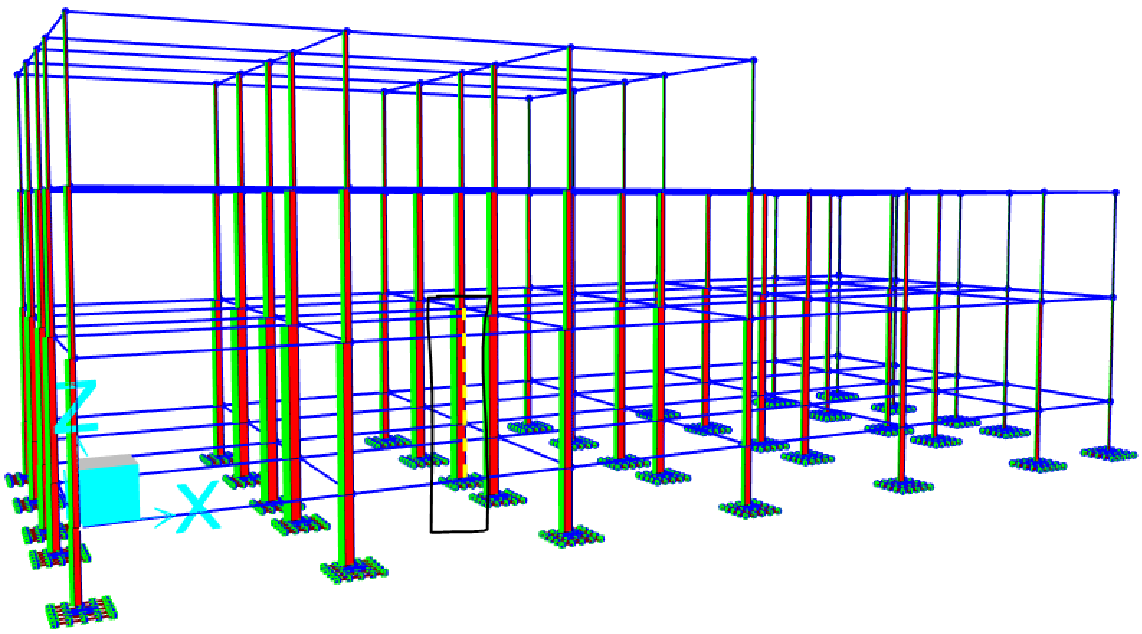


Figure 3.18. Pillar receiving maximum axial force column for the investigation

The footing receiving the maximum stresses obtained by analysis under SLS envolop load combination is illustrated by the figure 3.19.

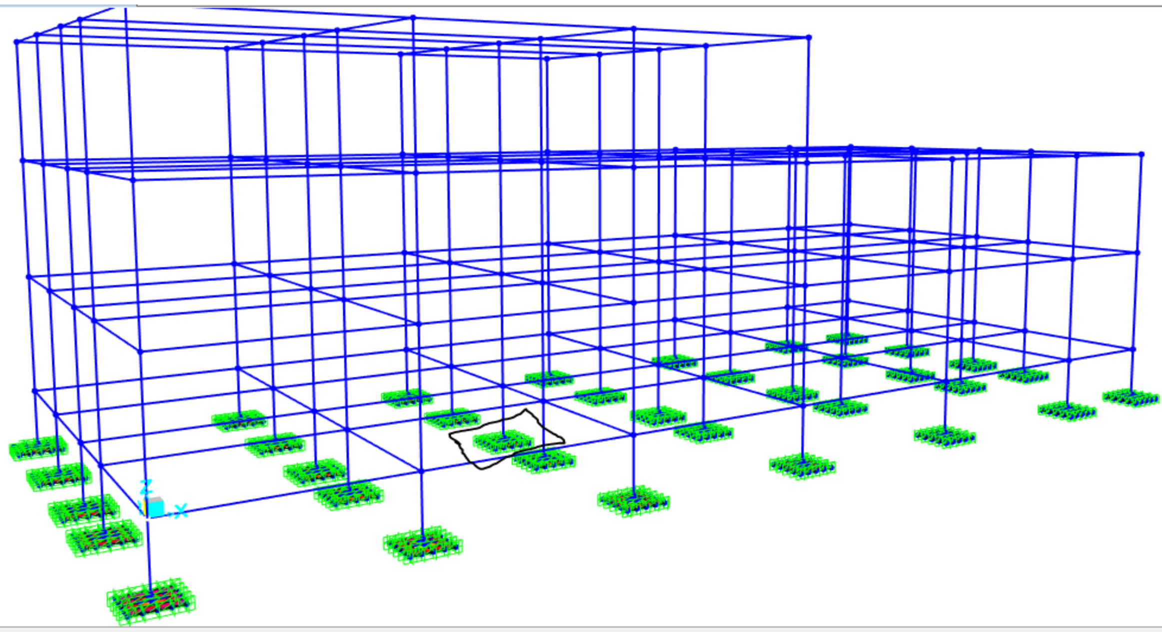


Figure 3.19. Maximum stress footing for the investigation

The figure 3.20 illustrates the distribution of stress on the lower fibre of the footing element with maximum stresses indicated in blue and minimum stress indicated in purples.

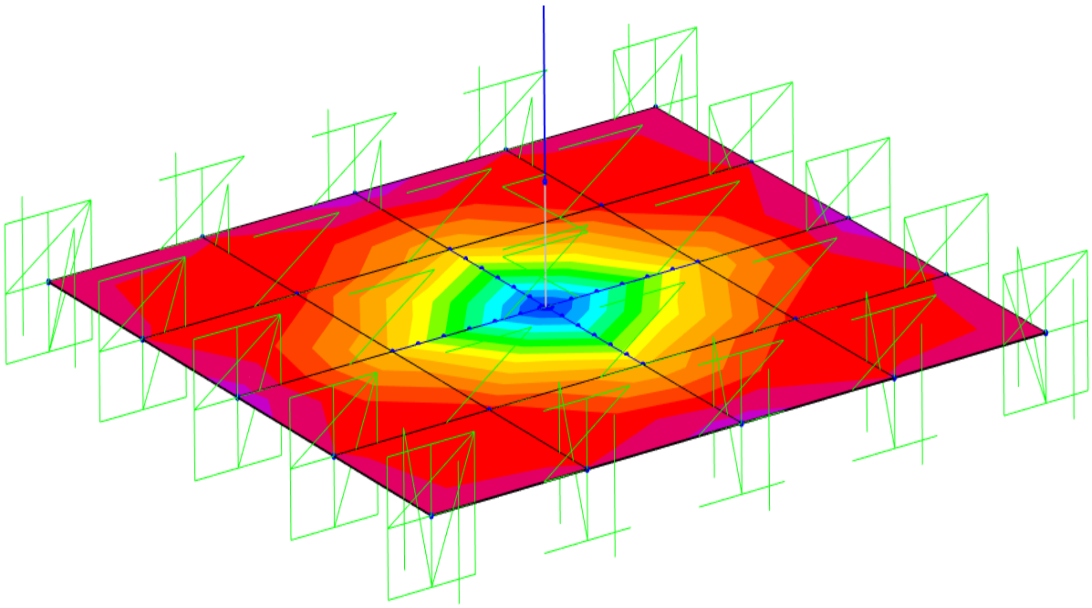


Figure 3.20. Maximum stressed footing for the investigation

3.6. Structural analysis results

The structural analysis results resulting from the dynamic and static analysis are the vibration modes and verifications of load bearing elements.

3.6.1. Vibration modes

The results of the modal analysis are given mode shape, frequency and period. It can be noticed from these results that the numerical model was well represented because of the low value of the first period of vibration and the convergence of the following periods and frequencies. Figures 3.21, 3.22 and 3.23 respectively illustrate the 3 vibration modes selected for a mass participating ratio greater than 90%. The figure 3.21 illustrates the first vibration mode which is translational with a period of 1.68s

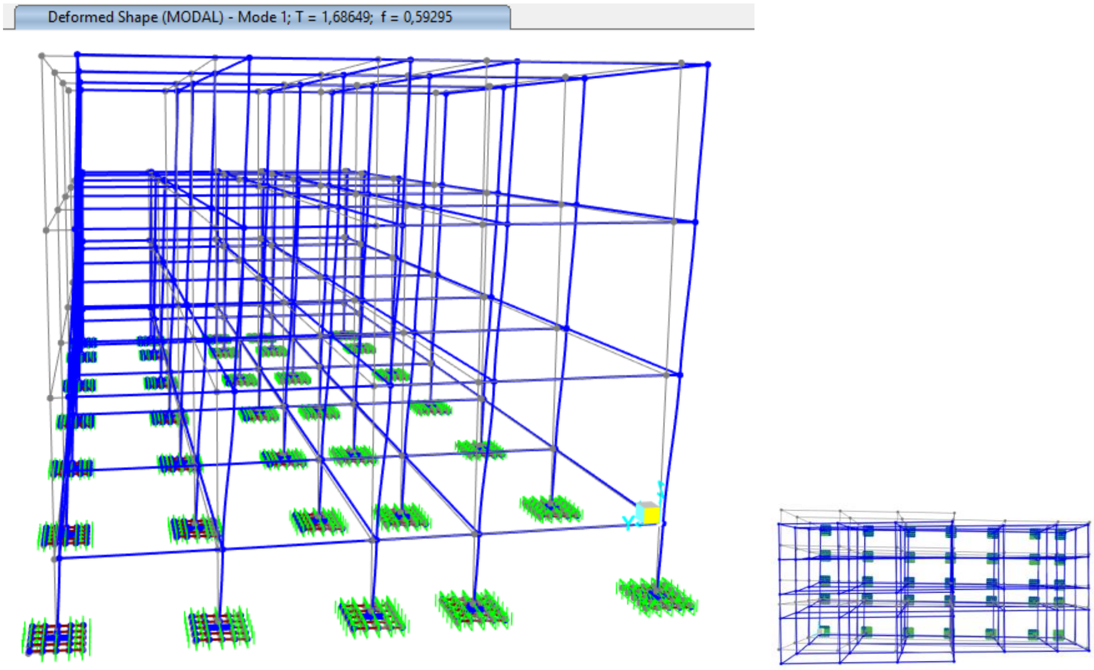


Figure 3.21. First vibration mode (Translational)

The figure 3.22 illustrates the second vibration mode which is torsional with a period of 1.17s.

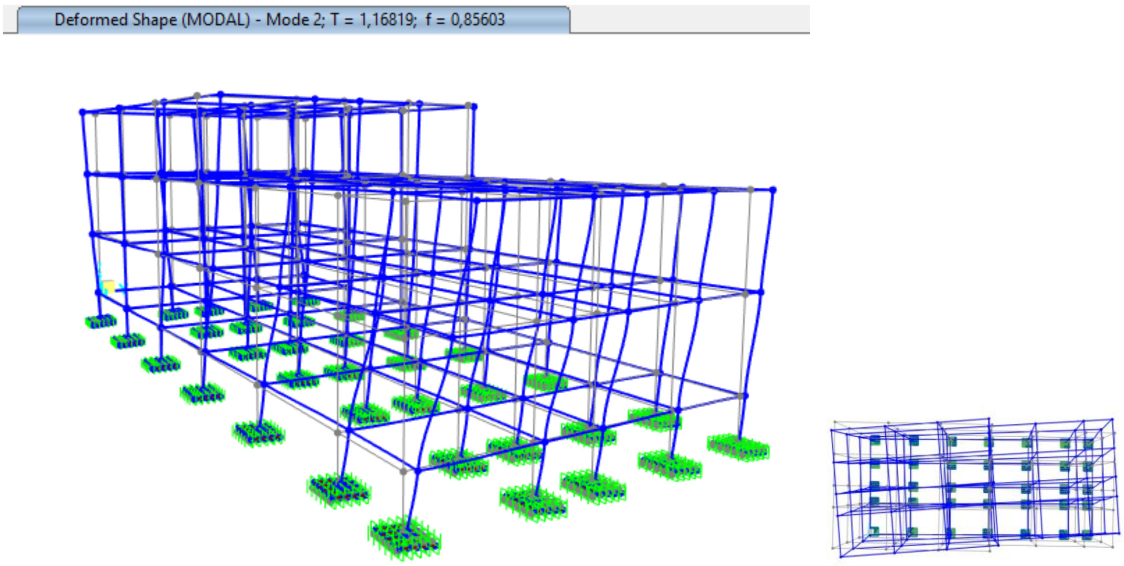


Figure 3.22. Second vibration mode (Torsional)

The figure 3.23 illustrates the third vibration mode which is torsional with a period of 1.132s.

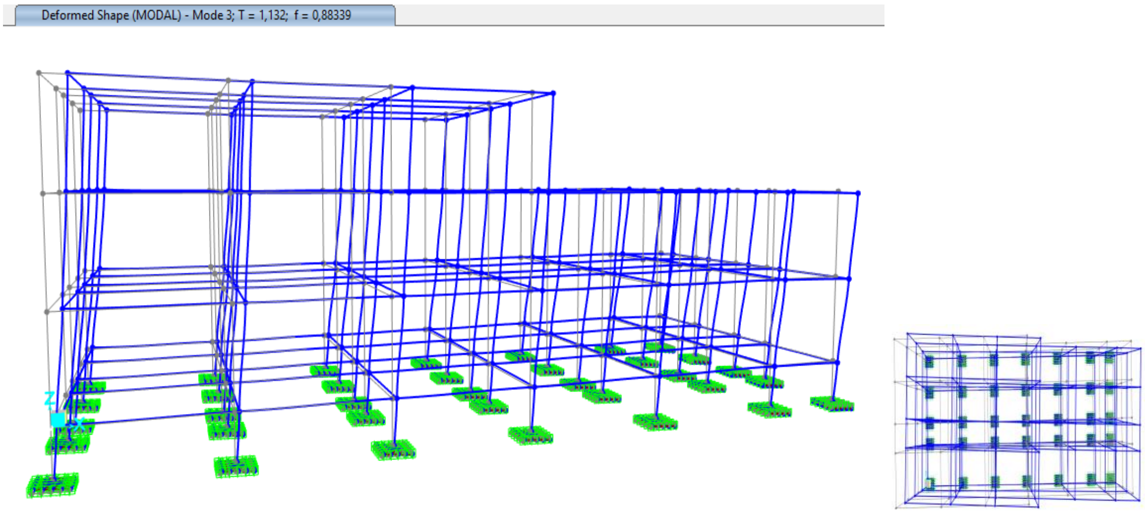


Figure 3.23. Third vibration mode (Torsional)

It can be noticed from these results that the torsional mode is dominant due to the unsymmetrical nature of the structure after remodelling which increases the eccentricity between the centre of rigidity and the centre of mass.

3.6.2. Verification of the load-bearing elements

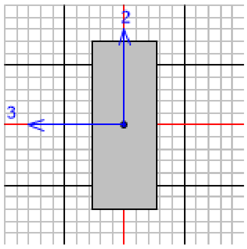
The verifications were done for load bearing members in order to assess the structure's possibility to receive the new floor distribution and activities. The load bearing members assessed are the maximum loaded beams, pillars and footings.

3.6.2.1. Beams

The verifications done on the maximum loaded beam element at ultimate limit state are reinforcement, moment and shear verifications respectively.

a. Reinforcement verification

The reinforcement verification is done by calculating the minimum reinforcement and the maximum reinforcement and identify if the reinforcement of the verified building beam element falls under the range specified by the code. Figure 3.24 illustrates the beam calculation sheet.



Units KN, m, C

```
Eurocode 2-2004 BEAM SECTION DESIGN  Type: DC HIGH MRF  Units: KN, m, C  (Summary)

L=5,31
Element      : 92             D=0,4          B=0,15         bf=0,15
Section ID   : B15*40         ds=0,          dct=0,03       dcb=0,03
Combo ID     : Enveloppe_ULS  E=31000000,    fck=25000,     Lt.Wt. Fac.=1,
Station Loc  : 0,             fyk=400000,    fywk=235000,

Gamma(Concrete):  1,5
Gamma(Steel)    :  1,15


Design Moments, M3
                       Positive    Negative
                         Moment      Moment
                         52,465     -104,93

Flexural Reinforcement for Moment, M3
                       Required     +Moment     -Moment     Minimum
                          Rebar       Rebar       Rebar       Rebar
   Top     (+2 Axis)  9,890E-04          0,   9,890E-04   1,779E-04
   Bottom  (-2 Axis)  4,449E-04   4,449E-04   3,773E-05   1,779E-04

Shear Reinforcement for Shear, V2
                          Rebar       Shear       Shear       Shear  Tan(Theta)
                          Asw/s         VEd        VRdc        VRds       Ratio
                          0,002     129,839      40,968     129,839          1,

Torsion Reinforcement for Torsion, T
                    Rebar       Rebar     Torsion    Critical        Area   Perimeter
                     At/s         Asl         TEd           T          Ak          uk
                1,284E-04   3,177E-05       0,786          0,       0,031        0,86
```

Figure 3.24. Beam calculation sheet

Minimum reinforcement, $A_{s,\,min} = 177\ mm^2$

Maximum reinforcement, $A_{s,\,max} = 240\ mm^2$

Longitudinal bottom reinforcement, $A_s = 444\ mm^2 > A_{s,\,max} = 240\ mm^2$

The result of this verification shows that the beam section is not applicable (N/A). To apply this reinforcement the beam section needs to be increased.

b. Moment verification

The moment verification is done by calculating the resisting moment and comparing the resisting moment to the design moment. The figure 3.25 illustrates the design moment obtained from the maximum loaded beam.

Design Moments, M3	Positive Moment	Negative Moment
	52,465	-104,93

Figure 3.25. Beam maximum design moments

Maximum positive design moment, $M^{+}_{ed} = 52.47$ kNm

Maximum negative design moment, $M^{-}_{ed} = -104.93$ kNm

Resisting moment, $M_{Rd} = 53.43$ kNm > $M_{ed} = 52.47$kNm

This result confirms the design of the section by the software, but because of a steel section greater than the maximum specified by the code the beam section is not applicable.

c. Shear verification

The shear verification is done by comparing the acting shear V_{Ed} to the allowable shear of the member without shear reinforcement $V_{Rd,}$. If no design shear reinforcement is required, the minimum shear reinforcement is applied according to the detailing of that member. The calculation sheet obtained from the software is illustrated by the figure 3.26.

Shear Reinforcement for Shear, V2	Rebar Asw/s	Shear VEd	Shear VRdc	Shear VRds	Tan(Theta) Ratio
	0,002	129,839	40,968	129,839	1,

Figure 3.26. Shear verification on maximum loaded beam

The results obtained from the calculation sheet illustrate that the acting shear V_{Ed} is greater than the design shear resistance of the member without shear reinforcement
$V_{Rd,C}$ ($V_{Ed} > V_{Rd,C}$), so a shear reinforcement is required.

Shear reinforcement calculated by the software, $\frac{A_{sw}}{s} = 0.002$.

Maximum shear reinforcement, $\frac{A_{sw,max}}{s} = 0.0027$

This result confirms the design of the shear reinforcement by the software.

3.6.2.2. Columns

The verifications done on the maximum loaded column element at ultimate limit state are reinforcement verification, moment-axial force verification, shear and slenderness verification.

a. Reinforcement verification

The existing columns of the building are labeled columns C1 to C35 as shown on figure 3.27.

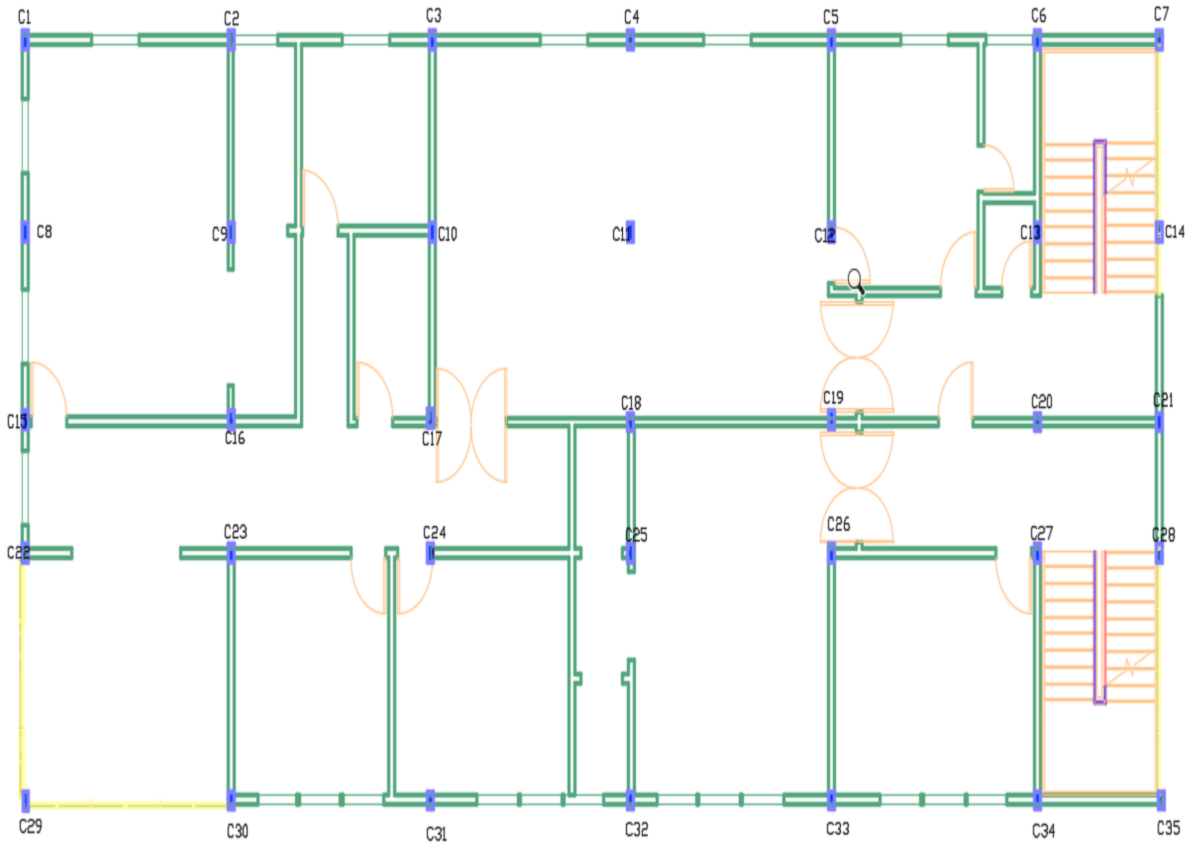


Figure 3.27. Column distribution

The reinforcement verification is done by calculating the minimum reinforcement and the maximum reinforcement and identifying if the reinforcement of the verified building beam element falls under the range specified by the code. Figure 3.28 illustrates the reinforcement of Column C17 obtained by the software.

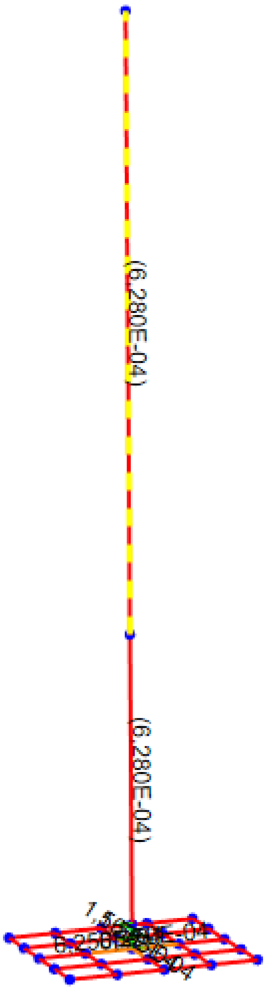


Figure 3.28. Column's reinforcement

$A_{s,min}$ and $A_{s,max}$ were calculated using the equations 2.14 and 2.15 respectively and the results obtained are presented on table 3.2

Table 3.2. Reinforcement verification for the pillars

Reinforcement Verification				
Columns	Reinforcement [mm2]	Minimum reinforcement [mm2]	Maximum reinforcement [mm2]	Code
C1-C35	628	204	1500	Eurocode 3-2005

Minimum reinforcement, $A_{s,\ min} = 204\ mm^2$

Maximum reinforcement, $A_{s,\ max} = 1500\ mm^2$

Column reinforcement, $A_s = 628\ mm^2$

$A_{s,\ min} = 204\ mm^2 < As < A_{s,\ max} = 1500\ mm^2$

The result of this verification shows that the reinforcement in the section satisfies the specifications.

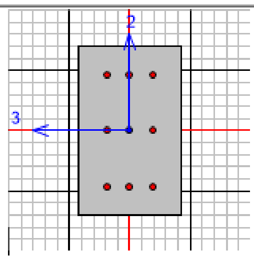
b. Moment-axial force verification

The design solicitations for the columns C1 to C35 is illustrated by table 3.3 with the maximum loaded pillar (C17) highlighted

Table 3.3. Columns design solicitations

Column	Combo	Ned	Med	Status
		KN	KN-m	
C1	Enveloppe_ULS	-280,96	8,6	
C2	Enveloppe_ULS	-479,69	11,49	
C3	Enveloppe_ULS	-487,73	9,76	
C4	Enveloppe_ULS	-412,38	-8,24	
C5	Enveloppe_ULS	-340,45	-6,81	
C6	Enveloppe_ULS	-242,95	-8,94	
C7	Enveloppe_ULS	-73,199	-9,658	
C8	Enveloppe_ULS	-391,32	16,09	
C9	Enveloppe_ULS	-655,75	17,96	Overstressed
C10	Enveloppe_ULS	-667,76	13,36	
C11	Enveloppe_ULS	-564,89	11,28	
C12	Enveloppe_ULS	-479,18	-11,02	
C13	Enveloppe_ULS	-346,61	-14,43	
C14	Enveloppe_ULS	-184,52	14,87	
C15	Enveloppe_ULS	-413,86	-17,27	
C16	Enveloppe_ULS	-697,57	19,20	Overstressed
C17	Enveloppe_ULS	-712,06	14,24	Overstressed
C18	Enveloppe_ULS	-602,08	12,04	
C19	Enveloppe_ULS	-507,12	-12,05	
C20	Enveloppe_ULS	-365,17	-15,27	
C21	Enveloppe_ULS	-196,72	-15,56	
C22	Enveloppe_ULS	-406,6	16,86	
C23	Enveloppe_ULS	-685,69	18,81	Overstressed
C24	Enveloppe_ULS	-699,74	14,00	Overstressed
C25	Enveloppe_ULS	-591,94	11,84	
C26	Enveloppe_ULS	-499,17	-11,73	
C27	Enveloppe_ULS	-360,25	-14,89	
C28	Enveloppe_ULS	-193,47	-15,30	
C29	Enveloppe_ULS	-327,60	14,29	
C30	Enveloppe_ULS	-546,62	15,18	
C31	Enveloppe_ULS	-559,29	11,19	
C32	Enveloppe_ULS	-473,57	9,47	
C33	Enveloppe_ULS	-392,46	-9,24	
C34	Enveloppe_ULS	-281,32	-11,22	
C35	Enveloppe_ULS	-151,87	-11,03	

After obtaining the bending moment and the axial force solicitations from the ultimate limit state envelop combination, the design is done through the M-N interaction diagram. For the verification to be validated the design moment, M_{ed} and the design axial force N_{ed} should belong to the domain of the M-N interaction diagram for the column section considered. The most loaded column C17 (figure 3.27) design sheet is illustrated by the figure 3.29.



Units KN, m, C

```
Eurocode 2-2004 COLUMN SECTION DESIGN  Type: DC HIGH MRF  Units: KN, m, C  (Summary)

L=1,5
Element      : 55                  B=0,15         D=0,25          dc=0,041
Section ID   : C15*25              E=31000000,    fck,cyl=25000,  Lt.Wt. Fac.=1,
Combo ID     : Enveloppe_ULS       fyk=400000,    fywk=235000,
Station Loc  : 0,                  RLLF=1,        SOM:  Nominal Curvature
Combo Eq.    :  Eq. 6.10

Gamma(Concrete):  1,5               AlphaCC = 1,    AlphaCT = 1,
Gamma(Steel)   :  1,15              AlphaLCC = 0,8  AlphaLCT = 0,8
Gamma(cE)      :  1,2               Alpha_e = 15,   phi_ef = 1,21

SLENDERNESS CHECK (governing permutation)
                     Slenderness Slenderness      Column   Governing
                           Ratio Limit Ratio   Condition Permutation
   Major Bending(M3)      20,785      13,765         N/A         N/A
   Minor Bending(M2)      34,641      13,765         N/A         N/A

AXIAL FORCE & BIAXIAL MOMENT CHECK FOR NEd, MEd2, MEd3 (governing permutation)
              Capacity      Design      Design      Design     Minimum     Minimum
                 Ratio         NEd        MEd3        MEd2          M3          M2
                 1,342     712,059      14,241     -14,241      14,241      14,241

FIRST-ORDER MOMENTS (governing permutation)
                           End I       End J     Section        Mimp         M02         M01
                          Moment      Moment      Moment      Moment      Moment      Moment
   Major Bending(M3)      -0,129       2,713      -0,129        2,67       5,383       2,541
   Minor Bending(M2)      -0,156       0,547      -0,156        2,67       0,547      -0,156
```

Figure 3.29. Column calculation sheet

The design moment value, M_{ed} = 14.24 kNm

The design axial force value, N_{ed} = 712.06 kNm

The moment-axial force verification for is shown on the figure 3.29

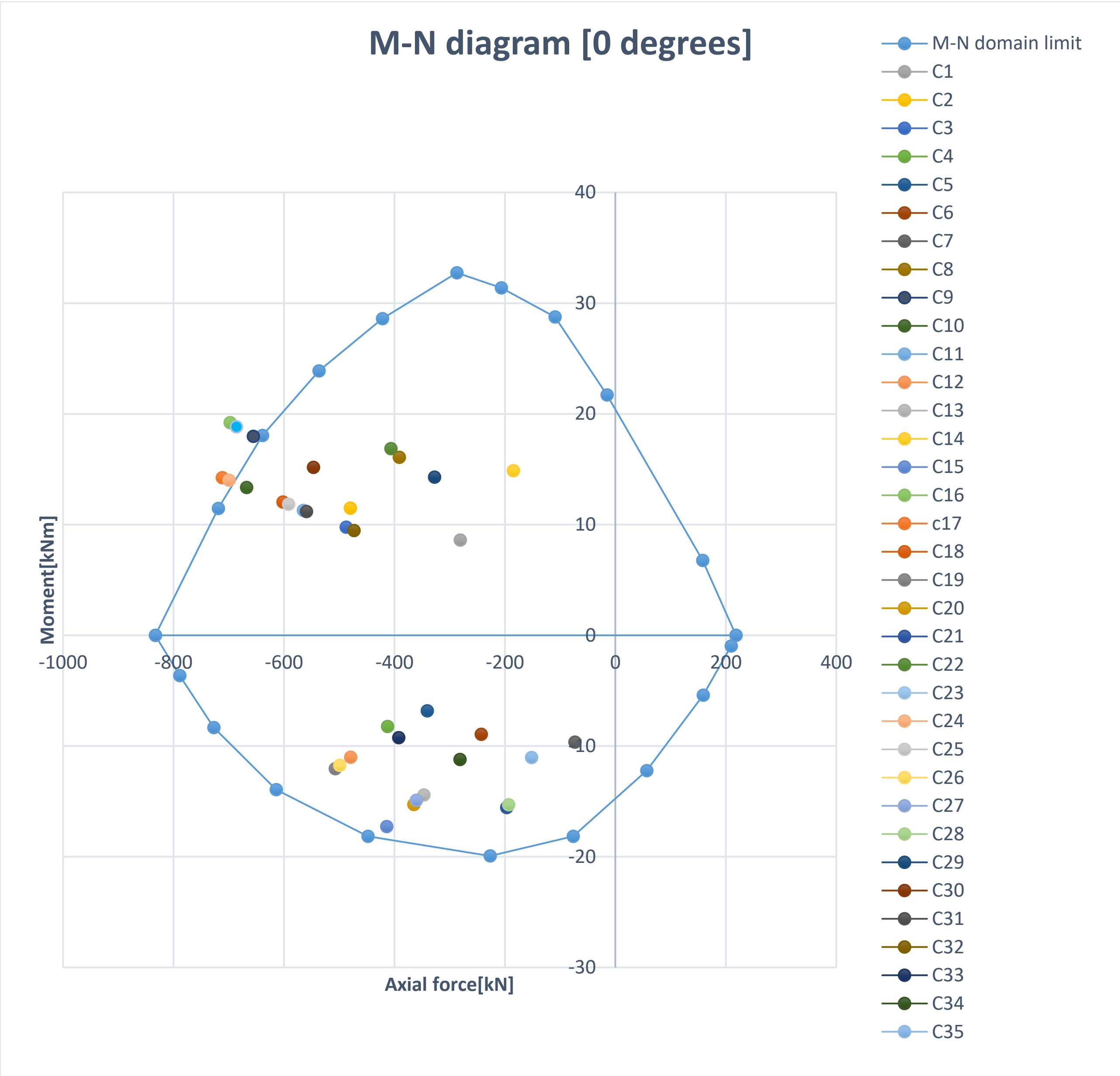


Figure 3.30. M-N interaction diagram for columns verification

What is noticed from the figure 3.30 is that 5 of the design solicitations do not belong to the M-N interaction domain. This result highlights the fact that the column C9, C16, C17, C23 and C24 are not applicable for the buildings remodelling and new functional usage and apart from these columns, some of the neighbouring columns were found to be overstressed.

c. Shear verification

The necessity of the shear reinforcement is verified by comparing the acting shear V_{Ed} to the allowable shear of the member without shear reinforcement $V_{Rd,C}$. If the allowable shear

$V_{Rd,C}$ exceeds the design shear V_{Ed} then no shear reinforcement is required hence the minimum shear reinforcement can be applied which consist of 6mm bar diameter as prescribed by Eurocode 2. Figure 3.31 illustrates the shear calculation sheet by the software.

SHEAR DESIGN FOR V2,V3

	Rebar Asw/s	Shear VEd	Shear VRdc	Shear VRds	Tan(Theta) Ratio
Major Shear(V2)	0,	0,833	37,447	0,	0,
Minor Shear(V3)	0,	0,532	33,673	0,	0,

Figure 3.31. Shear calculation sheet

d. Slenderness verification

The slenderness verification consists in verifying if the slenderness of the element is below a limit value defined by the Eurocode 2. This verification permits the decision if secondary effect has to be considered or not. Figure 3.32 illustrates the calculation done by the software to verify slenderness.

SLENDERNESS CHECK (governing permutation)

	Slenderness Ratio	Slenderness Limit Ratio	Column Condition	Governing Permutation
Major Bending(M3)	20,785	13,765	N/A	N/A
Minor Bending(M2)	34,641	13,765	N/A	N/A

Figure 3.32. Slenderness verification sheet

For the major bending and minor bending the slenderness ratio is greater than the slenderness limit ratio. The conclusion made from his verification is that the column is susceptible to buckling failure hence not applicable for the structural remodelling and new functional use of the building.

3.6.2.3. Footings

The verifications were performed for the footing element under column C17 considering the serviceability states envelop load combination. The verifications performed are the allowable stress verification and reinforcement verification.

a. Allowable soil stress verification

The verification is passed when the maximum work stress (σ_s) is less than the allowable bearing capacity of the soil (σ_{adm}). The figure 3.33 illustrates the soil pressure produced by building loads on the soil.

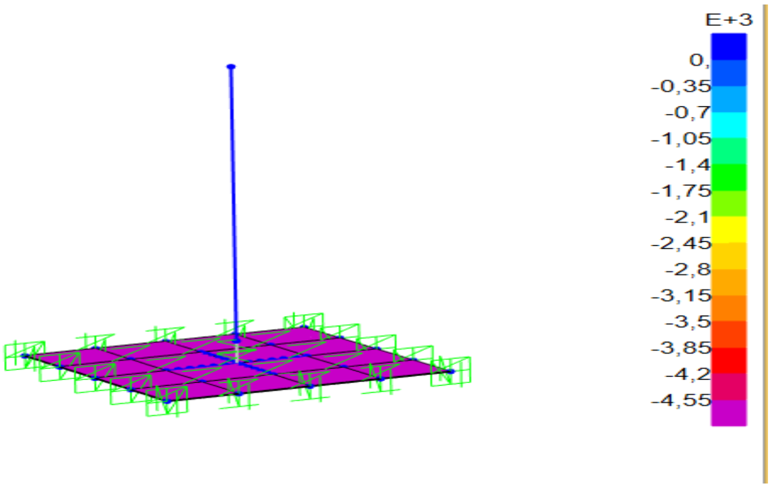


Figure 3.33. Soil pressure produced by building loads on the soil.

The blue indicates the values on the footings where the stresses are minimum hence corresponds to the minimum value while the purple corresponds to where the stresses are maximum which gives the maximum work stress of the building on the foundation.

Maximum work stress, $\sigma_s = 4550$ kN/m^2.

Allowable bearing capacity of the soil, $\sigma_{adm} = 2$bars ≡ 200 kN/m^2.

$\sigma_{adm} = 200$ kN/m^2 $<$ $\sigma_s = 4550$ kN/m^2.

The result of the calculation shows that the maximum work stress is far greater than the allowable bearing capacity of the soil hence the footing section is not applicable (N/A) and this result highlights the fact that the existing foundation cannot support the actions of the remodelled building.

b. Reinforcement verification

The range of reinforcement obtained from the software in the y-direction $A_{s,y}$ is illustrated by the figure 3.34.

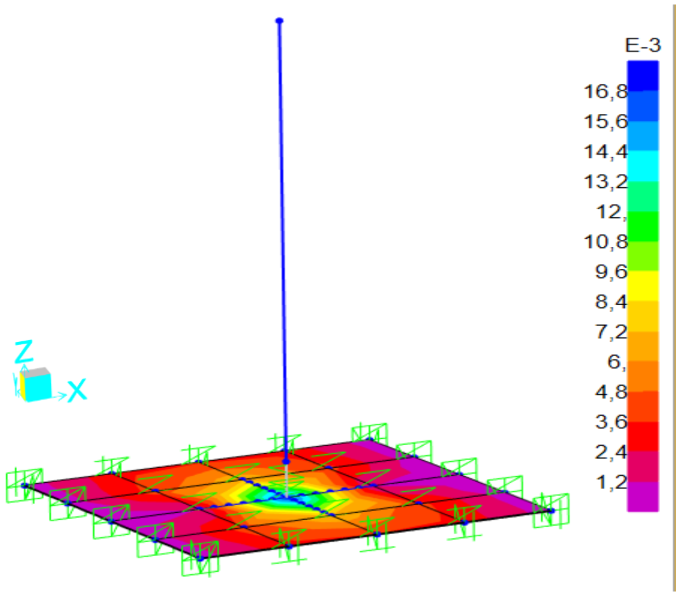


Figure 3.34. Footing reinforcement in the y-direction.

The maximum value of reinforcement in the y-direction calculated is 0,017 m^2/m indicated in blue by the contour object while the minimum reinforcement value is 0.0012 m^2/m indicated in purple by the contour object.

The plan annotated diagram used for the footing verification is indicated by figure 3.35.

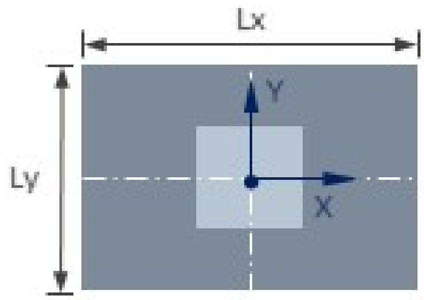


Figure 3.35. Footing annotated plan view

Here:

Lx = Ly = 1.00m

The maximum reinforcement prescribed by the code, $A_{s,\,max}$ = 0.004 Ac.

Footing width, Lx = 1.00m

Footing height, h = 0.25m

Area of concrete section, Ac = 0.25 m^2

$A_{s,\,max}$ = 0.001 m^2

The maximum reinforcement prescribed by the code for a section 0.25 m by 1.00 m is 0.001 m^2 which is less than the minimum reinforcement required by the footing in the y-direction calculated by the software this result highlights the fact that the reinforcement is not applicable to a footing section of length, 1m and width 1m

The reinforcement range obtained from the software in the x-direction $A_{s,\,x}$ is illustrated by the figure 3.36.

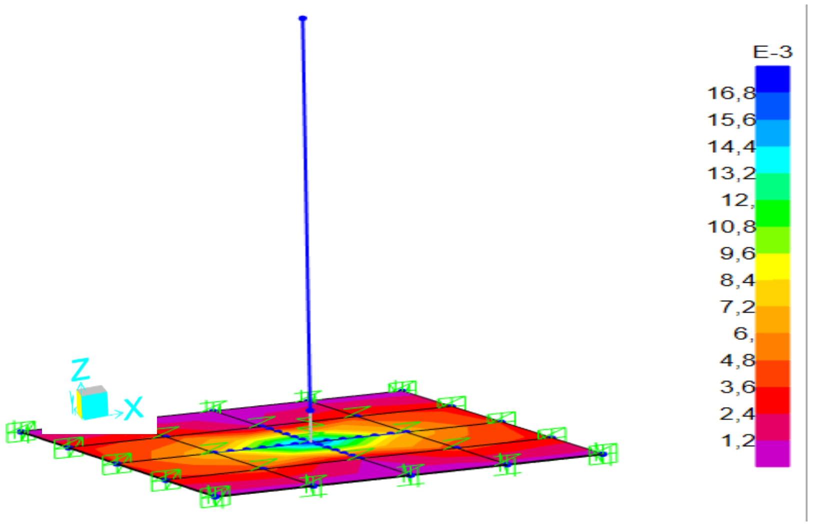


Figure 3.36. Footing reinforcement in the x-direction.

The maximum value of reinforcement in the x-direction calculated is 0.017 m^2/m indicated in blue by while the minimum reinforcement value is 0.0012 m^2/m.

The maximum reinforcement prescribed by the code, $A_{s,\,max} = 0.004$ Ac.

Footing width, Ly = 1.00m

Footing height, h = 0.25m

Ac = 0.25 m^2

$A_{s,\,max} = 0.001\ m^2$

The maximum reinforcement prescribed by the code for a section 0.25 m by 1.00 m is 0.001 m^2 which is less than the minimum reinforcement required by the footing in the x-direction calculated by the software this result highlights the fact that the reinforcement is not applicable to a footing section of length, 1m and width 1m.

The results obtained shows that the foundation of the building is not applicable for the remodelling and new functional use.

3.7. Failures evidences

After structural analysis for the verification of load bearing elements, key results were obtained for the analysis of failure during the remodelling of the Nsimalen building. These results permit the characterisation of failure and safety of the building towards the new remodelling project. The failures evidences were highlighted following safety factors, deformed shape of the structure, failure analysis and structural detailing.

3.7.1. Safety factors

The safety factors resulting from the static design of load bearing elements were calculated following the reinforcement done on site for the elements as defined by equation 3.21.

$$S.F = \text{Actual verification value} / \text{theoretical verification value} \qquad (3.21)$$

Where:

$S.F$ is the safety factor

This is calculation is done for load bearing elements designed by the software such as beams.

The safety factors resulting from the static verification of load bearing elements were calculated following design and resistance parameters evaluated by software defined by equation 3.22.

$$S.F = \text{Resistance verification value} / \text{Design verification value} \qquad (3.22)$$

Where:

$S.F$ is the safety factor

This is calculation is done for load bearing elements verified by the software such as columns and footings. Table 3.2 illustrates the safety factor values obtained from verifications.

Table 3.4. Safety factors for the design and verification of load bearing elements

Element	Verification	Actual	units	Theoritical	Safety factors	Valid for remodelling	Failure
Beam (N/A)	Reinforcement verification	416,00	mm2	1433,00	**0,29**	✗	Cracks
	Shear verification	932,58	mm2	2000,00	**0,47**	✗	Shear failure
Element	Verification	Design	units	Resistance	**Safety factors**	**Valid for remodeling**	**Failure evidence**
Column (N/A)	Reinforcement verification	628,00	mm2	628,00	**1,00**	✓	-
	Moment-axial force	14,24	kNm	6,75	**0,47**	✗	Rebar buckling
		712,00	kN	157,86	**0,22**	✗	Concrete crushing
	Shear verification	0,83	kN	37,45	**44,96**	✓	-
	Slenderness verification	20,79		13,77	**0,66**	✗	Column buckling
Footing (N/A)	Allowable stress verification	4550	kpa	200	**0,04**	✗	Differential settlement
	Verification	Actual	units	Theoritical	**Safety factors**	**Valid for remodelling**	
Verified	Reinforcement verification	420	mm2/m	17000	**0,02**	✗	Cracks

In red colour are the critical values showing that the sections are not applicable for the newly added floor and the new functional usage. The failures forecast if the remodelling was done for the beam element are cracking due to the fact the section is small and cannot resist to the internal stresses without an increasing its section and bringing an additional reinforcement. Shear failure occurs in the beam because the necessary steel section to resist shear is not provided by the beam.

For the column the failure forecast is column buckling due to the high value of the axial load that causes rebar buckling and concrete crushing.

For the footings elements the failure obtained is differential settlement due to the fact that the allowable bearing capacity of the soil is not verified hence due to high load values coming from the superstructure there will be a form of shifting of the soil beneath the foundation due to the weak bearing capacity offered by the soil supporting the remodelled building.

3.7.2. Deformed shape

The deformed shape resulting from ultimate limit state load combination was obtained for load bearing elements resultant displacement values as illustrated by figure 3.37.

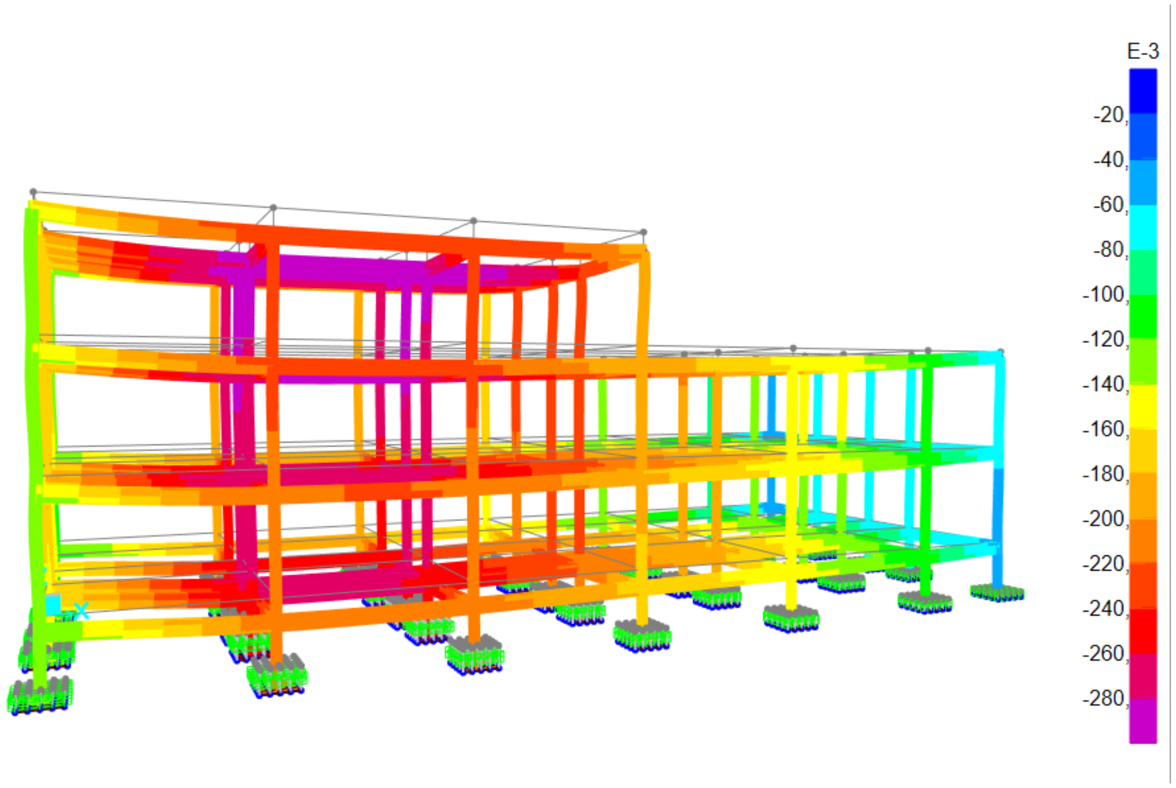


Figure 3.37. Deformed shape resulting from the displacement of elements

The resultant maximum displacement value is evaluated at 49.0mm which is above the allowable value of L (span length)/250 indicating that the remodelling will not be favourable for the building.

The deformed shape for the buildings actions for soil pressure (soil structure interaction) was obtained in order to highlight the failure that can lead to the collapse of the structure resulting from the allowable bearing capacity verification as illustrated by figure 3.38.

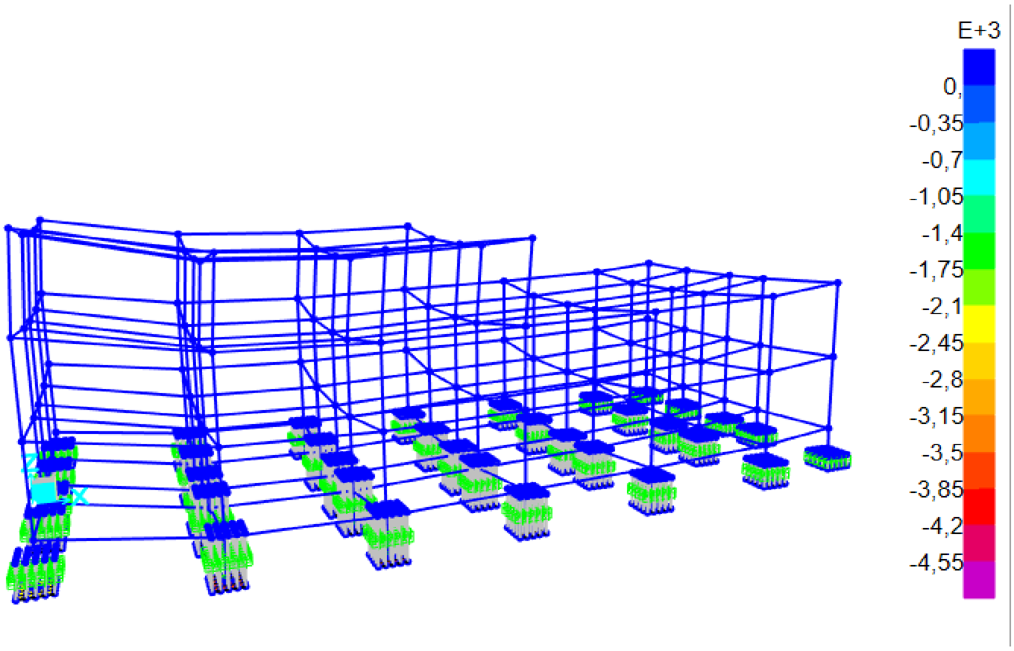


Figure 3.38. Existing foundation failure resulting from overloading in the case of remodelling

The failure is initiated by the maximum loaded pillars C17 and C16 as shown by the section illustrated by figure 3.39.

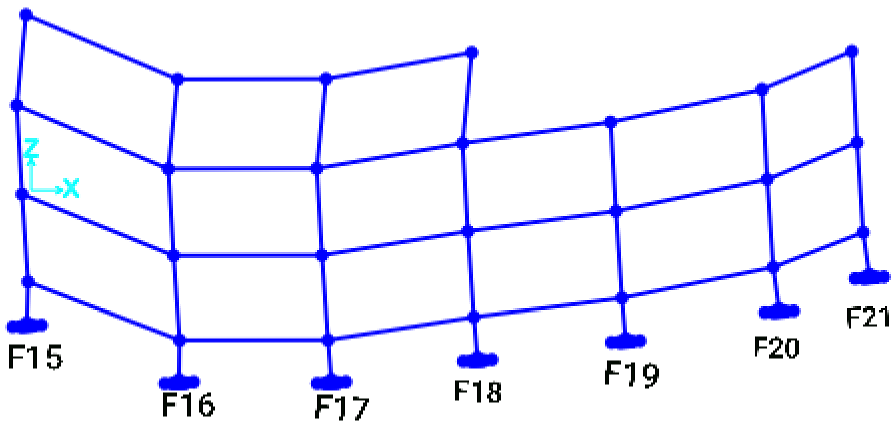


Figure 3.39. Differential settlement initiated by maximum loaded pillars

The failure described by the displacement is called differential settlement which is caused by some form of shifting of the soil beneath the foundation due to the weak bearing capacity offered by the soil supporting the remodelled building.

The table 3.5 illustrate the resultant displacements for the footings F15, F16, F17, F18, F19, 20 and F21 with U1 being the displacement in the x-direction, U2 the displacement in the y-direction, and U3 the displacement in the z-direction of each footing.

Table 3.5. Resultant footings displacements

Footing	OutputCase	U1	U2	U3
		m	m	m
F1	COMB_ELS_Enveloppe	-0,00254	-0,00080	-0,11513
F2	COMB_ELS_Enveloppe	-0,00450	0,00471	-0,17879
F3	COMB_ELS_Enveloppe	-0,00003	0,00467	-0,17844
F4	COMB_ELS_Enveloppe	0,00047	0,00392	-0,15155
F5	COMB_ELS_Enveloppe	0,00040	0,00359	-0,12612
F6	COMB_ELS_Enveloppe	0,00302	0,00256	-0,08998
F7	COMB_ELS_Enveloppe	0,00348	0,00122	-0,04943
F8	COMB_ELS_Enveloppe	-0,00648	0,00127	-0,14840
F9	**COMB_ELS_Enveloppe**	**-0,00584**	**0,00276**	**-0,24153**
F10	COMB_ELS_Enveloppe	0,00061	0,00277	-0,24059
F11	COMB_ELS_Enveloppe	0,00097	0,00225	-0,20407
F12	COMB_ELS_Enveloppe	0,00086	0,00191	-0,17440
F13	COMB_ELS_Enveloppe	0,00479	0,00122	-0,12550
F14	COMB_ELS_Enveloppe	0,00540	0,00046	-0,06795
F15	COMB_ELS_Enveloppe	-0,00705	-0,00041	-0,15651
F16	**COMB_ELS_Enveloppe**	**-0,00613**	**-0,00037**	**-0,25631**
F17	**COMB_ELS_Enveloppe**	**0,00071**	**-0,00036**	**-0,25697**
F18	COMB_ELS_Enveloppe	0,00120	-0,00038	-0,21692
F19	COMB_ELS_Enveloppe	0,00108	-0,00046	-0,18405
F20	COMB_ELS_Enveloppe	0,00511	-0,00048	-0,13171
F21	COMB_ELS_Enveloppe	0,00567	-0,00047	-0,07204
F22	COMB_ELS_Enveloppe	-0,00683	-0,00125	-0,15400
F23	**COMB_ELS_Enveloppe**	**-0,00610**	**-0,00188**	**-0,25218**
F24	**COMB_ELS_Enveloppe**	**0,00067**	**-0,00189**	**-0,25178**
F25	COMB_ELS_Enveloppe	0,00113	-0,00164	-0,21350
F26	COMB_ELS_Enveloppe	0,00102	-0,00146	-0,18137
F27	COMB_ELS_Enveloppe	0,00499	-0,00112	-0,13015
F28	COMB_ELS_Enveloppe	0,00557	-0,00079	-0,07109
F29	COMB_ELS_Enveloppe	-0,00595	-0,00240	-0,12524
F30	COMB_ELS_Enveloppe	-0,00517	-0,00365	-0,20264
F31	COMB_ELS_Enveloppe	0,00016	-0,00364	-0,20284
F32	COMB_ELS_Enveloppe	0,00068	-0,00318	-0,17225
F33	COMB_ELS_Enveloppe	0,00059	-0,00294	-0,14418
F34	COMB_ELS_Enveloppe	0,00360	-0,00231	-0,10319
F35	COMB_ELS_Enveloppe	0,00408	-0,00153	-0,05705

A histogram was plotted to better visualise the values of settlements for the footings labelled 1 to 35 and the footings on the section initiating the differential settlement highlighted as shown on figure 3.40.

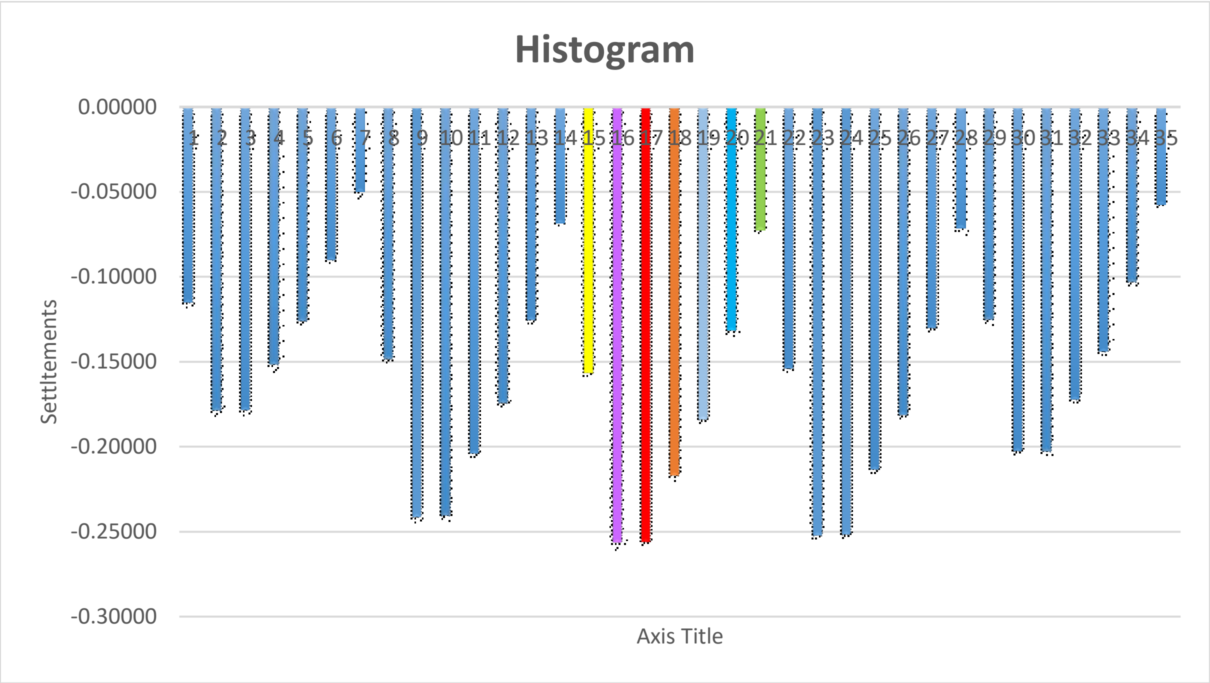


Figure 3.40. Differential settlement histogram

The settlement values are far above the admissible value of 25mm by Eurocode 7-part 3 section 5 [23] indicating that the beam, column and footing sections are not applicable for remodelling and that the existing foundation soil level does not support the loads from the structure.

Conclusion

The main objective of this chapter was to present the results obtained from applying a forensic engineering structural approach for the remodelling of the building. Firstly, the site was presented comprising of the location, relief, climate, population, hydrology, economic activities etc. Secondly, the physical description of the site was given and the general presentation of the project where the various plans, material characteristics, geotechnical data and structural data were given. This was closely followed by the numerical simulation of the building on CSI SAP 2000 software. The static and dynamic analysis was done and the results obtained were presented. From structural analysis we obtain the required results of the moments, axial forces and shears acting on the structure in order to do the verifications of the load bearing capacity of the beams, columns and footings. The modal analysis results obtained from the software were presented. With the results obtained the load bearing elements were discovered not applicable for the structural remodelling because the resisting efforts of the verified elements were below the design efforts after remodelling. The analysis was concluded with the presentation of the failure evidences made up of a table showing the safety factors of the load bearing elements, deformed shape of the structure after the application of the additional loads and lastly the overall failures that the building can be subjected to after remodelling and the failure mechanism of the structure.

GENERAL CONCLUSION AND PERSPECTIVES

The main objective of this work was to develop a forensic engineering framework for failure evidences during the structural remodelling of a reinforced concrete structure. This will permit the avoidance of failure and collapse of structures during and after their remodelling. In this work case study was the use of forensic engineering methods to determine whether or not an R+1 building in Nsimalen Yaounde could be remodelled to bear additional loads, dancing and sports activities exploitation. Modelling the structure on SAP 2000 permitted to verify if the existing load bearing elements (beams, columns and footings) of the old structure could bear the additional loads of the remodelled structure.

The methodology consisted of a general site recognition and site visit, collection of the necessary data which was made up of the architectural data, geotechnical data and structural data. With this a modelling of the structure was done on SAP 2000 software taking into consideration the new load combinations with the introduction of the additional loads on the structure. There was also the need to carry out a modal analysis to control vibration and perform static verifications of the load bearing elements of the structure being the beams, columns and footings. For the beam it was a question of verifying the steel reinforcement, the moment verification and the shear verification. As for the columns, the steel reinforcement, moment-axial force verification (M-N interaction), shear verification and slenderness verification. For the footings, it was necessary to verify the stress on the soil and the steel reinforcements. Finally, a failure characterisation was done, which led to obtaining the safety factors and failure evidence of each load bearing element and soil structure interaction for the failure of the structure.

The results of this study revealed that the existing beam reinforcement section of As = 440 mm^2 is greater than the maximum reinforcement required. So, the section of the beams will need to be increased. The shear reinforcement of the beam was verified to be acceptable. As for the columns, the reinforcement section As = 626 mm^2 proved acceptable as it falls in the range between the minimum and maximum allowed reinforcements of 204 and 1500mm^2 respectively. Nevertheless, the column did not verify the M-N interaction as the point plotted was outside the boundary required which highlighted the fact that the column section is not applicable for the building's remodelling and new functional use. It can be remedied by increasing the concrete section. Shear was verified for the columns and considered acceptable. The slenderness verification found the columns susceptible to buckling. As for the footings, calculations showed that the maximum working stress of the building $\sigma_s = 4550 kN/m^2$ was far greater than the allowable bearing capacity of the soil which is $\sigma_{allow} = 200\ kN/m^2$. With these results it was possible to come out with the various failure evidences using the safety factors which were calculated. Most of the safety factors obtained from calculations were less than 1. Every section was by these results found to be non-applicable for the remodelling of the structure. As recommendation, the proprietor has to stop the remodelling as the results show that this structure will not be safe if remodelled. So, including a sports complex at the last floor of the building is not possible. To solve this problem, the proprietor should change the exploitation of the building.

The limitations encountered during this research difficulty to gather all the design documents and verify that the structure was also constructed following the design. This is a difficult task due to the fact that with time these documents related to the existing structure are not easy to obtain. Secondly there is equally the need to do a site investigation to be able to ascertain the reinforcement used and such procedures require sophisticated equipment. Knowing the way that foundation was realised is very important. Although difficult, this procedure or framework based on forensic engineering needs to be followed before attempting to remodel a structure.

The perspectives emitted is the application of the work methodology using data resulting from site testing to verify the effective reinforcement for each load bearing elements. Furthermore, the application of the methodology using data from accelerometer sensors to better calibrates the building numerical model.

BIBLIOGRAPHY

1. Mohammed Arafa, Husam Wadi, Mamoun A. (2018), "International Journal of Engineering and Technical Research (IJETR)", ISSN: 2321-0869 (O) 2454-4698 (P) Volume-8, Issue-12.
2. David Moore, P.E. (2004). "The Roman Pantheon: The Triumph of Concrete", Archived 6 October 2014 at the Wayback Machine, Romanconcrete.com. Retrieved 19 February 2013.
3. Ariana Z. (2016), "16 Materials Every Architect Needs to Know (And Where to Learn About Them)", ArchDaily. *Archived* from the original on 2021-07-09. Retrieved 2021-07-09.
4. Dan Bawden, CAPS, GMB, CGR, CGP, and President of Legal Eagle Contractors. (2019), "Everything you need to know about structural remodelling", Legal Eagle Contractor Newsletter 713-723-8850, Volume 20.
5. Fasi Ur Rahman (2016), "Structural failures of concrete structures", The Constructor, India, 1p.
6. Carlos Videla C. (2012), "Reinforced concrete structures failures causes and responsabilities", Videla & ASOCIADOS, Chile, 40pp.
7. Wang C. K., McGraw-Hill (1983), "Structural Analysis", ASME, Newyork, 234pp.
8. CEN (1992), ENV 1992-1-1-3 Eurocode 2: Design of concrete structures, Final Draft, Paris, Cedex, 227pp.
9. Madeh Izat Hamakareem (2001), "Failure modes in concrete beams, Flexural and shear failures",
10. Jones D.R.H (2001), "Failure analysis case studies II: a sourcebook of case studies selected".
11. Capital Geotechnical Services (2016), "Geotechnical failures", PLLC, Austin, Texas, 10pp.
12. Tyagher S. T, Akpen G. D, Obam S. O," Geotechnical failures", Department of civil Engineering, university of agriculture.
13. Roy E. Hunt (2007), "A field Guide for Geotechnical engineers", Geological Hazards, CRC Press.

14. Noon R. K. (2000), "Forensic engineering investigation", CRC Press.
15. Ratay R.T. (2017), ″An Overview of Forensic Structural Engineering," Structural Engineering International, vol. 27, no. 3, pp. 338-343.
16. Brown E. (2006), "Forensic engineering for underground construction," in Rock Mechanics in Underground Construction: (With CD-ROM): World Scientific, pp. 3-18.
17. Kardon J. B. (2012), "Guidelines for forensic engineering practice", ASCE.
18. R. T. Ratay (2010), "Forensic structural engineering handbook", McGraw-Hill, New York.
19. Carper K. L. (2000), "Forensic Engineering", Second Edition ed. CRC Press.
20. C. Madryas (2008), "Forensic investigations of buried utilities failures in Poland," Tunnelling and Underground Space Technology, vol. 23, no. 2, pp. 199-205.
21. Juárez-Ferreras R, González-Nicieza C., Menéndez-Díaz A, A. E., and Álvarez-Vigil M. Álvarez-Fernández I. (2009), "Forensic analysis of hydraulic props in longwall workings", Engineering Failure Analysis, vol. 16, no. 7, pp. 2357-2370.
22. D'Antinio et al (2016), "M-N interaction diagram", Skyciv, Sydney, 1p.
23. CEN (1997), ENV 1997-3 Eurocode 7: Geotechnical design, Final Draft, Paris, Cedex, 172pp.
24. Khan (2005), "Geotechnical Engineering", PHI, New Delhi, 496pp.

ANNEXES

IREG ENGINEERING

Bureau de Recherche Géotechnique et Génie Civil

REPUBLIQUE DU CAMEROUN
******
Paix-Travail-Patrie
******

REPUBLICOF CAMEROON
******
Peace-Work-Fatherland
******

M. ZOCK ARMAND CYRILLE

PROJET DE CONSTRUCTION D'UN IMMEUBLE
DE TYPE R+1
AU LIEU-DIT ; NTOUN
ARRONSDISSEMENT DE BIKOK
RAPPORT D'ETUDE GEOTECHNIQUE DES FONDATIONS AU PENETROMETRE DYNAMIQUE LOURD

IREG ENGINEERING
BUREAU DE RECHERCHE GEOTECHNIQUE ET DE GENIE CIVIL
B.P 791 Yaoundé Tel : 677 585 456/694 019043
NC: PO88412247348X N°RCCM: RC/YAO/2015/A/582

ETUDES GEOTECHNIQUES DES FONDATIONS AU PENETROMETRE DYNAMIQUE

LE DIRECTEUR

IREG ENGENEERING The Director

Ingénieur des Travaux Génie Civ.

Agrément N°013/A/MINTP/CAB DU 05 FEV 2021

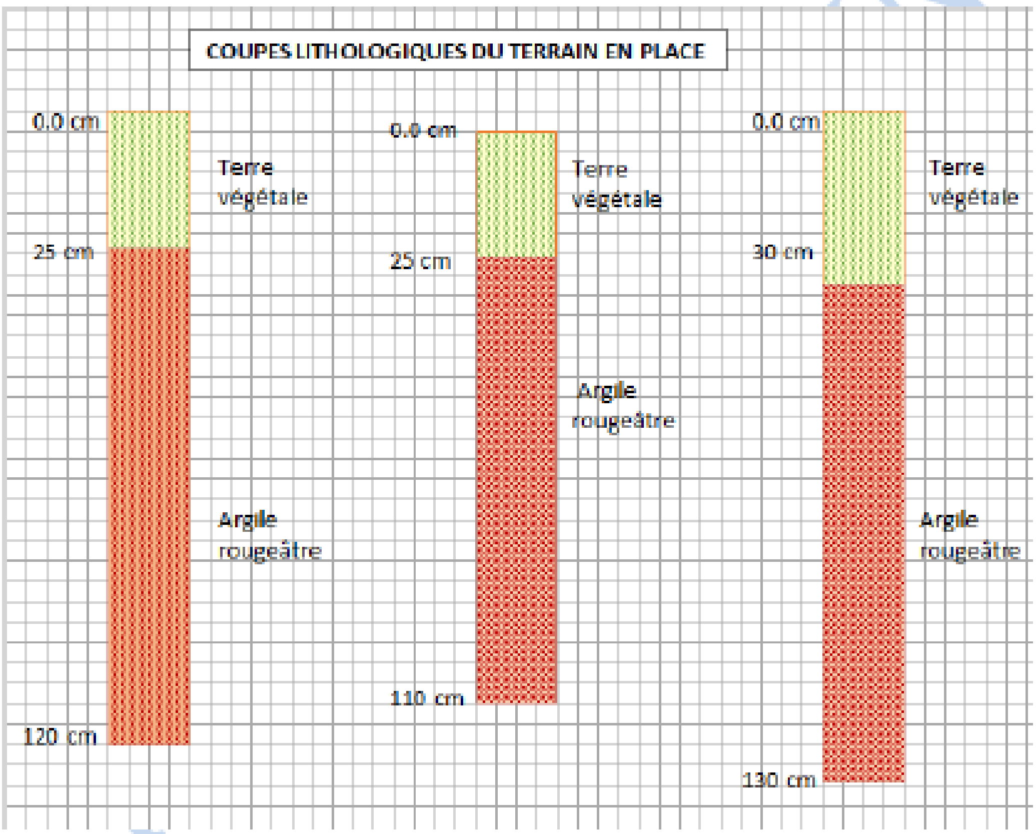
1

ANNEX I. Geotechnical report

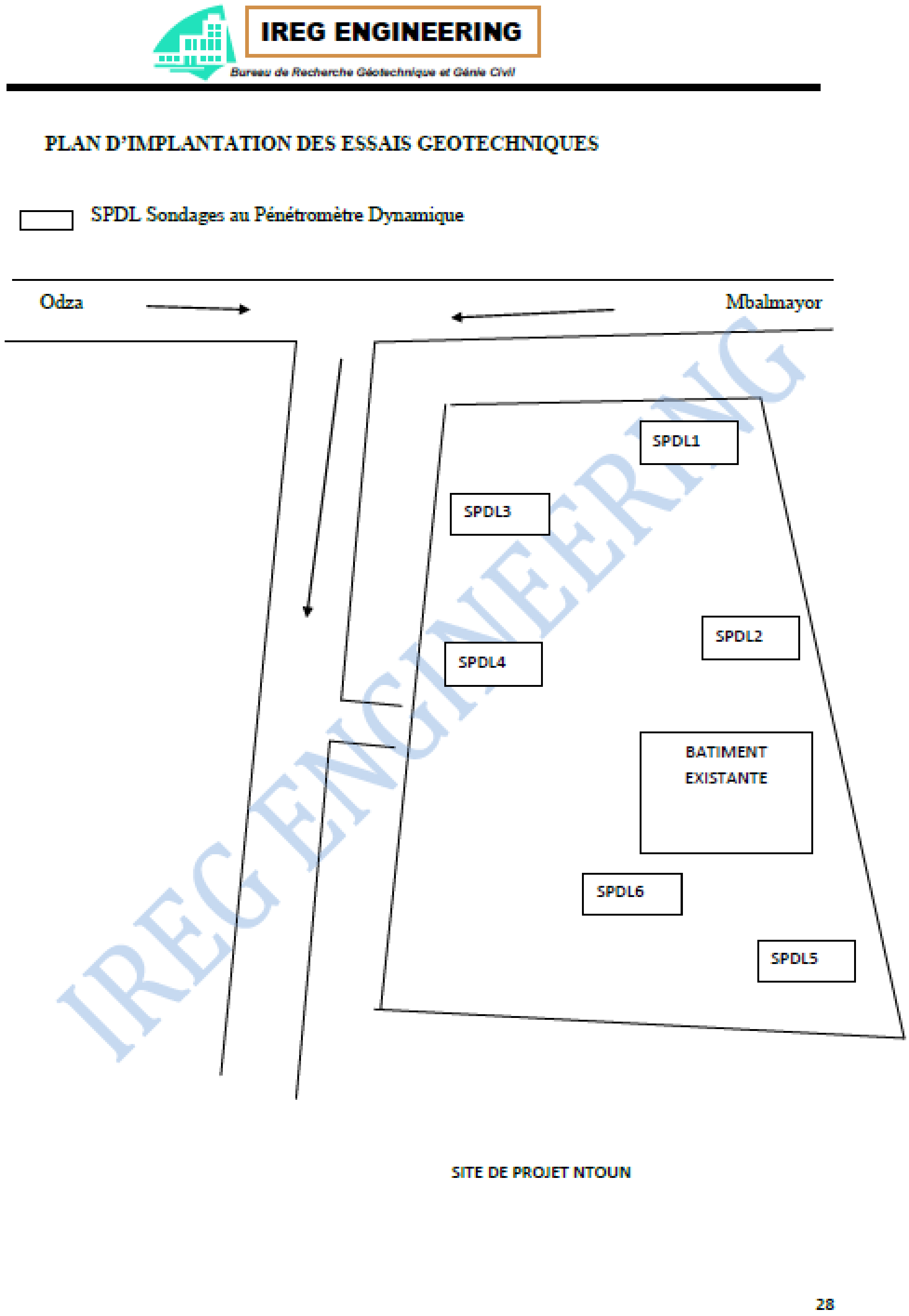
4.2. Données hydrauliques

N° du Sondage	Date de l'essai	Profondeur de sondage	Prof. Du niveau d'eau/TN (m)	Commentaires
SPDL 1	9/7/2021	6.00 m	-	Aucune venue d'eau
SPDL 2	9/7/2021	5.60 m	-	Aucune venue d'eau
SPDL 3	9/7/2021	5.20 m	-	Aucune venue d'eau
SPDL 4	9/7/2021	4.00m	-	Aucune venue d'eau
SPDL 5	9/7/2021	4.00m	-	Aucune venue d'eau
SPDL6	9/7/2021	3.60m	-	Aucune venue d'eau

ANNEX II. Hydrology data



ANNEX III. Lithology data



ANNEX IV. Implantation points for the geotechnical test

IREG ENGINEERING

Bureau de Recherche Géotechnique et Génie Civil

Tableau 1 : Contraintes admissibles (□adm) du sol sous une fondation superficielle en fonction des profondeurs d'ancrage (DF), à partir des sondages pénétrométriques

	SPDL 1			SPDL 2			SPDL 3		
Profondeur en m	σadm	KN/m²	MPa	σadm	KN/m²	MPa	σadm	KN/m²	MPa
0 à 0.20	0.84	084	0.84	0.84	084	0.84	0.42	042	042
0.20 à 0.40	0.84	084	0.84	0.84	084	0.84	0.42	042	042
0.40 à 0.60	0.84	084	0.84	0.84	084	0.84	0.42	042	042
0.60 à 0.80	1.27	127	0.127	1.27	127	0.127	0.42	042	042
0.80 à 1.00	1.27	127	0.127	1.27	127	0.127	0.42	042	042
1.00 à 1.20	1.18	118	0.118	1.18	118	0.118	0.78	078	078
1.20 à 1.40	1.18	118	0.118	1.18	118	0.118	0.78	078	078
1.40 à 1.60	1.57	157	0.157	1.57	157	0.157	1.57	157	0.157
1.60 à 1.80	1.57	157	0.157	1.18	118	0.118	1.57	157	0.157
1.80 à 2.00	1.96	196	0.196	1.57	157	0.157	1.18	118	0.118
2.00 à 2.20	2.56	256	0.256	1.46	146	0.146	1.10	110	0.110
2.20 à 2.40	1.83	183	0.183	1.83	183	0.183	1.10	110	0.110
2.40 à 2.60	2.56	256	0.256	2.56	256	0.256	1.46	.146	0.146
2.60 à 2.80	2.56	256	0.256	2.56	256	0.256	1.46	146	0.146
2.80 à 3.00	2.56	256	0.256	2.56	256	0.256	1.46	146	0.146
3.00 à 3.20	3.09	309	0.309	3.09	309	0.309	2.06	206	0.206
3.20 à 3.40	2.74	274	0.274	2.74	274	0.274	2.74	274	0.274
3.40 à 3.60	2.40	240	0.240	2.40	240	0.240	2.74	274	0.274
3.60 à 3.80	3.43	343	0.343	3.43	343	0.343	2.40	240	0.240
3.80 à 4.00	3.43	343	0.343	3.43	343	0.343	1.71	171	0.171
4.00 à 4.20	3.55	355	0.355	3.55	355	0.355	1.94	194	0.194
4.20 à 4.40	2.58	258	0.258	2.58	258	0.258	2.26	226	0.226
4.40 à 4.60	2.90	290	0.290	6.45	645	0.645	2.90	290	0.290
4.60 à 4.80	3.23	323	0.323	10.65	1065	0.1065	3.55	355	0.355
4.88 à 5.00	3.55	355	0.355	11.29	1129	0.1129	5.49	549	0.549
5.00 à 5.20	3.05	305	0.305	11.28	1128	0.1128	10.36	1036	0.1036
5.20 à 5.40	6.10	610	0.610	11.89	1189	0.1189			
5.40 à 5.60	10.06	1006	0.1006	12.80	1280	0.1280			
5.60 à 5.80	10.67	1067	0.1067						
5.80 à 6.00	11.28	1128	0.1128						
6.00 à 6.20	11.26	1126	0.1126						
6.20 à 6.40	12.13	1213	0.1213						

12

ANNEX V. Bearing capacity evaluation sheet 1

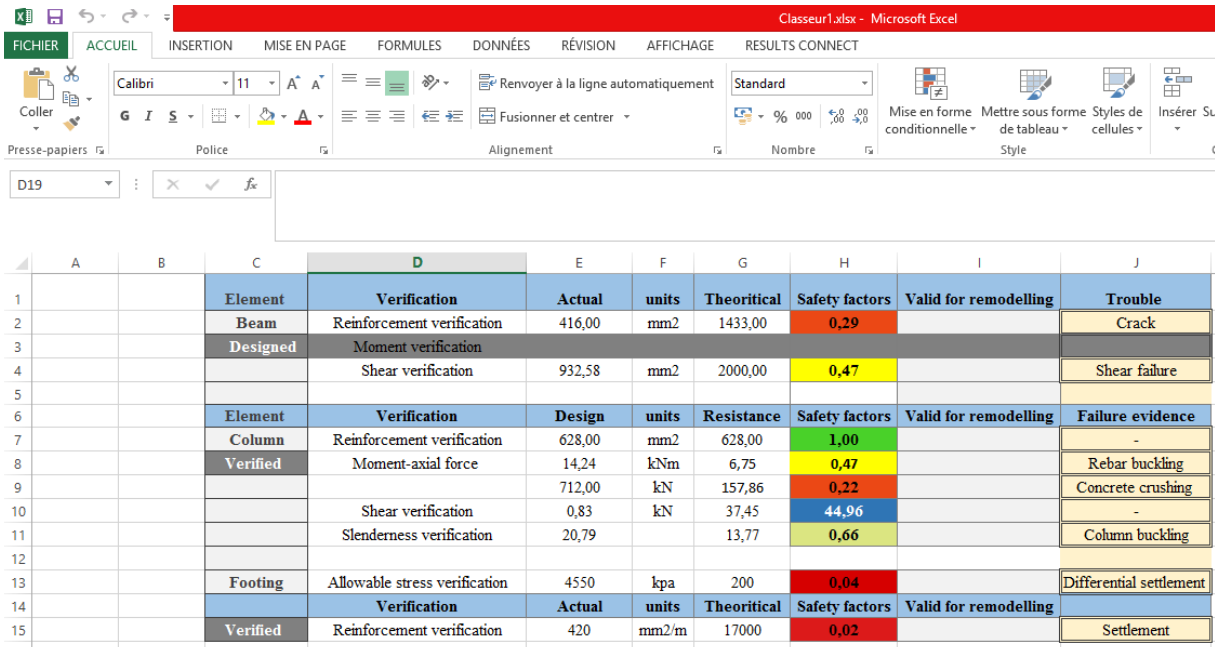
IREG ENGINEERING

Bureau de Recherche Géotechnique et Génie Civil

	SPDL 4			SPDL 5			SPDL 6		
Profondeur en m	σadm	KN/m²	MPa	σadm	KN/m²	MPa	σadm	KN/m²	MPa
0 à 0.20	1.67	167	0.167	1.67	167	0.167	2.09	209	0.209
0.20 à 0.40	1.25	125	0.125	1.67	167	0.167	1.67	167	0.167
0.40 à 0.60	1.25	125	0.125	1.67	167	0.167	1.67	167	0.167
0.60 à 0.80	1.67	167	0.167	1.67	167	0.167	1.67	167	0.167
0.80 à 1.00	1.67	167	0.167	1.67	167	0.167	2.09	209	0.209
1.00 à 1.20	1.55	155	0.155	1.55	155	0.155	1.94	194	0.194
1.20 à 1.40	1.16	116	0.116	1.16	116	0.116	1.94	194	0.194
1.40 à 1.60	1.94	194	0.194	1.16	116	0.116	1.16	116	0.116
1.60 à 1.80	1.94	194	0.194	1.55	155	0.155	1.16	116	0.116
1.80 à 2.00	3.10	310	0.310	1.55	155	0.155	1.16	116	0.116
2.00 à 2.20	2.53	253	0.253	1.45	145	0.145	1.45	145	0.145
2.20 à 2.40	2.53	253	0.253	1.45	145	0.145	1.09	109	0.109
2.40 à 2.60	3.26	326	0.326	1.81	181	0.181	1.09	109	0.109
2.60 à 2.80	4.70	470	0.470	1.81	181	0.181	1.45	145	0.145
2.80 à 3.00	6.87	687	0.687	3.26	326	0.326	1.45	145	0.145
3.00 à 3.20	7.80	780	0.780	3.73	373	0.373	1.36	136	0.136
3.20 à 3.40	8.48	848	0.848	6.78	678	0.678	8.57	857	0.857
3.40 à 3.60	9.83	983	0.983	10.17	1017	0.1017	13.37	1337	0.1337
3.60 à 3.80	11.53	1153	0.1153	11.19	1119	0.1119			
3.80 à 4.00	11.53	1153	01153	14.24	1424	0.1424			

20

ANNEX VI. Bearing capacity evaluation sheet 2



Element	Verification	Actual	units	Theoritical	Safety factors	Valid for remodelling	Trouble
Beam	Reinforcement verification	416,00	mm2	1433,00	0,29		Crack
Designed	Moment verification						
	Shear verification	932,58	mm2	2000,00	0,47		Shear failure
Element	Verification	Design	units	Resistance	Safety factors	Valid for remodelling	Failure evidence
Column	Reinforcement verification	628,00	mm2	628,00	1,00		-
Verified	Moment-axial force	14,24	kNm	6,75	0,47		Rebar buckling
		712,00	kN	157,86	0,22		Concrete crushing
	Shear verification	0,83	kN	37,45	44,96		-
	Slenderness verification	20,79		13,77	0,66		Column buckling
Footing	Allowable stress verification	4550	kpa	200	0,04		Differential settlement
	Verification	Actual	units	Theoritical	Safety factors	Valid for remodelling	
Verified	Reinforcement verification	420	mm2/m	17000	0,02		Settlement

ANNEX VII. Excel safetufactor calculation sheet