REPUBLIC OF CAMEROON

REPUBLIQUE DU CAMEROUN

Ecole Nationale Supérieure des Travaux Publics

DEPARTMENT OF CIVIL ENGINEERING

DEPARTEMENT DE GENIE CIVIL

MINISTRY OF HIGHER EDUCATION

MINISTERE DE L'ENSEIGNMENT SUPERIEUR

UNIVERSITÀ DEGLI STUDI DI PADOVA

DEPARTMENT OF CIVIL, ARCHITECTURAL

AND ENVIRONMENTAL ENGINEERING

ANALYSIS AND DESIGN OF INTERVENTIONS ON EXISTING REINFORCED CONCRETE BUILDING IN CAMEROON

CASE STUDY : EGLISE FRATENELLE LUTHERIENNE DU CAMEROON PAROISSE MIMBOMAN IN YAOUNDE

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

Curriculum: **Structural Engineering**

Presented by:

BILOG Samuel Joseph
Student number: **16TP21157**

Supervised by:
Prof. Eng. Carmelo MAJORANA

Co-supervised by:

Dr. Eng Guillaume Hervé POH'SIE

Eng. Giuseppe CARDILLO

Eng. Joseph FOTSO

Academic year / Année académique
2020/2021

DEDICATION

To my father

BILOG Theophile

`

ACKNOWLEDGMENT

This work would not have been completed without the combined efforts of individuals who contributed directly and/or indirectly to its realisation. I wish to express my sincere thanks and gratitude to:

- The President of the jury **Pr. NKENG George ELAMBO** for the honour of accepting to preside this jury;
- The **Examiner Pr. TALLA André** of this jury for accepting to bring his criticisms and observations to ameliorate this work;
- My supervisors Prof. Eng. **Carmelo MAJORANA,** Dr Eng. **Guillaume Hervé POH'SIE,** Eng. **Giuseppe CARDILLO** and Eng. **Joseph FOTSO** for all the guidance, advice and patience provided to me during this work;
- Professors **George NKENG** and **Carmelo MAJORANA** for all their academic and administrative support during these five years spent at ENSTP in the MEng program in partnership with the University of Padua in Italy;
- The vice-director of ENSTP, Dr **M. BWEMBA Charles Loic** for his perpetual help during our sojourn in this school;
- Prof. **MBESSA Michel**, the Head of Department of Civil Engineering for his tutoring and valuable advice;
- All the **teaching staff** of NASPW and the University of Padua for their good quality teaching and the motivation they belt in us to take our studies seriously;
- The **BILOG** family and more especially my father **BILOG Theophile** and my mother **BILOG Brigitte**, for the love, care, education and financial support during all these years, my siblings for their support and demonstration of unfailing love toward me;
- Eng. **GUITIYA Moïse** for his advice and time dedicated to helping me during this work;
- Eng. **TCHIEGO Christian** for his precious advice and help;
- All my **classmate**s and my **friends** who were a source of encouragement. As a team, together we have been able to achieve much;

ABSTRACT

The main aim of this thesis was to make a study on the analysis and design of interventions on existing reinforced concrete buildings in Cameroon. This work focused on determining the need for an intervention on the building of L'Eglise Fratenelle Lutherienne du Cameroun at Mimboman. Another main focus of this work was to make a comparative study on three different jacketing; Concrete, steel and carbon fiber reinforced polymer jacket strengthening methods and determine which of them is more suitable for the case study at hand. The evaluation of the state of the building was achieved through questionnaires and interviews. This proved that the building roof collapse was due to the excess weight of the new ceiling added to the structure. The fall of the roof led to a greater intervention in the building than just the roof restoration. A mezzanine was added to the structure and hence, new loads were considered. A load assessment on the building taking into account the action of wind on the building and the self-weight of the new metallic roof was made. This load assessment and the increase in the seating capacity of the building led to the need for the strengthening of the pillars. The axial force and the moment acting on each pillar were gotten from the numerical model of the building belt on Sap2000. The pillar with the highest amount of stress had an axial force of 572.02 N and a moment of 302 kNm. Further analysis was done considering the maximum load and a comparison between the three jacketing method was made. After designing or choosing any of the required sections for the jacket strengthening, an MN-interaction diagram of the column with the jacket section was plotted in other to verify that all the sections will resist all the incoming loads. Finally, a comparative study was carried out between the jacket methods considering the stresses, deflections and cost. Concrete jacketing was found to be the most appropriate strengthening method because it experienced lesser stress with a value of 12.24 MPa, ranked second in terms of deflection with a value of 16.47 mm and caused a 43.9% cost saving in contrast to carbon fiber jacketing. The cost of concrete jacketing all the 16 strengthen pillars was 3,800,000 FCFA whereas that of steel and carbon fiber were 4,896,000 FCFA and 6,768,000 FCFA respectively.

Keywords: Design intervention, Jacketing, deflection and stress.

RÉSUMÉ

L'objectif principal de ce mémoire était de faire une étude sur l'analyse et la conception des interventions sur les bâtiments existants en béton armé au Cameroun. Ce travail a porté sur la détermination de la nécessité d'une intervention sur le bâtiment de l'Eglise fraternelle luthérienne du Cameroun à Mimboman et sur le bâtiment. Un autre objectif principal de ce travail était de faire une étude comparative sur les différentes méthodes de chemisage; en béton, en acier et en polymère renforcé de fibres de carbone et de déterminer laquelle d'entre elles est la plus appropriée pour cette étude. L'évaluation de l'état du bâtiment s'est faite au moyen des questionnaires et des entrevues. Cela a prouvé que l'effondrement du toit du bâtiment était dû au poids excessif du nouveau plafond ajouté à la structure. La chute du toit a entraîné une intervention plus importante dans le bâtiment qu'une simple restauration du toit parce qu'une mezzanine a été ajoutée à la structure. Une évaluation de la charge sur le bâtiment tenant compte de l'action du vent sur le bâtiment et du poids propre de la nouvelle toiture métallique a été réalisée. Cette évaluation montre la nécessité de renforcer les piliers. La force axiale et le moment agissant sur chaque pilier ont été obtenus à partir du modèle numérique du bâtiment sur SAP2000. Le poteau avec la plus grande contrainte avait une force axiale de 572,02 N et un moment de 302 kNm. En tenant compte de la charge maximale sur le poteau, une comparaison entre les trois méthodes de chemisage a été faite. Après avoir conçu ou choisi la section requise pour le renforcement par chemisage, une courbe d'interaction du poteau a été tracé afin de vérifier que la section résiste les charges entrantes. Enfin, une étude comparative a été réalisée entre les méthodes de chemisage compte tenu des contraintes, des déflexions et des coûts. Le chemisage en béton s'est avérée être la méthode de renforcement la plus appropriée car elle a subi la plus petite contrainte avec une valeur de 12,24 MPa, s'est classée deuxième en termes de déflexion avec une valeur de 16,47 mm et a entraîné une économie de 43,9 % par rapport au chemisage en fibre de carbone. Le coût du revêtement en béton de tous les 16 piliers renforcés était de 3 800 000 FCFA tandis que celui de l'acier et de la fibre de carbone était respectivement de 4 896 000 FCFA et 6 768 000 FCFA.

Mots-clés : Conception d'intervention, chemisage, déflexion et contrainte.

LISTS OF FIGURES

LISTS OF TABLES

SUMMARY

LISTS OF SYMBOLS AND ABBREVIATIONS

LISTS OF ABBREVIATIONS

RC	Reinforced concrete
RCS	Reinforced concrete structure
RCM	Reinforcement for concrete material
TMT	Thermo mechanically treated
psi	Pounds per square inch
RCC	Reinforced cement concrete
RCB	Reinforced concrete building
ACI code	American concrete institute code
WSM	Working stress method
ULM	Ultimate load method
LSM	Limit state method
NDTs	Nondestructive tests
SDTs	Semi destructive tests
DTs	Destructive tests
RCPT	Rapid Chloride Permeability Test
CL	Concrete library
JSCE	Japan society of civil engineering
G	Permanent load
Q	variable loads
FRP	Fiber reinforced polymer
CFRP	Carbon fiber reinforced polymer
GFRP	Glass fiber reinforced polymer

AFRP	Aramid fiber reinforced polymer
FEM	Finite element method
EFLC	Eglise fratenelle lutherienne du Cameroun
DPS	Dynamic penetrometer survey
EN 1990	Eurocode 0: Basis of Structural Design
EN 1991-1-1	Eurocode 1: Actions on structures
EN 1992	Eurocode 2: Design of concrete structures
EN 1994	Eurocode 4: Design of composite steel and concrete structures
IS	Indian standard

LISTS OF SYMBOLS

fyd	Design strength of steel
fcd	Design strength of concrete
fyk	Characteristic strength of steel
fck	Characteristic strength of concrete
Ec	Modulus of elasticity of concrete
Es	Modulus of elasticity of steel
f'c	Compressive strength
Wc	Weight of the concrete
Gc	Shear modulus
Vc	Poisson's ratio
Ft	Tensile strength
f_v	Shear strength of concrete
$\boldsymbol{Q_d}$	peak resistance

v_b	basic wind velocity
$v_{b,0}$	Fundamental value of the basic wind velocity
C_{dir}	Wind directional factors
C_{season}	Wind season factor
v_m:	Mean wind velocity
C_r	Roughness factor
C_o	Orography factor
C_e	Exposure factor
k_T	Terrain factor
Z_o	Roughness length
I_v	Turbulence intensity
k_1	Turbulence
q_p	Peak velocity pressure
q_b	Basic velocity pressure
W_e	Wind pressure acting on the external surfaces
W_i	Wind pressure acting on the internal surfaces
A_{ref}	Reference area of the structure or structural element
$G_{k,j}$	Characteristic value of permanent action j;
$Q_{k,1}$	Characteristic value of leading variable action;
$Q_{k,i}$	Characteristic value of variable action i;
$\gamma_{G,j}$	Partial factor for permanent actions j;
γ_p	Partial factor for prestressing actions;
$\gamma_{Q,1}$	Partial factor for leading variable actions;
$\gamma_{Q,i}$	Partial factor for variable actions i;

$\boldsymbol{\psi_0}$	Factor for combination value of a variable action
$C_{S,i}$	Compressive strength of i-layer of steel in column section
Cc	Compressive strength of the concrete
$T_{S,i}$	Tension strength of i-layer of steel in column section
Y_{Cc}	Lever arms of concrete compressive strength on column section
$Y_{s,i}$	Lever arms of i-layer of steel strength on column section
$\boldsymbol{\varepsilon_{c,u}}$	Concrete strain
N	Axial force
M	Moment
$\boldsymbol{N_u}$	Designed resistance of original pillar section to axial load
A_c	Area of steel
A_s	Area of the concrete
$\boldsymbol{Z_a}$	Plastic section modulus for steel angles
b	Width of column section
h	Height of column section
L	Length of column
tf	Thickness of CFRP
n	Number of turns of CFRP
h'	Effective height
b'	Effective width
Ae	Effective area of concrete
εfu	Ultimate strain of CFRP
Ef	Elastic modulus of CFRP
f1	Compressive strength of CFRP

εfe	Effective strain of fiber
εccu	Maximum compressive strain of confined concrete with CFRP
f'cc	Maximum compressive strength of confined concrete
ε’t	The transient strain

GENERAL INTRODUCTION

The population of the planet is increasing at an alarming rate. Every year, the global population rises by 89 million people, meaning more homes and facilities are needed worldwide every day. Unfortunately, there is only so much land on which to build them. Urbanisation has been a phenomenon for thousands of years, but it has traditionally been accompanied by horizontal urbanization. Horizontal urbanization is the process of where a city spreads outwards across the ground, increasing the overall surface area of the city itself. A 2014 United Nations report estimates that the number of people living in urban areas will increase from 54 % to 66 % by 2050 (Boonedam, 2019).

This overall increase in the population of the world has also greatly affected Africa and Cameroon. In the search for a better life in an urban area, a massive rural exodus has been experienced over the years in Cameroon. With this growing pressure on housing, energy and infrastructure, governments have been looking at ways to improve urbanization. The solution is vertical urbanization. Vertical urbanisation looks to build upwards instead. The idea is based on the fact that doing so allows you to build more within a smaller area of land. Also, this will lead to a reduction in global warming since the gas emission caused by long distant transport will drop because jobs will be closer to home.

Therefore, in the upcoming years, an increase in high-rise buildings will be experienced in Cameroon. But since vertical urbanization comes mostly when the city experiences saturation, it becomes detrimental to construct high-rise buildings with the minimum destruction of the already existing buildings. This is achieved by performing analysis and designing interventions on the existing concrete building. These interventions permit the strengthening of the structural elements of the building thereby increasing the capacity of the structure without necessarily destroying the existing building.

Interventions on an existing building are not only made when there is a need for an increase in the building capacity but they can also be done to increase the resistance of the existing building to seismic forces. Also, defects in the building can lead to an analysis followed by the design of intervention for the repair of the structure.

Throughout the years, different strengthening technics were developed to improve the existing concrete building. But the need of finding more cost-efficient, stronger, easy to implement and environmentally friendly ways of strengthening a building has created the desire of researchers engaged in this field of study. The discovery of new materials like carbon fiber

reinforced polymer opened a new perspective to structural enhancement. But since this material is relatively new research is still ongoing in finding ways of better understanding and normalizing the codes for global use of the material in structural engineering.

This study aimed to present the different forms of analysis that can be done on a building in other to make a correct assessment of the state of the building and to design the appropriate intervention required to strengthen the building. A comparative study between old and more recent methods of intervention was made. In this study, a comparison between the different jacketing technics is done. The old concrete jacket technic will be compared to the more recent steel jacketing and the new carbon fiber reinforced polymer jacket. The adequacy of using these technics in terms of cost in a particular case study in Cameroon is studied. The methods for verification of the structure strengthened with these jacketing technics using an interaction diagram are presented in this work.

CHAPTER 1 : LITERATURE REVIEW

Introduction

Today, second only to water, concrete is the most consumed material, with three tonnes per year used by every person in the world. Twice as much concrete is used in construction as all other building materials combined (Colin, 2014). In a note on the Cameroonian cement sector, published by the ministry of economy, it is revealed that the demand for cement will grow by 10% annually considerably thanks to the implementation of various major projects (ports, dams and roads). With time, RCB progressively deteriorates. Hence to have a building with restored or increase strength, there is a need to find remedial solutions for the different deterioration it suffered over time. A building initially constructed for a specific function can, later on, be modified into a building with a different use, hence leading to a structural intervention on the building. The technological rise in the construction industry has led to a deeper and well-structured analysis that can be carried out on a building in other to better assess the state of the building. Analysis of the building should be performed followed by a design of the interventions that could be carried out in other to restore the building to its original or even a more suitable state according to the need at hand. In this chapter, an attempt will be made to explain what reinforced concrete buildings are and how they are designed, the different failures that may occur over time, and how analysis and the design of interventions could be performed.

1.1. Reinforced concrete structure

Concrete has always been a useful material, but as architecture and building design developed, it is becoming more and more the material of choice for a variety of structures and purposes. Reinforced Concrete Building (RCB) is seen from the traditional strength of a concrete dam to the acoustic performance of a concrete concert hall. Despite the great rise of Reinforced concrete buildings in the building construction world this recent century, great studies have been performed in other to make sure that reinforced concrete buildings are safer and more sustainable. Trying to have a more profound understanding of RCB, further analysis will be made on the structural elements of RCB, their design methods and the failure that occur in RCB.

1.1.1. Structural elements

Are reinforced concrete building can be looked upon as a combination of different structural elements such as slabs, beams, columns, footing and wall that when aligned and connected properly, makes up the building Structure. These different structural elements of a reinforced concrete building and how they are aligned can be seen in figure 1.1.

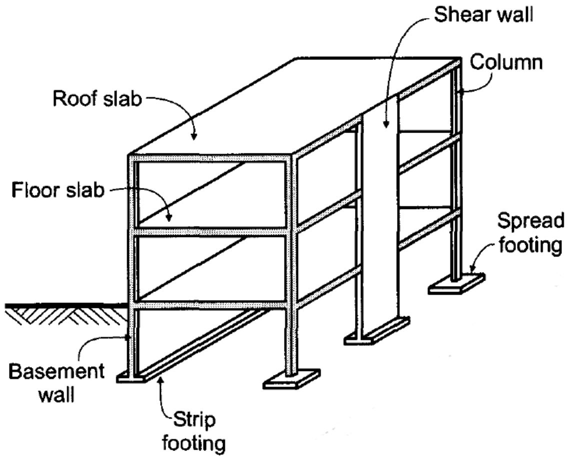


Figure 1.1. Structural elements of a reinforced concrete building (Engineersdaily, 2014)

1.1.2. Design method of reinforced concrete buildings

A design method or philosophy is a set of assumptions and procedures which are used to meet the conditions of serviceability, safety, economy and functionality of the structure. Several design philosophies have been introduced from different parts of the world. There are 3 main Concrete design methods; working stress method, ultimate strength method, and limit state method are used for the design of reinforced concrete, steel, and timber structures. In the following lines, we are going to examine the 3 main concrete design methods.

1.1.2.1. Working Stress Method (WSM)

This was the traditional method of design not only for reinforced concrete but also for structural steel and timber design. The method assumes that the structural material behaves in a linear elastic manner and that adequate safety can be ensured by suitably restricting the stresses in the material induced by the expected "working loads" on the structure. As the specified permissible stresses are kept well below the material strength, the assumption of linear elastic behaviour is considered justifiable. The ratio of the strength of the material to the permissible stress is often referred to as the factor of safety. However, the main assumption of linear elastic behaviour and the tacit assumption that the stresses under working loads can be kept within the 'permissible stresses' are not found to be realistic. Many factors are responsible for this such as a long-term effort of creep and shrinkage, the effects of stress concentrations, and other secondary effects. All such effects result in significant local increases in a redistribution of the calculated stresses. The design usually results in relatively large sections of structural members, thereby resulting in better serviceability performance under the usual working loads.

1.1.2.2. Ultimate Load Method (ULM)

With the growing realization of the shortcomings of WSM in reinforced concrete design, and with an increased understanding of the behaviour of reinforced concrete at ultimate loads, the ultimate load of design is evolved and became an alternative to WSM. This method is sometimes also referred to as the load factor methods are the ultimate strength. In this method, the stress condition at the site of the impending collapse of the structure is analyzed, and the nonlinear stress-strain curves of concrete and steel are made use. The concept of 'modular ratio' and its associated problems are avoided entirely in this method. The safety measure design is introduced by an appropriate choice of the load factor, defined as the ratio of the ultimate load to the working load. The ultimate load method makes it possible for different types of loads to be assigned different load factors under combined loading conditions, thereby overcoming the related shortcoming of WSM. This method generally results in more slender sections, and often economical designs of beams and columns, particularly when high-strength reinforcing steel and concrete are used. However, the satisfactory 'strength' performance at ultimate loads does not guarantee satisfactory 'serviceability' performance at the normal service loads. The designs sometimes result in excessive deflections and crack widths under service loads, owing to the slender sections resulting from the use of high-strength reinforcing steel and concrete. The distribution of stress resultants at ultimate load is taken as the distribution at the service loads, magnified by the load factor(s); in other words, the analysis is still based on linear elastic theory.

1.1.2.3. Limit State Method (LSM)

The philosophy of the limit state method of design represents a definite advancement over the traditional design philosophies. Unlike WSM which based calculations on service load conditions alone, and unlike ULM, which based calculations on ultimate load conditions alone, LSM aims for a comprehensive and rational solution to the design problem, by considering safety at ultimate loads and serviceability at working loads. The LSM philosophy uses a multiple safety factor format that attempts to provide adequate safety at ultimate loads as well as adequate serviceability at service loads, by considering all possible 'Limit State'. A limit state is a state of impending failure, beyond which a structure ceases to perform its intended function satisfactorily, in terms of either safety or serviceability that is, it either collapses or becomes unserviceable. There are two types of limit states Ultimate limit states (limit states of collapse) which deal with strength, overturning, sliding, buckling, fatigue fracture etc. Serviceability limit states that deal with discomfort to occupancy and/ or malfunction, caused by excessive deflection, crack width, vibration leakage etc., and also loss of durability etc.

1.1.3. Failure of reinforced concrete buildings

A reinforced concrete building may experience a phenomenon known as structural failure or collapse. Structural failure is the total or partial loss of the structure's integrity and its load-bearing capacity. Under the failure of RCB, there will be a discussion on the factors that causes failure and the type of failure that may occur in a reinforced concrete building.

1.1.3.1. Factors that cause failure in a reinforced concrete building

The factors that cause the failure of a reinforced building are an incorrect selection of concrete materials, design calculation and detailing errors of concrete structures, improper construction techniques and insufficient quality control.

a. Incorrect selection of concrete materials causes failure of a structure

Normally, the selection of materials required for a given project should be acceptable according to code specifications to use suitable materials. If materials are selected properly and meet code requirements, then they will meet the conditions of the place where the material is placed such as soil conditions. Failure in material selection is one of the major factors that detrimentally affect the strength and safety of the structure and eventually could lead to failure. Various detrimental factors such as the presence of sulfide in soil or groundwater and the occurrence of freezing and thawing should be carefully considered while materials are selected to prepare the concrete mixture.

b. Design calculation and detailing errors of concrete structures

It is considered important to practice substantial care while design calculation is conducted otherwise undesired events and costly structural improvement would prevail. That is why it is recommended to conduct an entire design check to guarantee that reinforced concrete section sizes, thicknesses, and reinforcement spacing and sizes are sufficient to support the most critical load combinations. Not only does the check need to involve the entire stability of the structure but also its serviceability and robustness. Regarding detailing, it is the most well-known factor that leads to initiate cracks and sometimes failure of the structure. It is advised to use correct, robust and efficient arrangements for the structure. The building should be constructed in such a way that prevents water to remain on the structure and consequently deteriorate the structure. Movement joints should be placed properly to avoid crack development. Code specifications regarding reinforced concrete elements shall be followed correctly. For example, concrete cover which protects steel bars from aggressive attacks and fire, maximum and minimum reinforcement ratio, steel bar spacing which restricts cracks, lap length and anchorages.

c. Improper construction techniques and insufficient quality control

Improper construction methods, poor workmanship, low-quality material and insufficient supervision of the construction process would cause several problems that reduce the performance of concrete structures substantially and subsequent failure.

1.1.3.2. Types of failure in a reinforced concrete building

When talking of the type of failures that may occur in a reinforced concrete building, there is the crushing failure of columns, shear cracking of columns, cracking of column-beam junction, short column effect, and reinforcing bars pull out.

a. Crushing failure of columns

When reinforced concrete columns are subjected to intense loading conditions, both steel and concrete yield and the column fails. Earthquake loads that apply lateral stresses highly affect the solidity of the column which may get crushed and lose its bearing capacity.

b. Shear cracking of columns

Reinforced concrete columns may undergo shear failure mostly caused by seismic forces. The cracks usually appear diagonally and may take a spiral shape when the structure experience twisting.

c. Cracking of column-beam junction

The joint of a column and a beam usually experience high bending and axial stresses which may lead to severe cracking reducing the strength of the junction.

d. Short Column Effect

This phenomenon is related to the construction of infills walls attached to the columns. The interaction of the column with the wall leads to high-stress concentration. The column is restricted by the wall and therefore, its lateral deformability (compared to its height) is highly increased. Consequently, a short column is much stiffer than other columns and accumulates higher shear stress which leads to diagonal cracking and failure.

e. Reinforcing bars pull out

Reinforcing bars can be pulled out due to tensile stresses caused in the column when their anchor length inside the column is inadequate and they cannot fully reach their tensile strength.

1.2. Analysis on existing reinforced concrete building

In other to know the exact intervention that is necessary for a given building, engineering judgment based on rational, scientific principles is critical in the strength evaluation of the concrete structures. The analysis or structural evaluation of the existing concrete building is a structural assessment that helps to determine the structural adequacy of existing concrete structures and also permits us to determine any form of defect that can be found in the structure. The analysis permits us to determine structures deteriorated from prolonged exposure to the environment or were damaged in an extreme event, such as a fire, earthquake, or explosion. Under the analysis of existing reinforced concrete buildings, we are going to study the different types of analysis on an existing reinforced concrete building and also the steps needed to carry out a proper analysis.

1.2.1. Types of analysis of existing reinforced concrete building

To assess an existing reinforced concrete (RC) building, a significant experimental campaign is needed. Experimental tests must be conducted in such a way as to avoid high structural damage. Non-Destructive Tests (NDTs) or Semi-Destructive Tests (SDTs) play an important role in reducing the number of specimens that can be extracted from the structure for Destructive Tests (DTs). The advantage of SDT and NDT test methods is that they can be used extensively with limited damage, without damage to the existing structures. The drawback is that the measured values are not of a mechanical nature, as in the case of DT tests, which implies a need for reliable correlation laws (Silvia, 2020).

1.2.1.1. Nondestructive tests (NDTs)

Nondestructive testing is defined as the method of inspecting, testing or evaluating materials, components or assemblies without destroying the serviceability of that part or system. Hence with this test, we can obtain the properties of the concrete without damaging or destroying the existing structure. This method of testing also helps us to investigate cracks depth, micro cracks and deterioration of concrete. These non-destructive methods may be categorized as penetration tests, rebound tests, pull-out techniques, dynamic tests, and radioactive tests.

a. Penetration Tests on Concrete

The Windsor probe is generally considered to be the best means of testing penetration. To complete a penetration resistance test, a device drives a small pin or probe into the surface of the concrete. The force used to penetrate the surface, and the depth of the hole is correlated to the strength of the in-place concrete. The figure1.2 below shows a Windsor probe system how it is mounted and the shape of the failure zone in probe penetration testing.

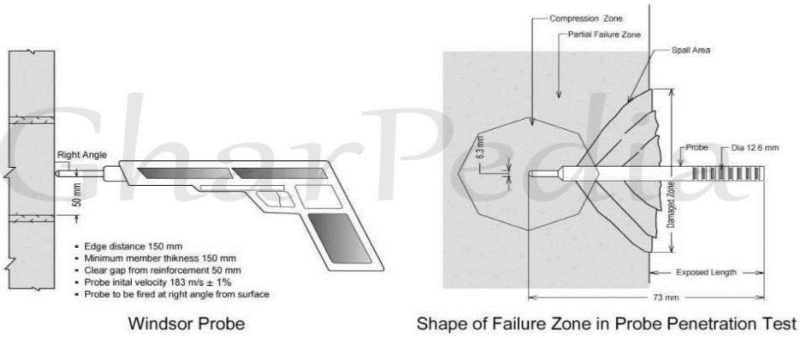


Figure 1.2. Windsor probe testing system with powder-actuated gun and steel probe (Hemali, 2017)

b. Rebound hammer (Schmidt hammer test)

This is the fastest method to evaluate the quality of concrete based on hardness, which is indicated by the rebound number. If the strength of concrete is high, the rebound number is also high. The surface hardness of the concrete gives the rebound number, which is further related to the compressive strength of the concrete. The average value of rebound hammer for a different quality of concrete as per Indian Standard IS: 13311 Part-21992 is given in Table 1.1. The relationship between the cube compressive strength and the rebound number is given in figure 1.3.

Table 1.1. Rebound number for the quality of concrete according to Indian standard (IS: 13311 Part-21992)

Instrument	Average rebound number	Quality of concrete
Schmidt Hammer N-type	>40	Very good hard layer
	30–40	Good layer
	20–30	Fair
	<20	Poor concrete
	0	Delaminated

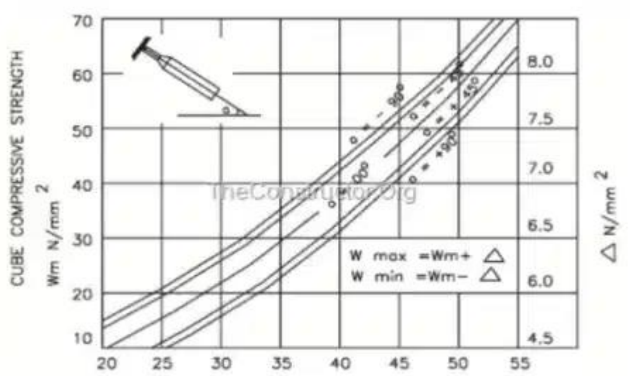


Figure 1.3. Relationship Between Cube Strength and the Rebound Number (Theconstructor, 2022)

c. Concrete carbonation

Carbonation of the concrete is the reaction of Ca(OH)2 with the atmospheric CO2 and its conversion into CaCO3. This reaction decreases the pH value of the pore water up to 8.5. As time passes, carbonation proceeds deeper into the concrete mass. If the carbonation depth reached the depth of steel in concrete, then the steel is prone to corrosion damage. By carbonation test, we can measure the carbonation depth of the concrete. To determine the path of the carbonation, drilling a hole is done in stages and the phenolphthalein solution is spread over it after every stage. As soon as the colour of the concrete becomes pink, we stop the drilling process and the depth of the hole is measured.

d. Pachometer analysis

It is a non-destructive investigation to identify reinforcement rods. This method exploits the principle of the magnetic field absorption measurement, produced by the equipment itself, which is highlighted by an analogic or digital system. It is common to work on elements about which no data concerning the reinforcement disposition, the structure building and the used material characteristics is available. Thus, it is necessary to know the actual arrangement of the reinforcing bars, their number, diameter and the measure of the concrete cover thickness without damaging the structure being assessed. The pachometer permits the assessment of the position, direction and the main and secondary reinforcement bar number.

1.2.1.2. Semi destructive tests (SDTs)

The semi-destructive testing of concrete is that, under the prerequisite that the bearing capacity of the structure to be tested is subjected to no affection, partial destructive testing is directly made on the structural concrete or, the characteristic strength as the testing result can be transformed directly from the measurements of the sample. Some examples of semi-destructive tests may include; the Core sampling test, Pullout Test, and Resistivity Test.

a. Core sampling test

Core tests are generally performed to assess whether suspect concrete in a new structure complies with strength-based acceptance criteria or not. In addition, it is critically used to determine in-place concrete strengths in an existing structure for the evaluation of structural capacity. The Concrete Core Test Is required when concrete is hardened and the laboratory cube test result shows a negative result then to ensure the quality or strength of concrete work the Core Test on Concrete is performed. In figure 1.4. below shows the concrete being drilled from an existing concrete sample.

Figure 1.4. Concrete extracted using a cutting machine and concrete sample (Bhushan, 2022)

b. Pullout Test

The main principle behind this test is to pull the concrete using a metal rod that is cast in place or post-installed in the concrete. The pulled conical shape, in combination with the force required to pull the concrete, is correlated to compressive strength. The advantage of this test is that it is easy to use and can be performed on both new and old constructions. On the other hand, this test involves crushing or damaging the concrete. A large number of test samples are needed at different locations of the slab for accurate results.

c. Resistivity Test

The Surface Resistivity Test (SRT) can be used to evaluate the electrical resistivity of water-saturated concrete to provide a rapid indication of the concrete resistance to chloride ion penetration. Based on several published research studies, measurements from this test have shown excellent correlations with other electrical indication tests, such as the Rapid Chloride Permeability Test (RCPT). The primary advantage of the Surface Resistivity Test is it takes less than 5 minutes to take readings. The more widely used RCPT test (including the sample preparation) takes more than 2 days to perform. Figure 1.5 below shows a surface resistivity test equipment

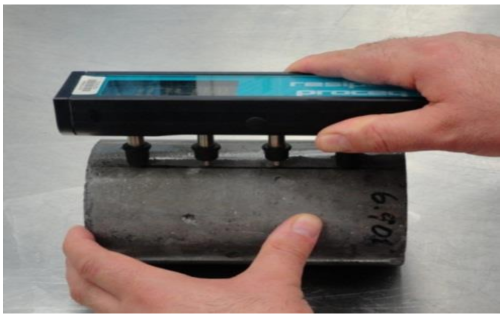


Figure 1.5. Surface Resistivity Test Equipment (Michael, 2022)

1.2.1.3. Destructive tests

The destructive test of concrete helps to understand the behaviour and quality by breaking the test specimen at certain loads. The primary step of the destructive test is to cast test specimens from freshly made concrete. It includes methods where the concrete specimen is broken to determine mechanical properties such as hardness and strength. This type of testing such as the compressive strength test and flexural strength test is very easy to carry out, easier to interpret, and yields more information.

a. Compressive strength test of concrete

The compressive strength of concrete is the ability of the concrete to withstand loads without cracking or deformation. The apparatus for performing this test is a Compression testing machine. The load at which the specimen fails measures its strength using the formula stated in equation 1.7.

$$\text{Compressive strength of the concrete} = \frac{\text{Load at which the concrete breaks}}{\text{Cross-sectional area of the specimen}} \qquad (1.7)$$

Figure 1.6. Measuring the compressive strength of concrete (Hemali, 2019)

The unit of compressive strength of concrete is N/mm^2. The test should be done at 7, 14 and 28 days. Figure 1.6 shows the setup of the apparatus used to measure the compressive strength of concrete

b. Flexural strength test

The flexural strength test measures the tensile strength of concrete. The flexural strength test of concrete measures the tensile strength of concrete through an indirect method. This test measures the ability of concrete to resist failure in bending. The modulus of rupture as shown in equation 1.8 is the measure of tensile strength. Its unit is MPa or psi.

$$\text{Modulus of rupture, } MR = \frac{3PL}{2bd^2} \tag{1.8}$$

Where:

P is the Ultimate applied load;

L is the span length;

b & d are the average width and depth of specimen at fracture.

1.2.2. Steps to carry out a proper analysis on building

The analysis procedure done on a reinforced concrete building has many basic steps. The analysis, however, should address the unique characteristics of the structure in question and the specific concerns that have arisen regarding its structural integrity. Generally, the analysis consists of:

- Defining the existing condition of the structure (This is done by first reviewing available information, then conducting a condition survey, then determining the cause and rate of progression of existing distress and finally determining the degree of repair to precede the evaluation),
- Selecting the structural elements that require detailed evaluation,
- Assessing past, present, and future loading conditions to which the structure has and will be exposed under anticipated use,
- Conducting the evaluation,
- Evaluating the results,
- Preparing a comprehensive report including the description of the procedure.

1.3. Interventions on existing reinforced concrete building

Analysis of the building helps to better design and choose the adequate interventions required by the building. When a structure is repaired, it is necessary not only to apply various repair methods to restore its performance to the originally specified level but also to reduce the effects of those factors that cause the deterioration of the performance. Under this topic of interventions on concrete buildings, a study will be made on the causes of interventions on a building, the different kinds of interventions on existing reinforced concrete buildings and the Steps to carry out a proper intervention in a reinforced concrete building.

1.3.1. Causes of interventions in a building

Different reasons may lead to an intervention on an already existing concrete building. These different reasons do not point toward the same interventions in all cases. The interventions vary according to the reasons. Some of the reasons for interventions on concrete buildings are ageing of the building, defects in the building, the collapse of the whole or part of the building, functional change or change in use, and design code update.

1.3.1.1. Ageing of the building

Due to the effect of time, the building turns to become less and less efficient and it even loses its energy efficiency. Ageing may also cause a loss in the building's strength and capacity. Hence in other to restore the building to its original state or even to a better state, some interventions need to be done as seen in figure 1.7. Most times, the ageing will lead to a complete renovation of the building.

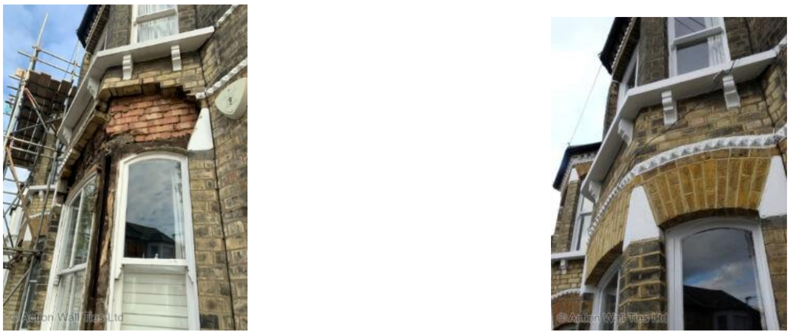
Figure 1.7. Maintenance on Aging building (Grainger, 2019)

1.3.1.2. Defects in the building

One of the major causes of interventions on a building may be the defects that arose throughout the service life of the building. some of the defects we see in a building that causes interventions are Honeycombs, exposed steel reinforcements, cracks etc. This may cause a rehabilitation of the building.

1.3.1.3. The collapse of the whole or parts of the building

Natural hazards such as earthquakes and tsunamis may lead to the collapse of the building or some parts of the building. But that's not all. Some human factors like design errors or accidents such as fire accidents may lead to the collapse of the building. Hence, this collapse in the structure may call for interventions in the structure either to build back the destroyed part or to strengthen the existing parts to withstand upcoming natural hazards. Figure 1.8 show how a repair was made on a collapsed bay.



(a) **(b)**

Figure 1.8. (a) Collapsed Bay, Before Structural Repairs. (b) After Structural Repairs (Action, 2022)

1.3.1.4. Change in building occupancy

The change in occupancy of a building may be a call for intervention. The building may initially be able to support the loads its subjected to, but with time there is an increasing demand that is placed on the building in terms of occupancy of the building, thereby resulting in an increase of load on the building. This increase in occupancy will cause the need for an intervention to strengthen the building or to redesign some elements of the building so that the building will be able to withstand the new present load and future load variations.

1.3.1.5. Functional change or change of use

A building that was initially created for a purpose, later on in his service life can be changed for another purpose. For example, a building that was used as an office could be changed into a residential home as shown in figure 1.9. But this change in the use of the building is a direct call to interventions on the building because spaces are going to be redefined, and occupancy recalculated hence a redesigning and sometimes strengthening of the building will be needed.

Figure 1.9. Centre Point Tower in London, from an office high-rise to a residential home (James, 2019)

1.3.1.6. Design code update

A building that had been constructed previously under certain construction codes, may undergo some interventions so that the building will be able to satisfy the actual more modern construction code standards.

1.3.2. Different kinds of interventions

A reinforced concrete building during his lifetime may need remedial work such as; renovation, rehabilitation, and restoration. These remedial works may involve a strengthening of the structure. And there are different sets of interventions in other to strengthen a reinforced concrete structure some of them are adding steel bracing, jacketing and concrete replacement.

1.3.2.1. Renovation

A renovation results in an essentially new building within the framework of an old one. It typically meets new building code requirements. Complete tenant evacuation during construction is necessary, although floor-by-floor renovation is sometimes possible. Its goal is a building that is like new, rather than one that is fully repaired. This term often is used interchangeably with rehabilitation, but should not be. Its goal is a building that is like new, rather than one that is fully repaired.

1.3.2.2. Rehabilitation

Rehabilitation is generally considered to be less substantial than renovation even though it usually involves the repair of all the building’s basic systems and elements of construction. Repair may include replacement or strengthening of deficient or damaged structural elements. During rehabilitation, the building's systems are brought into general conformance with local codes and ordinances.

1.3.2.3. Restoration

A restoration project attempts to restore a building to its original condition or its condition at a certain date. Decisions are generally made based on historic data, early photographs or original architectural documents. For example, an old colonial building could be restored to its original state so that the cultural inheritance of that building may be preserved.

Retrofit is a process of adding some new features that were not there before. Retrofitting in the construction industry refers to the re-strengthening of the existing structure making it more resistant to loads and external actions. Retrofitting done for your home makes it resistant to seismic activity caused by earthquakes or high winds. Retrofitting can generally be classified into two categories: Global and local retrofitting.

1.3.2.4. Global retrofitting method

Global retrofitting techniques target the resistance of the whole building. It includes adding of shear wall and adding of steel bracing. These methods increase the global capacity (strengthening).

a. Adding Steel Bracing

Steel bracing is an effective solution in the retrofitting of the building when large openings are required. Potential advantages due to higher strength and stiffness and opening for natural light can be provided. The amount of work is also less so foundation cost may be minimized and adds much less weight to the existing structure. Figure 1.10 shows an example of retrofitting by steel bracing support

Figure 1.10. Retrofitting by steel bracing support (Bhushan, 2022)

b. Addition of shear wall

Adding concrete walls by infilling certain frame bays with reinforced concrete is popular for seismic retrofitting, but is covered by codes only if the connection of the old concrete to the new ensures monolithic behavior. Addition of new RC walls is one of the most common methods used for strengthening of existing structures. This method is efficient in controlling global lateral drift, thus reducing damage in frame members(Chandrakar et al., 2017). Figure 1.11 shows the addition of shear walls to existing structures.

Figure 1.11. Adding Shear Walls to existing structures (Chandrakar et al., 2017)

1.3.2.5. Local retrofitting method

Local retrofitting techniques target the resistance of individual members. The local technique mostly involves Jacketing. Jacketing is a technique used to increase the strength of existing structural members, for example, Columns, Beams etc, by providing a "Jacket" of additional material around the existing member. This additional material or jacket can be of several types, for example, concrete, steel or Fiber reinforced polymer etc.

a. Reinforced concrete jacketing

RC jacketing has been used extensively for strengthening and repairing deficient and damaged RC columns, respectively. In traditional reinforced concrete jacketing, the section of the column is enlarged by casting a new reinforced concrete/mortar section over a part or the entire length of the column. The new section is bonded to the original section through anchor rebars or high-strength bolts. Although this technique improves the seismic performance of the column in terms of axial load carrying capacity, flexural strength and ductility, it is costly and time-consuming due to the installation of the formwork. Furthermore, it results in a change in the cross-sectional area of the column, thereby changing the mass and stiffness of the structure, and hence reducing the natural period of the structure, which consequently results in higher seismic demands on the structure (Raza et al., 2019). In figure 1.12, the diagram for the construction technique for column jacketing is presented and also figures 1.13 and figure 1.14 gives actual images. Some structural and practical remarks when dealing with RC jacketing are given in table 1.2.

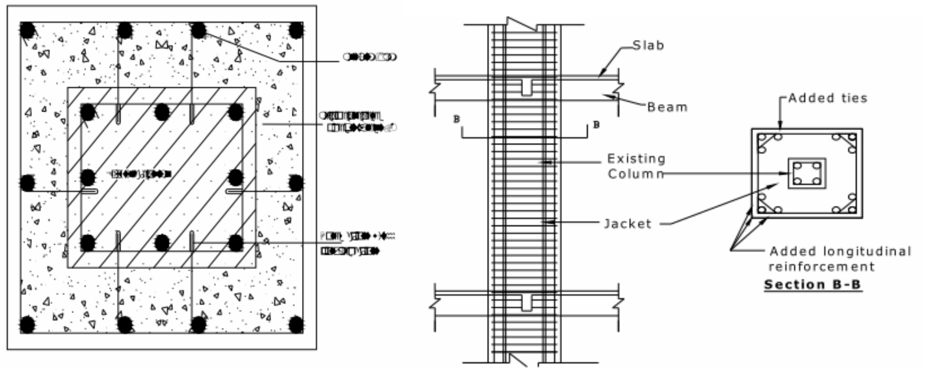


Figure 1.12. Construction Technique for Column Jacketing (Pravin et al., 2011)

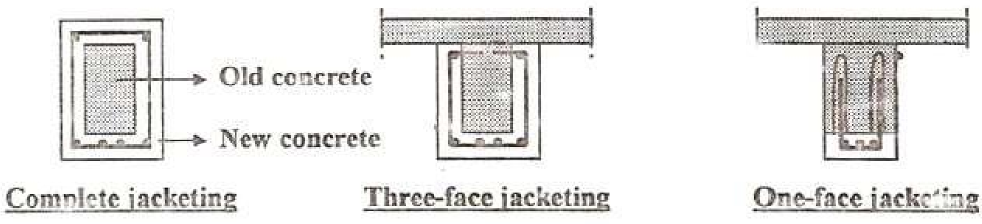


Figure 1.13. Different configurations of concrete jackets (Supervised et al., 2015)

Figure 1.14. Column retrofitting by concrete jacketing (Chandrakar et al., 2017)

Using R.C jacketing technique has advantages and disadvantages as presented in table 1.3. it is usually proposed in preference to other repair methods in the following cases:

- When the volume of repair material required is such that hand application or spraying concrete (shotcrete) is not appropriate,
- For repairs of concrete damaged by steel corrosion to protect the bars from future corrosion by restoration of an alkaline environment, similar to the original concrete,
- In areas where the repair must contribute to structural strength at high temperatures,
- When an exposed concrete finish must be maintained,
- When highly specialized workmanship is not available. The similarity with traditional cast-in-place concrete makes this method relatively easier to use than most repair techniques.

Table 1.2. Advantages and disadvantages of using RC jacketing (Pravin et al., 2011)

Advantages	**Disadvantages**
The material is mechanically and physically compatible with the original material	Heavy weight and large dimensions
Significantly enhance the strength and the stiffness	Relatively long construction period
Concrete is a durable material	Need evacuation of occupants
Normal skills are needed	

Table 1.3. Structural and practical remarks about concrete jacketing (Pravin et al., 2011)

Item	Description
Properties of the concrete jacket	▪ Match with the concrete of the existing structure.
	▪ Compressive strength greater than that of the existing structures by five N/mm^2 or at least equal to that of the existing structure.
Minimum width for jacket	▪ 8 cm if concrete cast in place or 4 cm for shotcrete
Longitudinal reinforcement	▪ The percentage of steel on the jacket should be limited to 50% of the total area of the composite section.
Shear reinforcement	▪ Ignore the effect of existing shear reinforcement
	▪ New reinforcement should have 135^o hooks and at each corner of the tie, there must be at least one longitudinal bar.
	▪ The bar used for the tie should have at least an 8 mm diameter
	▪ Multiple piece ties can be used.
Depth of jacketed beam: The following items should be taken into consideration before choosing the final depth of jacketed beam	▪ Span/depth ratio ▪ Story height ▪ Ductile behaviour
Shear stress at the interface	▪ Provide an adequate shear transfer mechanism to assure monolithic behaviour.
	▪ A relative movement between both concrete interfaces (between the jacket and the existing element) should be prevented.
	▪ Chipping the concrete cover of the original member and roughening its surface may improve the bond between the old and the new concrete.

Connectors	▪ Distributed uniformly around the interface, avoiding concentration in specific locations.
	▪ It is better to use reinforced bars (rebar) anchored with epoxy resins of grouts.

When executing concrete jacketing the following considerations should be taken into account:

- In most circumstances, and particularly in cases of concrete deterioration by corrosion, durable repairs will be obtained by cutting out the concrete all around the original reinforcement to an extent that can allow a good cleaning of the back of the bar.
- The concrete substrate must be thoroughly clean and mechanically sound to provide a rough, aggregate exposed concrete surface.
- The exposed reinforcing bars should be cleaned to the standards of new construction.
- The anchorage and cover of additional steel bars must comply with the appropriate code provisions.
- Bonding agents are usually recommended by repair manuals on the grounds of improving adhesion.
- The concrete mix should provide reasonably high workability for satisfactory placement and compaction.
- The formwork must allow good concrete placing and compaction in the recasting process, preventing the risk of trapping pockets of air. In most cases, it has to be built up in stages as the work proceeds.

b. Steel Jacketing

Steel jacketing is also an effective method to increase basic strength capacity. Steel jacketing not only provides enough confinement but also prevents deterioration of shell concrete, which is the main reason for bond failure and buckling of longitudinal bars. Steel jacketing refers to encasing the section with steel plates and filling the gap with non-shrink grout. It is a very effective method to remedy the deficiencies such as inadequate shear strength and inadequate splices of longitudinal bars at critical locations. But it may be costly and its fire resistance has to be addressed. In practice, the most commonly used strengthening technique is steel strips and angles. Steel jacketing helps to restore the strength, ductility, and energy absorption capacity of columns thus it seems to be effective in retrofitting columns. And also, the steel jacket helps to increase the flexural strength and ductile behavior of the lap-spliced column thus increasing the lateral performance of columns. Figure 1.15 shows the different forms of steel jacketing reinforcement for shear strengthening and figure 1.16 shows a practical example of steel jacking.

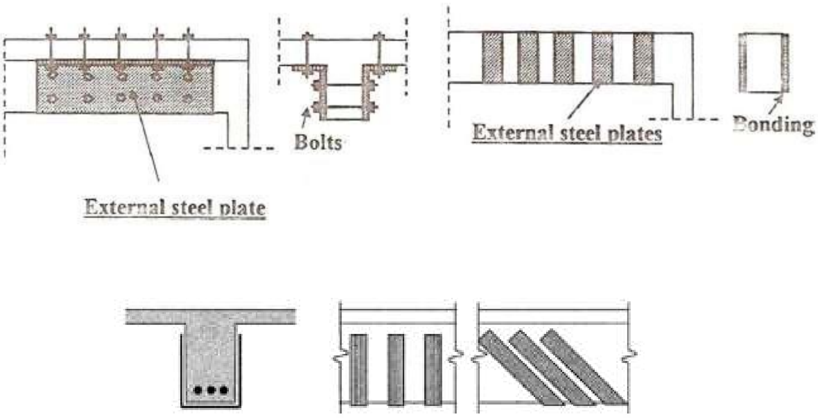


Figure 1.15. Different forms of steel jacketing reinforcement for shear strengthening (Qasem, 2015)

Figure 1.16. Shows a practical example of a steel jacking column.
(Chandrakar et al., 2017)

The insitu rehabilitation or upgrading of RC beams using bonded steel plates has been proven in the field to control flexural deformations and crack widths, and to increase the load-carrying capacity of the member under service load for ultimate conditions. It is recognized to be an effective, convenient and economic method of improving structural performance(Supervised & Ziara, 2015). In addition to this, table 1.4 presents the advantage and disadvantages of using the plate.

Table 1.4. Advantages and disadvantages of using steel plate reinforcements (Pravin et al., 2011)

Advantages	Disadvantages
Possible increase in the beam ductility. When sufficient plate is effective then shear failure will be less sudden and more gradual (plate controlled)	Uncertainty regarding the durability and the effects of Corrosion Weight of the plates (transporting, handling and installing)
Short construction period, little or no downtime due to fast hardening of the adhesive	Extensive shoring is required to hold the steel plates in the position while the adhesive cures
Simple process, fast and convenient construction	In the case of splicing, welding at the joints would destroy the adhesive bond
It may be possible to strengthen the structure while it is still in use	Relatively labour intensive.
The relatively small increase in the size and weight of the existing section	

To minimize the possibility of corrosion, all chloride-contaminated concrete should be removed before bonding, the plates must be subjected to careful surface preparation, storage and the application of resistant priming systems and the integrity of the primer must be periodically checked. When the steel jacketing method needed to be used, the following parameters are important:

- To allow the additional steel mobilization for the service loads must be removed from the structures during the strengthening execution.
- The concrete surface must be well prepared to receive the epoxy and the surface of the steel plate must also be well cleaned and polished to a high standard using grit blasting. A high roughness is inconvenient because it led to an elevated resin thick.
- The epoxy must be carefully selected for both concrete and steel.
- After mixing the two parts of the adhesive (resin and hardener) the pot life should be observed according to the manufacturer's instruction.
- The entire steel plate surface in contact with the concrete must be covered with epoxy.
- The thickness of the adhesive may be controlled carefully by metal spacers. The epoxy resin should be allowed to cure for 14 days in all cases prior to testing.

- When mechanical shear connectors are used, high attention should be done to drilling holes especially regarding the position of embedded reinforcement.

c. Fiber reinforced polymer (FRP) jacketing

Fiber reinforced polymers is a composite material which is made of two entities: a matrix, which is usually made of resin such as epoxy, and fibers. The fibers are essential to give mechanical properties to the material. There can be a mix of different types of fibers used such as Glass, carbon or Aramid, and the matrix is a resin made of polyester, or epoxy. FRP is widely used for its properties such as high strength to weight ratio, stiffness, good impact properties, and high resistance to corrosion in harsh environmental and chemical conditions, and also it causes only a minimum alteration to the geometry of structural elements than other methods.

i. Carbon fibers (CFRP)

Carbon fiber reinforced polymers (figure 1.17) is one of the stiffest and lightest composite materials, they are much more substantial than other conventional materials in many fields of application. In CFRP the reinforcement material is carbon fiber that provides the strength and stiffness and for the matrix, the commonly used polymer is epoxy resin, which binds the reinforcement in an organized way. Some advantages of CFRP are its high strength, excellent creep level, resistance to chemical effects, low conductivity, low density and high elastic modulus. The weak side of carbon fibers is being expensive and anisotropic materials with low compressive strength.

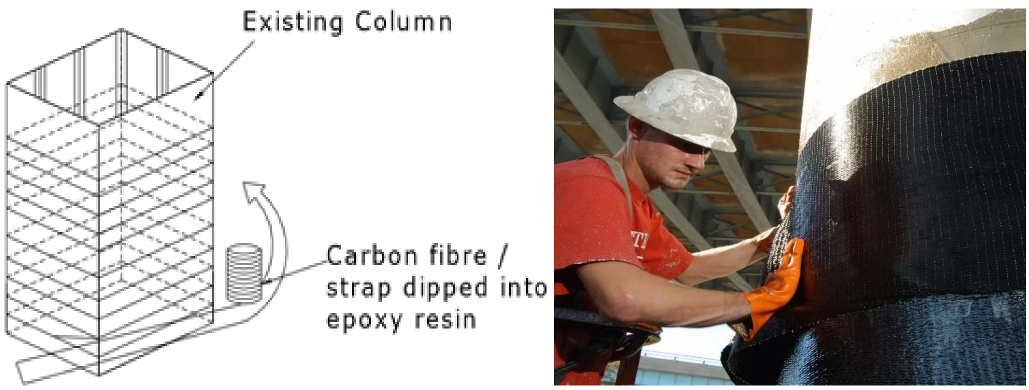
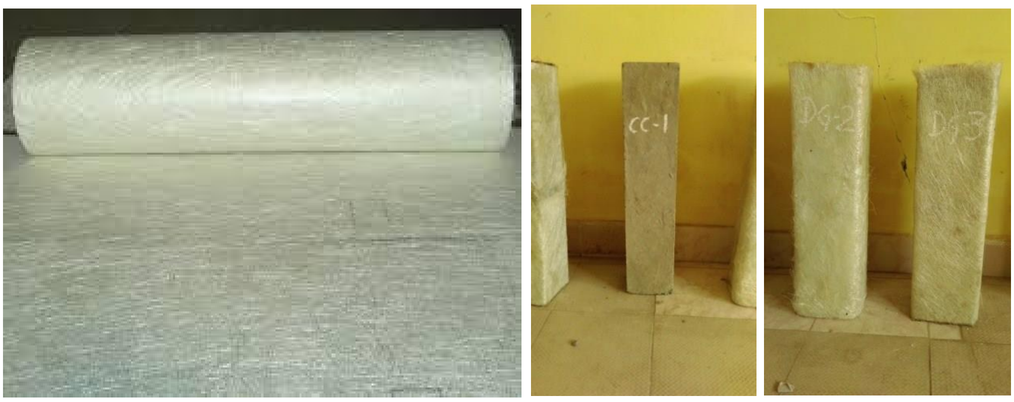


Figure 1.17. Strengthened circular column with CFRP (Pravin et al., 2011)

ii. Glass fiber (GFRP)

Glass fiber reinforced polymer (GFRP) is a fiber reinforced polymer made up of a plastic matrix reinforced by fine fibers of glass (figure1.18). Fiber glass is a lightweight, strong, and tough material used in different industries due to its excellent properties. Although strength properties are lower than carbon fiber and it is less stiff, the material is typically far less brittle and raw materials are much less expensive.



(a) **(b)** **(c)**

Figure 1.18. GFRP strengthening of column, a) GFRP Sheet, b) Control Columns, c) Double layered GFRP wrapped Columns (Sudhakar et al., 2017)

iii. Aramid Fibers (AFRP)

Aramid fiber also known as Kevlar fiber is shown in figure 1.19. The structure of aramid fiber is anisotropic in nature and usually yellow in colour. Aramid fibers are more expensive than glass moderate stiffness, good in tension application (cables and tendons) but lower strength in compression. Aramids have high tensile strength, high stiffness, high modulus and low weight and density. Impact-resistant structures have been usually produced using these materials. There are five classes of Kevlar with the different engineering properties Kelvar-29, Kelvar-49, Kelvar-100, Kelvar-119, Kelvar-129.

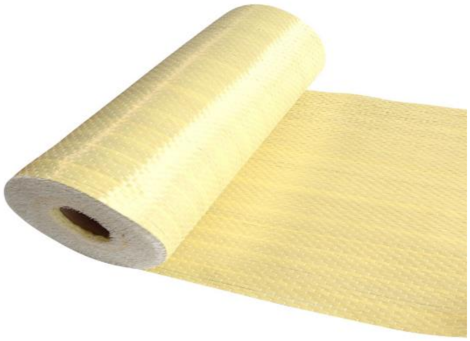


Figure 1.19. Typical aramid fiber (constrofacilitator, 2021)

1.3.3. Factors affecting the selection of strengthening method

- Strengthening a building structure for extension purpose do have several confusing methods. Not only the methods themselves but also the requirements and factors that are needed to be considered. When selecting strengthening methods and materials, outside constraints must be considered such as:
- Limited access to work areas;
- Operating schedule (when the owner will allow work to take place);
- Budget & financial limitations;
- The required useful life of the structure (The strengthening program should be consistent with the objective of the owner. For example, the minimum strengthening should be done if the structure is to be demolished in a few years);
- Environmental aspects and implications of weather;
- Effect of strengthening on loading mechanism for other adjacent structural members;
- Architectural requirements.

1.3.4. Steps to carry out a proper intervention

To perform intervention rationally and effectively, it is important to first clarify the required performance and design service life of the structure and then formulate an intervention plan that puts together the organized basic information about the target structure, the timing of and system for the implementation of the intervention, the intervention design and construction methods, the contents of post-intervention maintenance, etc.

Figure 1.20 show a flow of and intervention plan for an existing structure according to the Concrete Committee of Japan Society of Civil Engineer (JSCE). The Japan society of civil engineering guidelines specifies the standard intervention methods using cement-based materials.

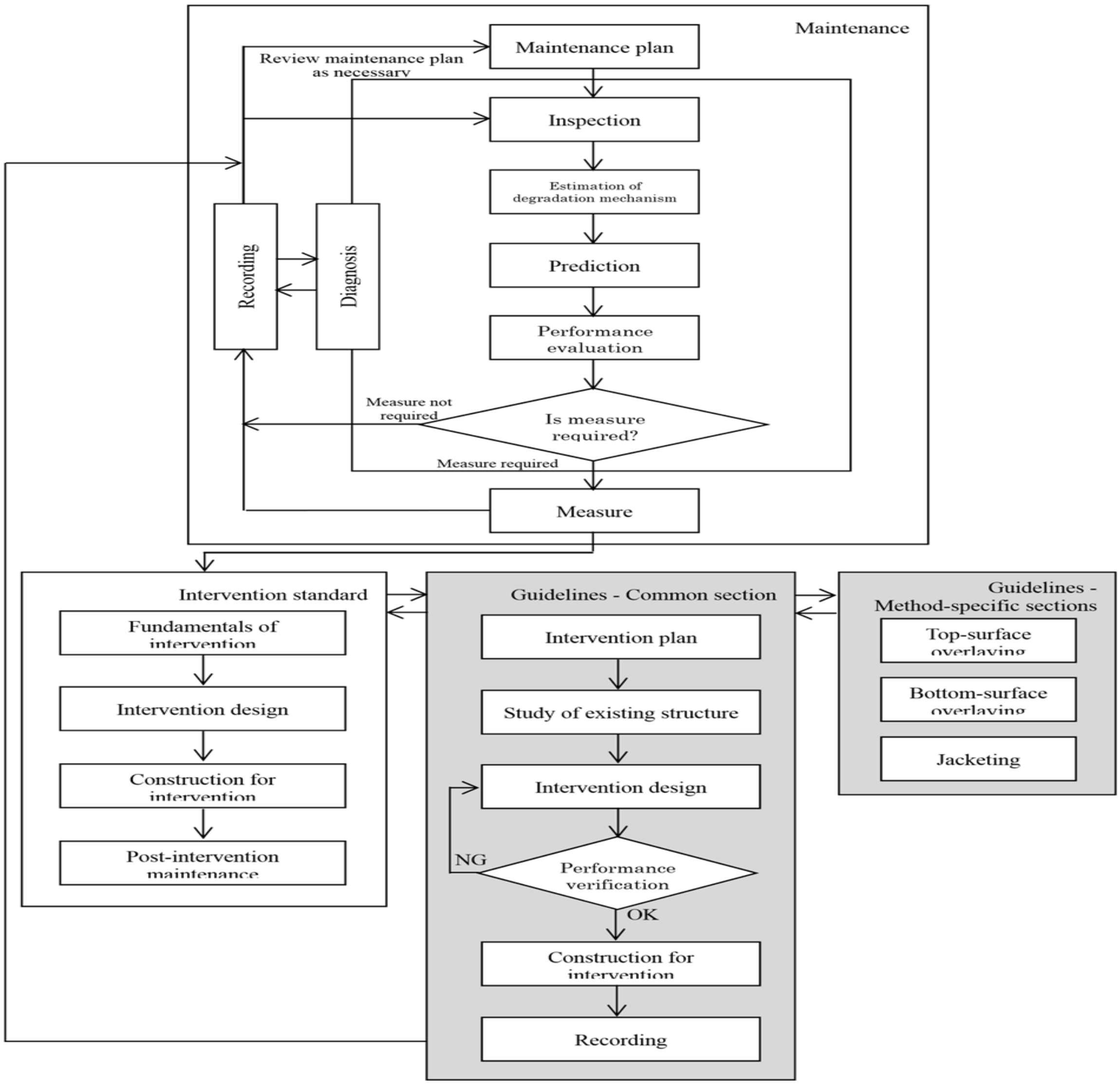


Figure 1.20. Positions of the guidelines and the flow of intervention (JSCE, 2018)

Conclusion

The objective of this chapter was to have a good understanding of RC building (and structure) and how analysis and designs of the interventions could be performed. The properties, uses and defects of reinforced concrete were noted. It was also seen that reinforced concrete buildings were made up of different structural elements such as beams, columns, slabs and many others. The different design methods and failure of the reinforced concrete building were seen. It was seen that because of the different defects and failures that may occur in the lifespan of a building, there was a need for the design of interventions to remediate those problems, thereby strengthening the building. The different kind of interventions was discussed and the steps to carry out the interventions were shown on a chart. It was also seen that a proper design of an intervention cannot be done without an adequate analysis of the existing building. In that light, types of analysis and the step to carry out a proper analysis on an existing reinforced concrete building were noted. In Cameroon, one of the main kinds of intervention in RCB pillars is jacketing. In the following part of this work, a comparative study will be made on the different types of jacking that could be done on a pillar and alternative methods of jacketing such as steel and FRP jacketing will be studied.

CHAPTER 2 : METHODOLOGY

Introduction

The previous chapter enabled a better understanding of reinforced concrete buildings and the defects and failures that may occur in the building. The need for interventions on RC buildings was spelt. Under the methodology chapter, the procedure required in other to carry out an intervention on an existing RC building in Cameroon will be discussed. The steps in other to achieve this objective begins with a general site recognition, followed by a site visit. Thirdly, data collection and fourthly a structural analysis of the building. The fifth and sixth steps are respectively the design interventions and a comparative study between different design interventions. In Cameroon, a popular structural enhancement technique used on existing buildings is jacketing. Hence, the domain of interest of this study will be the comparison between three types of jacketing methods defined in the previous chapter on a specific case study.

2.1. General site recognition

To properly carry out an analysis and interventions on a building, data related to the site are very important. This data gives a clearer view of how nature and the surrounding environment may affect the building. Information such as the geographical location of the site, climate and the relief will be useful as important parameters for the design of and appropriate intervention on the building. The general site recognition was done through documentary research on the internet.

2.2. Site visit

A site visit is going down to the place where the construction is been done in other to carry out a visual investigation. The site visit was done in two phases that involves visual observation and questionnaires and interviews.

2.2.1. Visual observation

The visual observation was made when the intervention and strengthening of the building done by the enterprise had already started. Pictures were taken of the actual building and its state. An eye inspection was made on some of the structural elements and the global state of the building.

2.2.2. Questionnaires and interviews

Information before the intervention on the building, such as the state of the building and the analysis done by the enterprise on the building was obtained using a questionnaire and an interview with the chief engineer in charge of the project.

2.2.2.1. Questionnaires

A questionnaire was formulated and given to people who frequently used the building so that the initial state of the building before the intervention should be known. The questions in the questionnaire are:

- When was the initial building constructed?
- What was the main purpose of the building?
- What were the socio-economical impacts of the building on the population?
- Which days of the week is the building most frequently used?
- On average, how many people use the building on the most populous day?
- What was the cause or reason for an intervention on the building?
- When did the collapse of the roof happen?
- What happened on the day of the collapse?
- What could be the reasons for the collapse?
- Before the collapse of the roof, were they any previous interventions on the building? If yes, which ones?
- Were they some visible defects in the building before the collapse? If yes, which ones?
- What was the need for the mezzanine in the building?

2.2.2.2. Interview

An interview was made with the chief engineer in charge of the construction on behalf of the construction firm Goglo business sarl. The interview revealed the different analyses made on the building in other to assess the state of the building. Some of the questions asked in the interview were:

- Why was there a need for an intervention on the building?
- What was the initial state of the building when the first site visit was done?
- Were they any observed defects in the building? if yes, which ones?
- What was the state of the structural elements in the building? beam, pillar, footing, slabs and roofing,
- Were are they other interesting observations made on the building?
- Any other observation made on the surrounding environment?

- Were they a need for a functional modification as an intervention in the building? if yes, why?
- Were they a need for intervention due to an increase in building capacity? If yes, why?
- Were they a need for intervention due to defects or damage found in the building? if yes, why?
- What is the different analysis or even methods of analysis made on the building?
- What were the observations or conclusions made after the analysis?
- What are the major difficulties observed when carrying out the analysis?
- What were the different interventions selected after the analysis.?
- Why were those interventions selected?
- What are the factors that favoured the selection of the intervention?
- How was the intervention put in place?
- What was the initial state of the soil?
- What geotechnical studies were carried out on the soil of the structure?

2.3. Data collection

Data collection permits a better assessment of the building. Data were collected from Goglo business sarl, the enterprise in charge of conducting the work on site. Structural data and the audit report were collected. From the audit report, the geotechnical data was accessed. The questionnaires previously mentioned were also a good source of information.

2.3.1. Structural data

Here, we are talking about the structural plans of the building. The structural plan shows the constructive disposition of the beams and columns and the type of slab adopted for the construction. There will be two categories of structural plans: initial structural plans and modified structural plans. These plans are collected from the construction firm and also from the owners of the building.

2.3.2. Audit report

A structural audit is the health examination of the building in other easily determine the residual capacity of the structure. This analysis of the building is important for a proper assessment of the building. A structural audit involves a documentary study, visual inspection, as well as nondestructive testing. A well-written report of the structural audit was taken from the enterprise in charge of carrying out the intervention on the building.

As a part of the audit report, a geotechnical survey was done on the soil. Geotechnical data permit us to know the type of soil on site and the bearing capacity of the soil. The geotechnical studies were conducted by an enterprise Sol solution Afrique central. And from their report, the geotechnical data was gotten. The tests were carried out to give the following resistance profile obtained by the Dutch formula in equation 2.1

$$Q_d = \frac{M^2 h}{M+p} \cdot \frac{1}{s} \cdot \frac{N}{e} \tag{2.1}$$

Where:

Q_d is the peak resistance;

M is the weight of the striking mass;

P is the weight of the hit mass (helmet + stake);

h is the drop height (m) permanent pile driving for several strokes;

N is the number of strokes for sinking and S is the point section.

2.4. Structural analysis

A global analysis is an analysis done on the entire building in other to get the different stresses on the different structural elements of the building. This was done using Sap2000. But in other to carry out a good structural analysis, there was the need of doing an action and load assessment followed by a numerical analysis and modelling in the software.

2.4.1. Actions and load assessment

The durability of a structure is influenced by the different actions and loads that act on the building. Hence, a correct assessment and computation of them was a subject of prime importance. The necessary codes, the evaluation of the action and the influence area where they act are to be considered.

2.4.1.1. Codes

The different codes that were used for the computations, analysis and design intervention are presented in table 2.1.

Table 2.1. Code used in the design

Code	Abbreviation
EN 1990 Eurocode 0: Basis of Structural Design	EN 1990
EN 1991 Eurocode 1: Actions on structures	EN 1991-1-1
EN 1992 Eurocode 2: Design of concrete structures	EN 1992
EN 1994 Eurocode 4: Design of composite steel and concrete structures	EN 1994
IS 15988 (2013): Seismic evaluation and strengthening	IS 15988: 2013

2.4.1.2. Evaluation of actions and loads

There are many categories of actions that act on a structure. These actions are permanent and variable or accidental loads. They represent according to EC1 direct actions which are forces applied to the structure or indirect actions which are loads including permanent loads (G), variable loads (Q) or accidental loads (A).

a. Permanent loads

Also known as static or dead loads, these are actions that act on the whole nominal life of the structure with a negligible variation of their intensity in time. These include the self-weight of the structural elements (beam, pillars roofs) and the self-weight of the non-structural elements (mechanical and electrical plant, pipework, cable trays, suspended ceilings.) present throughout the nominal life of the structure. The characteristics or design values depend on each material used in the construction.

b. Variable Loads

These are actions on structures for which their variation in magnitude with time is not negligible. These actions include imposed loads, wind loads and seismic loads.

i. Imposed loads

They designate the variable actions which depend on the nature of each category of use of the structure and these values are presented in EC1 part 1.1. These are loads different from the weight of the structure for example loads due to people, furniture etc. They depend on building occupancy and maintenance (roof elements). The church belongs to building category C2 since it is an area with fixed seats as seen in table 2.2.

Table 2.2. Categories of use (EN 1991part 1.1, 2002)

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

Knowing our category, we were able to obtain the variable loads that will be applied to the model using the values in table 2.3.

Table 2.3. Imposed loads on floors, balconies and stairs in buildings (EN 1991 part 1.1, 2002)

Categories of loaded areas	**q_k [kN/m^2]**	**Q_k [kN]**
Category A		
Floor	1,5 to 2,0	2,0 to 3,0
Staires	2,0 to 4,0	2,0 to 4,0
Balconies	2,5 to 4,0	2,0 to 3,0
Catergory 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 - C5	4,5 to 5,0	3,5 to 7,0
	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

The underlined values are the recommended values from the Eurocode. Where necessary q_k and Q_k should be increased in the design (for example for stairs and balconies depending on the occupancy and dimensions).

ii. Wind loads

As the wind blows against a building, the resulting force acting on the elevations is called the 'wind load'. The building's structural design must absorb wind forces safely and efficiently and transfer them to the foundations to avoid structural collapse. The response of a building to high wind pressures depends not only upon the geographical location and proximity of other obstructions to airflow but also upon the characteristics of the structure like the size, shape and dynamic properties of the structure. The different parameters that helped in the computation of the wind load were Basic wind velocity, mean wind, wind turbulence and peak velocity pressure, wind pressure on the surface and wind force.

- **Basic wind velocity**

Form what was stipulated in EC1-1-4, The fundamental value of the basic wind velocity, $v_{b,0}$, is the characteristic 10 minutes mean wind velocity, irrespective of wind direction and time of year, at 10 m above ground level in open country terrain with low vegetation such as grass and isolated obstacles with separations of at least 20 obstacle heights. The basic wind velocity was calculated from the expression in equation 2.1. The fundamental value of the basic wind velocity was gotten from figure 2.1.

$$v_b = c_{dir} \times c_{season} \times v_{b,0} \tag{2.2}$$

Where:

c_{dir} and c_{season} are the directional factors and season factor respectively and recommended value is to be taken as 1.0 for both according to the EC1-1-4.

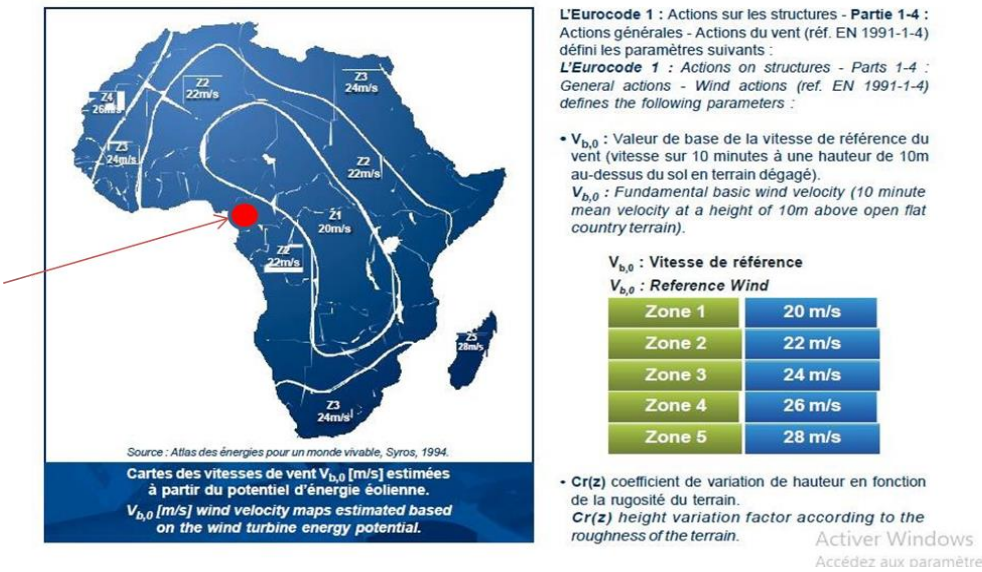


Figure 2.1. Fundamental basic wind velocity over Africa (CGTI, 2011)

Mean wind

The mean wind velocity $v_m(z)$ at a height z in m/s above the terrain depends on the terrain roughness and orography and the basic wind velocity in m/s, v_b, and was determined using the expression in equation 2.3.

$$v_m(z) = c_r(z) \cdot c_o(z) \cdot v_b \tag{2.3}$$

Where:

$C_r(z)$ is the roughness factor and

$C_o(z)$ is the orography factor, taken as 1,0.

But the terrain roughness factor was calculated using the expression in equation 2.4.

$$C_r(z) = kT \times \ ln\left(\frac{z}{z_0}\right) \text{ for } z_{min} \leq \ z \leq z_{max} \tag{2.4}$$

$$C_r(z) \ = \ C_r(z_{min}) \quad \text{for } z \leq z_{min}$$

Where:

Z_o is the roughness length (m);

k_T is the terrain factor (unitless), depending on the roughness length Z_o calculated in equation 2.5.

$$kT \ = \ 0.019 \ * \ \left(\frac{z_O}{z_{O'II}}\right)^{0.07} \tag{2.5}$$

Where:

$z_{O'II}$ = 0.05 (terrain category IV, taken from table 2.4) (m);

Z_{min} is the minimum height, z_{max} is to be taken as 200 m.

Z_0, Z_{min} depend on the terrain category (m).

Recommended values are given in Table 4.1

Table 2.4. Terrain categories and terrain parameters (EN1991-1-4, 2005)

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

- **Wind turbulence**

The turbulence intensity Iv(z) at height z is defined as the standard deviation of the turbulence divided by the mean wind velocity. Its value was calculated using the expression in equations 2.6 and 2.7.

$$I_v = \frac{k_1}{c_0(z) * In\left(\frac{z}{z_0}\right)} \quad \text{for} \quad z_{min} \le z \le z_{max} \tag{2.6}$$

$$I_v = I_v(z_{min}) \quad \text{for} \quad z < z_{min} \tag{2.7}$$

Where:

k_1 is the turbulence factor and the recommended value for k_1 is 1,0;

C_o is the orography factor and Z_0 is the roughness length.

- **Peak velocity pressure**

The peak velocity pressure $q_p(z)$ at height z in MPa, which includes mean and short-term velocity fluctuations, was determined. The recommended rule is given in the expression in equation 2.8.

$$q_p(z) = [1 + 7 \cdot I_v(z)] \cdot \frac{1}{2} \cdot \rho \cdot v_m^2(z) = c_e(z) \cdot q_b \tag{2.8}$$

Where:

ρ is the air density in kg/m^3, which depends on the altitude, temperature and barometric pressure to be expected in the region during wind storms;

$C_e(z)$ is the exposure factor given in expression in equation 2.9.

$$C_e(z) = \frac{q_{p(z)}}{q_b} \tag{2.9}$$

q_b is the basic velocity pressure given in the expression in equation 2.10.

$$q_b = \frac{1}{2} \cdot \rho \cdot v_b^2 \tag{2.10}$$

The recommended ρ value is 1,25 kg/m^3.

For flat terrain where $C_o(z)$ = 1,0. The exposure factor $c_e(z)$ is as a function of height above terrain and a function of terrain category as defined in table 2.4.

- **Wind pressure on surface**

In a Structure, we have the wind pressure acting on the internal surface and that acting on the external surface. The net pressure on a wall, roof or element is the difference between the pressures on the opposite surfaces taking into account their signs. Pressure, directed towards the surface is taken as positive, and suction directed away from the surface as negative.

The wind pressure acting on the external surfaces, W_e in MPa, should be obtained from equation 2.11.

$$W_e = q_p(z_e) \cdot c_{pe} \quad (2.11)$$

Where:

$q_p(z_e)$ is the peak velocity pressure (m/s);

z_e is the reference height for the external pressure (m);

C_{pe} is the pressure coefficient for the external pressure (MPa).

The wind pressure acting on the internal surfaces of a structure, w_i, should be obtained from equation 2.12.

$$w_i = q_p(z_i) \cdot c_{pi} \quad (2.12)$$

Where: $q_p(z_i)$ is the peak velocity pressure, z_i is the reference height for the internal pressure, c_{pi} is the pressure coefficient for the internal pressure?

- **Wind force**

The wind force, F_w (KN) acting on a structure or a structural element may be determined by vectorial summation of the forces $F_{w,e}$, and $F_{w,i}$ (KN) calculated from the external and internal pressures using the expressions in equations 2.13 and 2.14.

For external forces:

$$F_{w,e} = c_s c_d \cdot \sum_{surfaces} w_e \cdot A_{ref} \quad (2.13)$$

For internal forces:

$$F_{w,e} = c_s c_d \cdot \sum_{surfaces} w_i \cdot A_{ref} \quad (2.14)$$

Where: $c_s c_d$ is the structural factor;

A_{ref} is the reference area of the structure or structural element (m^2);

W_e is the external pressure on the individual surface at height z_e (MPa);

W_i is the internal pressure on the individual surface at height z_i (MPa).

iii. Seismic loads

The distribution of seismic activity is particularly denser in the area of Mount Cameroon and is related to the magmatic activity. Elsewhere the seismicity, as induced by the tectonic activity, is weak to moderate (Noel et al., 2014). Hence for our case study, the area of Mimboman Yaounde was considered an area with weak seismic activity. So, the seismic load was not considered in our study.

2.4.1.3. Load influence area

The loads are surface loads, and in some cases, the loads were transformed into linear loads in other to be applied in our structural model. These linear loads were applied to the beams and the load was transmitted to the pillars and then to the footings.

2.4.2. Global numerical analysis and modelling

A global numerical analysis in our case is the analysis made on the entire structure. This analysis permitted us to have the different elementary stresses on our structure such as the normal force (N), shear force (T) and bending moments (M). The software Sap2000 for the global analysis, the description of the structural element model, the load combination used in our model and the verification of our model are discussed.

2.4.2.1. Description of software SAP2000

SAP2000 is general-purpose civil-engineering software ideal for the analysis and design of any type of structural system. Basic and advanced systems, ranging from 2D to 3D, of simple geometry to complex, may be modelled, analyzed, designed, and optimized using a practical and intuitive object-based modelling environment that simplifies and streamlines the engineering process. The model domain may be component, system, or global-level in scope while encompassing sub-grade components and soil-structure interaction. Powerful built-in templates also simplify and expedite the load-application process. Seismic, wind, vehicle, wave, and thermal forces may all be automatically generated and assigned according to a suite of code-based guidelines. An unlimited number of load cases and combinations can be defined and envelope. A range of innovative analysis techniques is integrated into the capabilities of SAP2000. There is the possibility to freely supplement the standard yet sophisticated analysis process by implementing advanced features for nonlinear and dynamic consideration. This versatility makes SAP2000 a practical and productive tool for any analysis type ranging from simple static, linear-elastic to more complex dynamic, nonlinear-inelastic. Output and display options are intuitive and practical. Finalized member design, deformed geometry per load combination or mode shape, moment, shear, and axial-force diagrams, section-cut response displays, and animation of time-dependent displacements outline a few of the graphics available upon conclusion of the analysis. SAP2000 automatically generates reports for the presentation of images and data.

2.4.2.2. Structural element modelling

The structural model was belt using sap2000. In building the model, the specific distance between each pillar and the distance between the beams were clearly defined according to the plan. The structural elements that made up the model were beams and pillars. keeping in mind that the focus of our model was to get the stresses on the pillars, the roof, mezzanine and footing were given special consideration.

a. Beam and pillars

The beam and the pillars were modelled using beam elements. The beam element is relevant to use when the aim is to analyse a slender structure undergoing forces and moments in any direction. Hence, it is the perfect element to analyze the pillar and beams of the structure. Also, the aim was to predict the global behaviour of the model. Therefore, using the beam element is more efficient because of its lower computational effort. Beam theory is based on the assumption that the deformation of the structure can be determined entirely from variables that are functions of position along the structure's length.

b. The mezzanine

The mezzanine was modelled making some considerations. The bleacher and the slab of the mezzanine were considered as a distributed load on the beams that framed the structure of our mezzanine. The beam’s structure that framed the mezzanine was modelled in the software using beam elements. The larger portion of the mezzanine was supported by two pillars placed at strategic locations in other to transmit part of the weight of the structure to the ground.

c. Roof

The roof was considered as a distributed load that was applied to the structure. Some weightless beams were designed in the model in other to transmit the load of the roof from one structural span end to the other.

d. Footing

The soil structure interaction was not taken into consideration in our model. Hence, the footing was simplified into fixed support in the model.

2.4.2.3. Load combination

A load combination was defined in the software. The design was done in the Ultimate Limit State (ULS). The ULS used the maximum possible factor loading, which represents the worst-case scenario. The mathematical model used in defining the load combination is given by equation 2.15. A particular load combination was used for the loading of the roof, another for the loading on the mezzanine and a joint load combination for representing the global loading of the structure. Since the pillars on the sides of the building were the main study concern, the wind variable load was considered as the main variable action on the structure.

$$\sum_{j\geq 1} \gamma_{G,j} G_{k,j} + \gamma_p P + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma Q_{,i} \Psi_{0,i} Q_{k,i} \tag{2.15}$$

Where:

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

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

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

$\gamma_{G,j}$ is the partial factor for permanent actions j;

γ_p is the partial factor for prestressing actions;

$\gamma_{Q,1}$ is the partial factor for leading variable actions;

$\gamma_{Q,i}$ is the partial factor for variable actions I;

ψ_0 is the factor for the combined value of a variable action.

The recommended values of factor ψ_0 and the partial factors were obtained using table A.1 and table A.2 respectively. Hence, the values of the partial factor for permanent action were taken to be 1.3, that for variable actions 1.5 and the factor for combination values of variable actions 0.7. The partial factor for permanent action was taken as 1.3 because the death load of our structure is been computed automatically by the software and hence was not considered among the additional permanent action we added to our structure. Hence, the load combination for the roof, mezzanine and the global load combination is given by equations 2.16, 2.17 and 2.18 respectively.

Combination on the roof:

$$1.3G_{k1} + 1.5Q_{k1} \tag{2.16}$$

Combination on the mezzanine:

$$1.3G_{k2} + 1.5Q_{k2} \tag{2.17}$$

Global combination:

$$1.3G_{k1} + 1.3G_{k2} + 1.5Q_{k1} + 1.5 \cdot 0.7 \cdot Qk2 \tag{2.18}$$

2.4.3. Stresses on the pillar

A table was made presenting the different values of the axial forces, the moment on the x-axis and that on the y-axis for each of the pillars in our structure. From the table, the pillar with the highest amount of stress was noted and was taken as a case study for further analysis of the structure.

2.4.4. Model verification

Two different verifications were made on the model. A modal verification was made to obtain the fundamental mode of vibration of the structure and an interaction diagram was done to verify the section of the pillar. Added to that, a more local analysis verification was carried out on a pillar before the strengthening intervention was made.

2.4.4.1. Modal verification

A dynamic modal analysis was conducted on the structure in other to validate and to verify the regularity of the numerical model. A check on the model was done in other to see if the first, second and third modes of vibration were two translations and torsion respectively.

2.4.4.2. Pillar verification with Interaction diagram

This verification was done to see if the design section of the pillar given the material property was able to resist the applied stresses. This was done using the plot of the moment and axial load acting on the pillar on a column interaction diagram for that given section.

An interaction Diagram in a column is a graph that shows a plot for the axial load N that a column could carry versus its moment capacity, M. This diagram is very useful in analyzing the strength of the column which varies according to its loads and moments. The load combinations under any case that falls inside the curve are satisfactory while the load combination under any case that falls outside the curve represents a failed design.

The loads applied to the column give rise to two moments in the section each according to a principal axis. There is moment 2-2 (moment along the x-axis) and moment 3-3 (moment along the y-axis). Hence, two interactions diagram was plotted in other to carry out the column resistance verification of the moments acting along the two axes.

The curves are plotted using 5 main points and these points are computed by making different assumptions on the concrete section. For moments 3-3, the points are obtained using equations 2.19 to equation 2.20.

- Point 1 (Pure compression). In other to plot the first point, we will assume that the section undergoes pure compression. Hence, no moment (M = 0 kN) but only axial forces.

$$f_{yd} = \frac{f_{yk}}{1.15} \tag{2.19}$$

$$f_{cd} = \frac{0.85 \cdot f_{ck}}{1.5} \tag{2.20}$$

Where:

f_{yk} and f_{ck} are the characteristic strength of steel and concrete respectively (MPa).

f_{yd} and f_{cd} are the yield strength of the steel and concrete respectively (MPa). The stress and strain distribution on the concrete is presented in figure 2.2

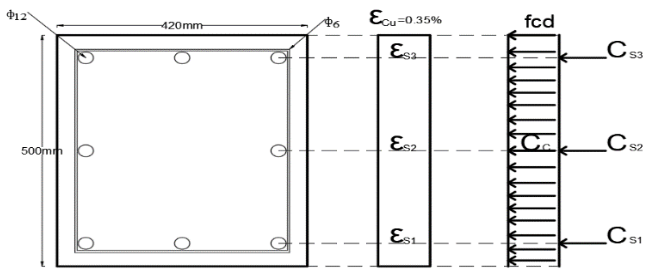


Figure 2.2. Strain and stress distribution for point 1(AutoCAD, 2022)

With a Strain of 0.35% across all the blocks, it means all our layers of steel are going to yield

$$C_{S1} = A_{S1} * f_{yd} \tag{2.21}$$

$$C_{S2} = A_{S2} * f_{yd} \tag{2.22}$$

$$C_{S3} = A_{S3} * f_{yd} \tag{2.23}$$

$$C_c = \beta_1 * (A_c - A_{s,tot}) * f_{cd} \tag{2.24}$$

Where:

C_{S1}, C_{S2}, and C_{S3} are the compressive strength (kN) from the first, second and third layer of steel respectively;

Cc is the compressive strength of the concrete;

A_{S1}, A_{S2}, A_{S3}, and A_C, are the areas of the first, second, and third layer of steel and the area of concrete respectively (mm^2);

f_{yd} and f_{cd} are the yield strength (MPa) of the steel and concrete respectively.

The axial force and moment are obtained from equation 2.25.

$$N = C_{S1} + C_{S2} + C_{S3} + C_c \tag{2.25}$$

- Point 2 (balance point). The balance point is the point when the concrete crushes and the tension steel yields.

  There the strain on the tension steel is $\varepsilon_s = \varepsilon_y = 0.196$ %

  The strain on the concrete $\varepsilon_{c,u} = 0.35$ %

  The stress and strain distribution considering the assumption made for point two are shown in figure 2.3.

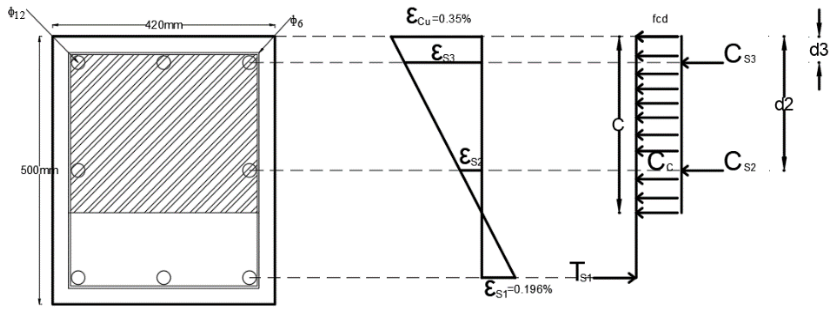


Figure 2.3. Stress and strain distribution for point 2 (AutoCAD, 2022)

The axial force value and the moment defining point 2 in the interaction diagram are obtained by equations 2.26 and 2.27 respectively. By convention, compression is considered positive and tension is considered negative.

$$N = -T_{S1} + C_{S2} + C_{S3} + C_c \tag{2.26}$$

$$M = Y_{S3} * C_{S3} + Y_{Cc} * C_c + Y_{S1} * T_{S1} \tag{2.27}$$

Where:

Y_{S3}, Y_{Cc}, and Y_{S1} are the lever arms for the stresses (m) C_{S3}, Cc and T_{S1} respectively.

- Point 3 (tension controlled). Here, the bottom steel strain has a value of 0.5 % ($\varepsilon s_1 = \varepsilon y = 0.5$ %).

  The strain on the concrete $\varepsilon_{c,u}$ = 0.35 %

  The stress and strain distribution diagram considering this assumption made is shown in figure 2.4.

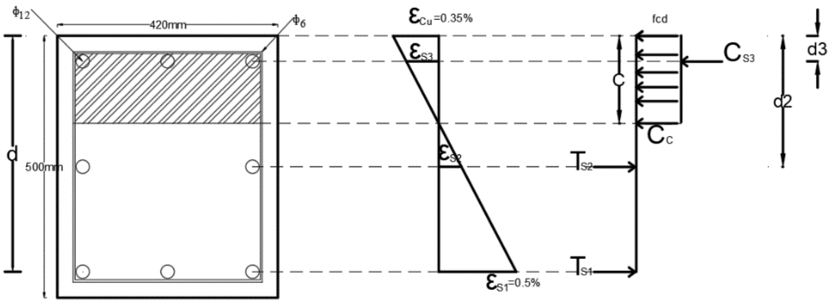


Figure 2.4. Stress and strain distribution for point 3 (AutoCAD, 2022)

The axial force value and the moment defining point 3 in the interaction diagram are obtained by equations 2.28 and 2.29 respectively.

$$N = -T_{S1} - T_{S2} + C_{S3} + C_c \tag{2.28}$$

$$M = Y_{S3} * C_{S3} + Y_{Cc} * C_c + Y_{S1} * T_{S1} \tag{2.29}$$

- Point 4 (pure bending). At this point, we will consider that there is no axial force (N = 0kN). The strain on the concrete $\varepsilon_{c,u}$ = 0.35 %
  The stress and strain distribution diagram considering this assumption made is shown in figure 2.5.

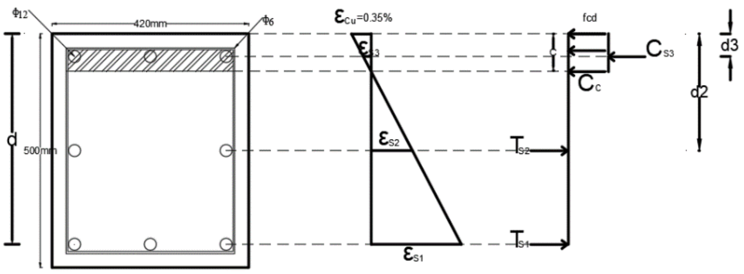


Figure 2.5. Stress and strain distribution for point 4 (AutoCAD, 2022)

The moment is obtained using an equation similar to that of equation 2.29.

- Point 5 (pure tension). Here, the concrete is assumed not to contribute at all to tension. Only the steel is responsible for the tension. So, the moment is considered null (M= 0kN). The stress and strain distribution diagram are shown in figure 2.6.

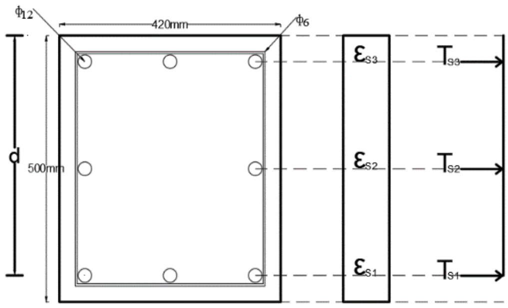


Figure 2.6. Stress and strain distribution for point 5 (AutoCAD, 2022)

The axial force for this point will be obtained using equation 2.30.

$$N = -T_{S1} - T_{S2} - T_{S3} \tag{2.30}$$

The same computation is done for the interaction diagram of moment 2-2 but considering the other principal axis. Hence the concrete section that was used in the computation is presented in figure 2.7.

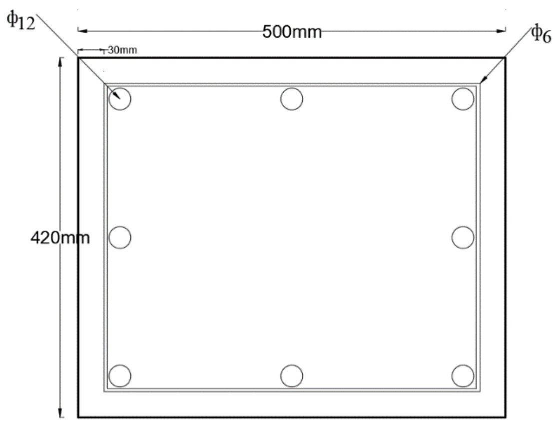


Figure 2.7. Section of column considered when computing the interaction diagram for moment 2-2 (AutoCAD, 2022)

2.4.4.3. Local analysis verification of a pillar with Abaqus

A local analysis of a pillar under the effect of the maximum axial and moment forces was done using a finite element simulation software ABAQUS. Firstly, the model was belt on the software then an analysis was carried out in other to verify if the results obtained through our mn-interaction curve were accurate.

a. Building of model on Abaqus

The ABAQUS finite element simulation software is particularly powerful, it not only solves the linear problem but also solves many difficult nonlinear problems. ABAQUS can analyze static stress, static displacement, viscoelastic response, viscoplastic response, heat conduction, mass diffusion, coupling, nonlinear dynamic stress, nonlinear dynamic displacement, transient temperature coupling, transient displacement coupling, and quasi-static (Runsheng & Yang, 2019).

Abaqus is divided into modules, where each module defines an aspect of the modelling process, for example, defining the geometry, defining material properties, and generating a mesh. As you move from module to module, you build the model from which Abaqus/CAE generates an input file that you submit to Abaqus/Standard or Abaqus/Explicit for analysis. The steps in modelling the concrete column were:

- Part

  A two-dimensional sketch of the concrete, main bar and stirrups was made. After that, a part was created for each of those constitutive elements of the column.
- Property

  Define the material properties and other section properties of the different elements that make up the column under this module
- Assembly

  The different parts of the column were assembled under this module in their exact position.
- Step

  Here, the analysis procedure and output requested were configured.
- Load

  The axial loads, bending moment and the boundary condition for the column were defined for our model.
- Mesh

  A mesh was applied to each element of the column.
- Job

  Under this module, a job was created and submitted for analysis. The analysis was monitored to check for any possible errors.
- Visualization

  The different results at the end of the analysis are seen under this module.

b. Analysis on model

Abaqus/Standard, the implicit solver, can solve for static equilibrium (the state where the sum of the forces is zero). It is also possible to include dynamic effects. If dynamic effects are not included, a static equilibrium must exist. Abaqus/Explicit always calculates dynamic equilibrium (force = mass x acceleration). There are more vibrations in an Explicit analysis compared to static implicit analysis, and the results are more oscillatory. Time has a physical meaning, and the loading rate is of importance. A static analysis, dynamic implicit and dynamic explicit analysis was carried out one at a time and verifications were made.

2.5. Design intervention

After the global study of the structure, the structure showed the need of strengthening some columns in other to increase their capacity to resist incoming loads on them. The pillar with the highest stress on it was considered as the case study for strengthening. Hence, a local analysis was carried out on the column of the building. The local retrofitting method that was used was jacketing. Three jacketing technics were used and each of them was an alternative strengthening technic for the pillars. These jacketing methods were: concrete column jacketing, steel jacketing of column and carbon fiber reinforced polymer jacket.

2.5.1. Concrete column jacketing

A reinforced concrete jacket was designed and added to the existing pillar of the structure. This permitted an increase of the section of the pillar, thereby increasing the strength and load carrying capacity of the pillars. An mn-interaction diagram was plotted for the new strengthened column section. Then further analysis of the behaviour of the column due to the applied load was made using a numerical software known as Abaqus.

2.5.1.1. Design of concrete jacket on pillar

The design specification of the different materials used for making up the jacket was noted. And the design computation was done in other to obtain the new section of the concrete jacketed column. The assumptions stated in table 1.5 of chapter one are taken into consideration during the design. For more safety, there was the neglect of the effect of the already existing reinforcement. The design axial load was computed using equation 2.31

$$N_u = 0.4 * f_{cd} * A_c + 0.67 * f_{yd} * A_s \tag{2.31}$$

Where:

N_u is the designed resistance of the original pillar section to axial load;

A_c and A_s are the area of the concrete and steel section respectively;

f_{cd} and f_{yd} are the designed yield strength of the concrete and steel respectively.

But A_s was considered as zero. There was a check if the original section of the pillar can withstand the incoming axial load. The amount of reinforcement that was needed to resist the incoming moment was gotten using an mn-interaction diagram for the rectangular section as seen in an annexe in figure A.1.

To know which interaction diagram to be used, we find the value of $\frac{C_c}{h}$, $\frac{N}{f_{cd}*b*h}$ and $\frac{M}{f_{cd}*b*h^2}$.

Where:

Cc is the concrete cove and h is the depth of the original column section.

N is the incoming axial load (KN),

M is the incoming moment (KNm),

b is the width of the column section.

These expressions were used in the MN interaction diagram in other to obtain the value of ω. Then the value of ω was replaced in equation 2.32 and the area of steel (As) was obtained using equation 2.33

$$\omega = \frac{A_s*f_{yd}}{b*h*f_{cd}} \tag{2.32}$$

Where:

ω is the ratio between the steel and the concrete strength.

$$A_s = \frac{\omega*b*h*f_{cd}}{f_{yd}} \tag{2.33}$$

A check was then made to see if the steel provided in the original section was enough. And the additional steel required in other to fully resist the loads was noted. Therefore, the section of the new concrete jacket was obtained considering the required steel and the remarks made in table 1.5 concerning concrete jackets.

2.5.1.2. Computation of interaction diagram for concrete jacket column section

A five-point mn-interaction diagram was plotted in excel. The points that constituted the curve were obtained as follows;

- Point 1 (Pure compression). In other to plot the first point, we will assume that the section undergoes pure compression. Hence, no moment (M = 0kN) but only axial forces. The stress and strain distribution on the concrete jacketed section is presented in figure 2.8. The computation of the axial forces that make up point one as shown in equation 2.34 was done using similar principles to those presented in equation 2.21 to equation 2.25.

$$N = C_{S1} + C_{S2} + C_{S3} + C_{S4} + C_{S5} + C_C \tag{2.34}$$

Where:

C_{S1}, C_{S2}, C_{S3}, C_{S4}, C_{S5} and C_C are the compressive strength of the first, second, third fourth and fifth layers of steel and that of concrete respectively.

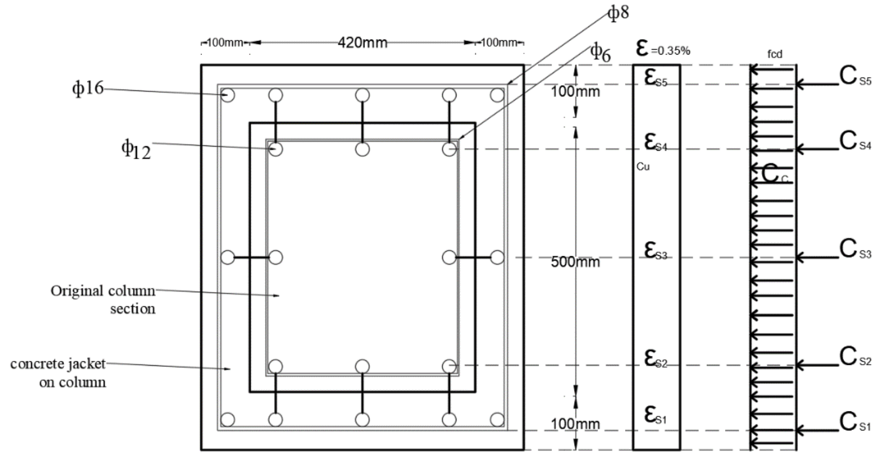


Figure 2.8. Strain and stress distribution of concrete jacket section, point 1 (AutoCAD, 2022)

- Point 2 (balance point). The balance point is the point when the concrete crushes and the tension steel yields. The same assumptions are made as in the balance point of the normal column cross-section presented in 2.4.4.2. The axial force (N) and the moment (M) that constitute point 2 were obtained from equation 2.35 and equation 2.36 respectively. Figure 2.9 shows the strain and stress distribution of the concrete jacket section for point 2.

$$N = -T_{S1} - T_{S2} + C_{S3} + C_{S4} + C_{S5} + C_C \tag{2.35}$$

$$M = Y_{S1} * T_{S1} + Y_{S2} * T_{S2} + Y_{S4} * C_{S4} + Y_{S5} * C_{S5} + Y_{Cc} * Cc \tag{2.36}$$

Where:

T_{S1}, T_{S2}, C_{S3}, C_{S4} and C_{S5} are the tension strength of the first and second layer of steel and the compressive strength third, a fourth and fifth layer of steel respectively. Cc is the compressive strength of the concrete. Y_{S1}, Y_{S2}, Y_{S4}, Y_{S5} and Y_{Cc} are the ever arms for the stresses T_{S1}, T_{S2}, C_{S4}, C_{S5} and C_C respectively (KN).

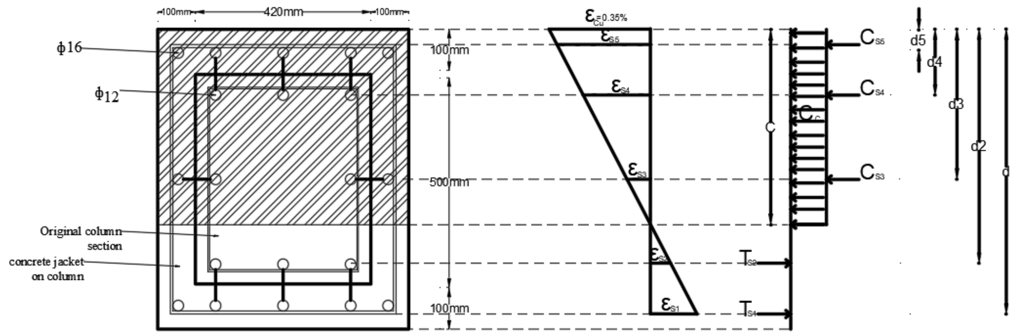


Figure 2.9. Strain and stress distribution of concrete jacket section, point 2 (AutoCAD, 2022)

- Point 3 (tension controlled). Here, the bottom steel strain has a value of 0.5 % ($\varepsilon s_1 = \varepsilon y = 0.5$ %). The strain on the concrete $\varepsilon c,u = 0.35$ %. The stress and strain distribution diagram considering this assumption made is shown in figure 2.10. The axial forces and moment that constitute this point were obtained using equations 2.37 and 2.38.

$$N = -T_{S1} - T_{S2} - T_{S3} + C_{S4} + C_{S5} + C_C \tag{2.37}$$

$$M = Y_{S1} * T_{S1} + Y_{S2} * T_{S2} + Y_{S4} * C_{S4} + Y_{S5} * C_{S5} + Y_{Cc} * C_c \tag{2.38}$$

- Point 4 (pure bending**).** At this point, we will consider that there is no axial force (N = 0kN).

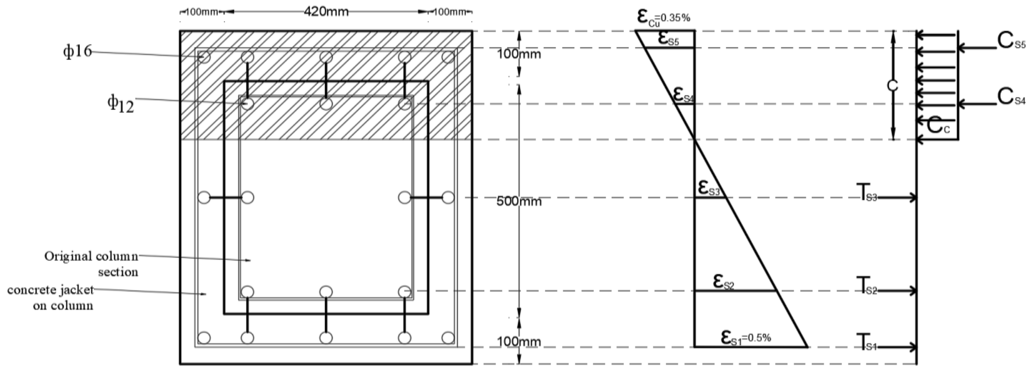


Figure 2.10. Strain and stress distribution of concrete jacket section, point 3

(AutoCAD, 2022)

The stress and strain distribution diagram for computing point 4 is shown in figure 2.11. The moment is obtained using an equation similar to that of equation 2.38.

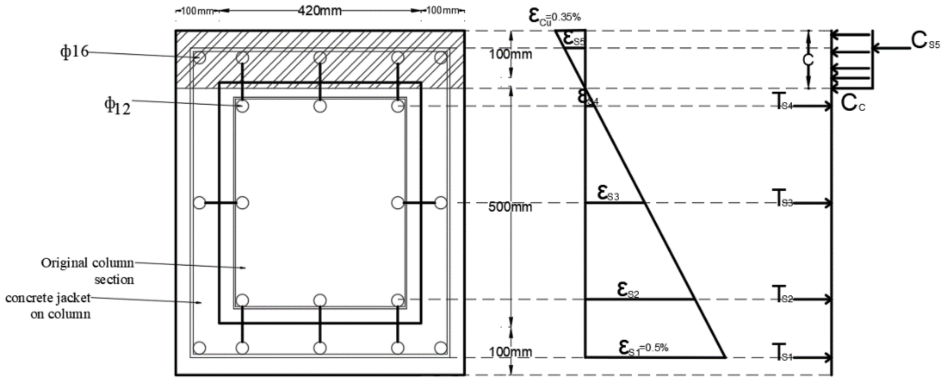


Figure 2.11. Strain and stress distribution of concrete jacket section, point 4 (AutoCAD, 2022)

- Point 5 (pure tension). Here, the concrete section is assumed not to contribute to resisting the loads and all the steel yield. So, the moment is considered null (M= 0kN). Figure 2.12 shows the Strain and stress distribution of the concrete jacket section for the calculation of point 5. The axial force that makes up point 5 was obtained from 2.39.

$$N = -T_{S1} - T_{S2} - T_{S3} - T_{S4} - T_{S5} \tag{2.39}$$

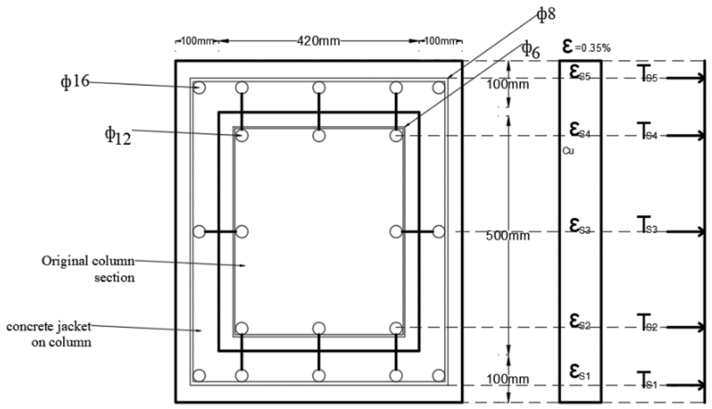


Figure 2.12. Strain and stress distribution of concrete jacket section, point 5

(AutoCAD, 2022)

2.5.1.3. Local analysis of concrete jacketing column on Abaqus

A local analysis of the column strength with a concrete jacket was made on Abaqus in other to have a more detailed description of the stress distribution and the displacement that occurred throughout the loading of the column.

The concrete jacket was modelled and added to the model containing the original column section. Interaction constraints such as tie constraints and embedded regions were used to define the interaction between the different elements of the concrete jacket column. A fix constrain was applied on one end of the concrete column and the other end was allowed free for the application of the loads. Due to the large number of loads going to be carried by the column and in other to take into account large displacements, dynamic explicit analysis was performed.

2.5.2. Steel jacketing on column

An alternative strengthening technic for the column was studied and this was done by reinforcing the column with steel plates. A choice of the steel section required for the strengthening was done and an mn-interaction diagram for the composite steel-concrete section was plotted as a verification of the choice made concerning the incoming loads. Finally, there was a local numerical analysis of the new column model that was done in Abaqus.

2.5.2.1. Choice of steel section for jacketing

Due to the lack of adequate norms on the design of steel jackets, a steel jacket section was chosen from a table presenting the characteristics of specimens analyzed in a parametric study of a column strengthen by different steel angle sections (Al-Sherrawi & Salman, 2017).

2.5.2.2. Computation of interaction diagram of column with steel jacket

The plot of the interaction diagram was done in other to verify if the composite concrete steel jacket and concrete section will resist applied axial force and moment on the pillar. The column section showing the steel jacket and the interaction diagram will be similar to that present in figure 2.13 and figure 2.14. respectively.

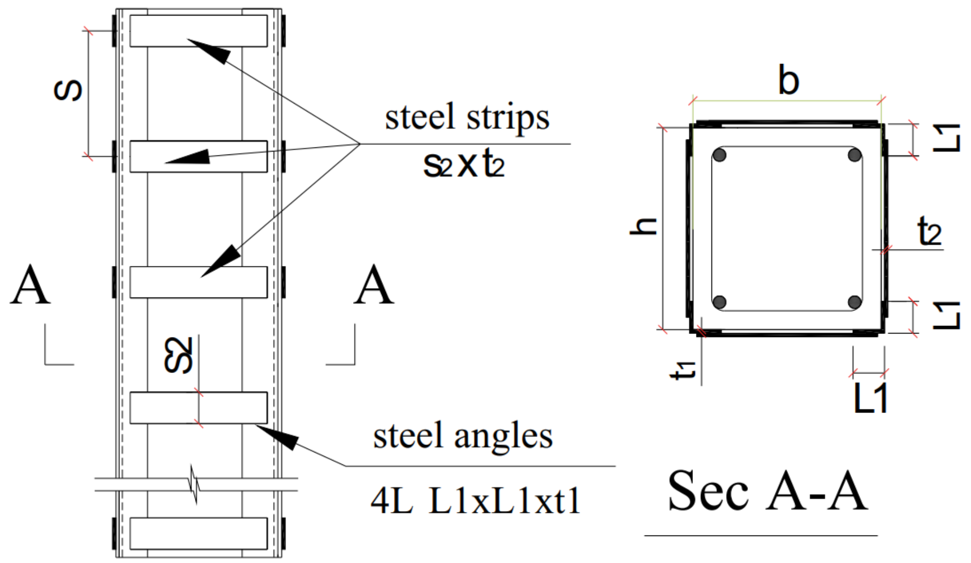


Figure 2.13. The dimensions of the strengthened column with steel angles and strips (Shallan et al., 2022)

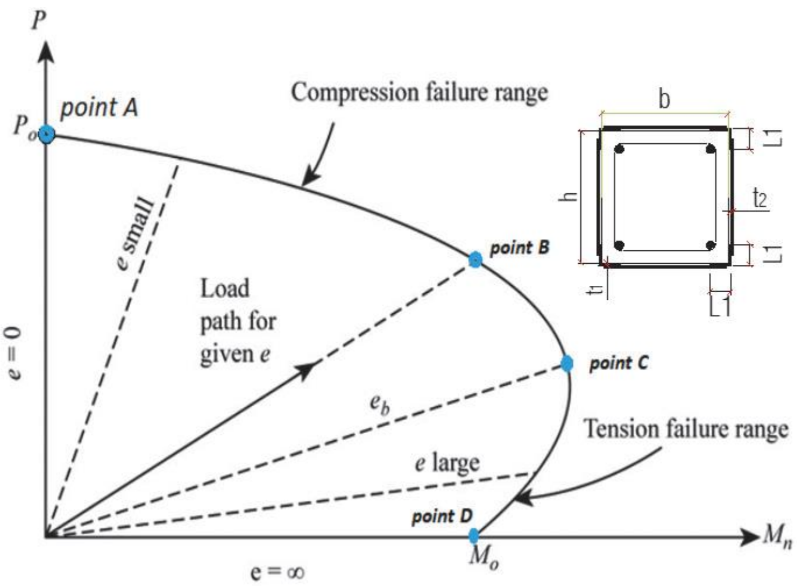


Figure 2.14. The main points used to plot the interaction diagrams for the strengthened column using steel angles and strips (Shallan et al., 2022)

Point A is defined with the design value of the resistance of the composite section to compressive axial force N_A while the bending moment M_A is zero (eccentricity e = 0). The axial force was obtained using equation 2.40. The stress distribution for the computation of point A is presented in figure 2.15.

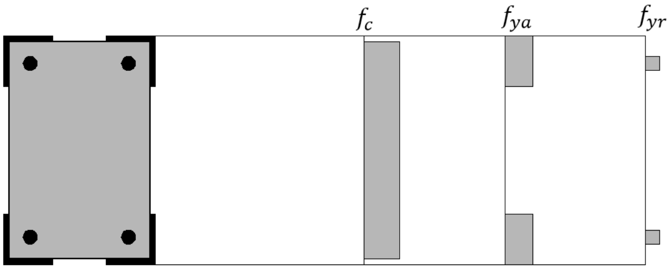


Figure 2.15. Stress distribution for point A (Al-Sherrawi & Salman, 2017)

$$N_A = f_c \cdot b \cdot h + f_{ya} \cdot A_s + f_{yr} \cdot A_{sr} \quad (2.40)$$

$$M_A = 0$$

Where:

b is the width of the column, h is the height of the column, As is the total area of steel angles, and Asr is the total area for reinforcing bars.

Point C is defined with the maximum design value of the resistance moment M_C in the presence of a compressive normal force N_C. The values of and were obtained from equations 2.41 and 2.42 respectively. The stress distribution for point C is presented in figure 2.16.

$$N_C = f_c \cdot \frac{h}{2} \cdot b \tag{2.41}$$

$$M_C = M_{max} = f_c \cdot \left[\frac{bh^2}{8}\right] + Z_a f_{ya} + f_{yr} \cdot A_{sr}\left[d - \frac{h}{2}\right] \tag{2.42}$$

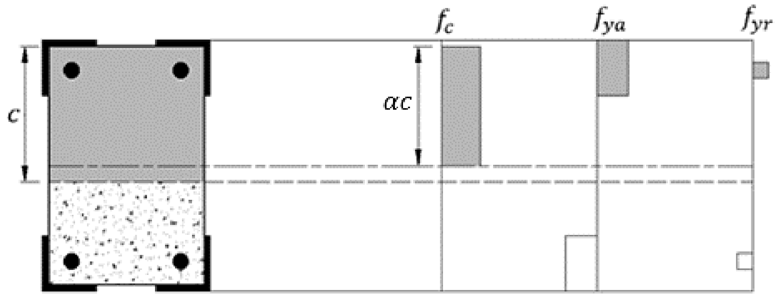


Figure 2.16. Stress distribution for point C (Al-Sherrawi & Salman, 2017)

Where:

αc is the height of the equivalent stress block, d is the effective depth for reinforcing bars, and Za is the plastic section modulus for steel angles. The value of Za was gotten from equation 2.43.

$$Z_a = 4\left[\left((L1 - t)t\left(\frac{h+t}{2}\right)\right) + \left(L1t\left(\frac{h}{2} + t - \frac{L1}{2}\right)\right)\right] \tag{2.43}$$

Where:

L_1 is the length of one side of the steel angle (mm) and t is the thickness of the steel angle (mm).

Point D is defined with the design value of the bending moment resistance of the composite section M_B while the axial force N_D is zero. The position of the neutral C axis must be assumed and to be checked later. By applying $\sum F = N_D$ and using $N_D = 0$, C can be found. Four possible assumptions may be considered to find c and as follows;

Case 1, $(L_1 - t) \leq c < \frac{h}{2}$ as shown in figure 2.17.

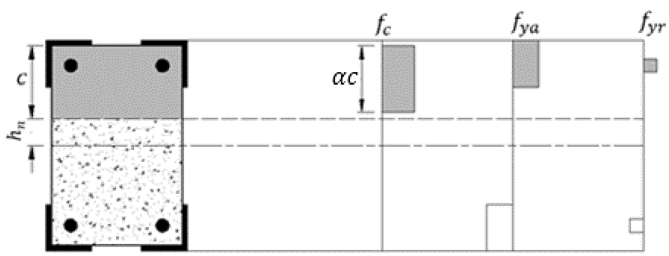


Figure 2.17. Stress distribution for point D case 1 (Al-Sherrawi & Salman, 2017).

$$N_D = 0$$

$$f_c \cdot b \cdot \alpha \cdot c + \frac{A_s}{2} \cdot f_{ya} + \frac{A_{sr}}{2} \cdot f_{yr} - \frac{A_s}{2} f_{ya} - \frac{A_{sr}}{2} f_{yr} = 0$$

$$f_c \cdot b \cdot \alpha \cdot c = 0$$

$$f_c \cdot b \cdot \alpha \neq 0$$

$$c = 0\ (error)$$

Case 2, $0 < c < (L_1 - t)$ and $c \geq (d' + \frac{dbar}{2})$ as shown in figure 2.18. The position of the neutral axis c and the consequent moment is given by equations 2.44 and 2.45 respectively.

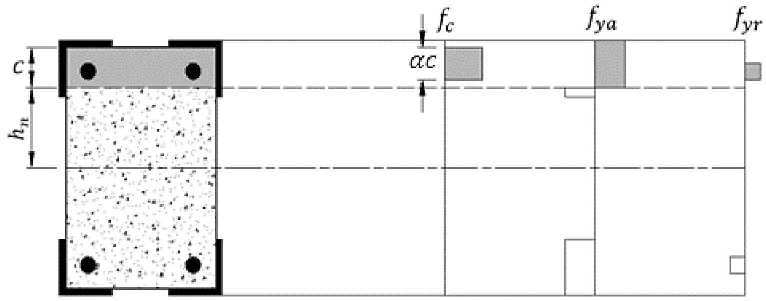


Figure 2.18. Stress distribution for point D case 2 (Al-Sherrawi & Salman, 2017)

$$N_D = 0$$

$$f_c \cdot b \cdot \alpha \cdot c + \frac{A_s}{2} f_{ya} + \frac{A_{sr}}{2} \cdot f_{yr} - \frac{A_s}{2} \cdot f_{ya} - \frac{A_{sr}}{2} f_{yr} - 4\left(t \cdot f_{ya}(L1 - t - c)\right) = 0$$

$$c = \frac{4 \cdot t \cdot f_{ya}(L_1 - t)}{\alpha \cdot f_c \cdot b + 4 \cdot t \cdot f_{ya}} \tag{2.44}$$

$$h_n = \frac{h}{2} - c$$

$$M_D = f_c b \alpha c\left(\frac{h}{2} - \frac{\alpha c}{2}\right) + Z_a f_{ya} + A_{sr} f_{yr}\left(\frac{h}{2} - d'\right) - \left[4t\,(L1 - t - c) f_{ya}\left(\frac{h}{2} - c - \left(\frac{L1 - t - c}{2}\right)\right)\right] \tag{2.45}$$

Case 3: $0 < c < (L_1 - t)$ and $(d' - \frac{dbar}{2}) < c < (d' + \frac{dbar}{2})$ as shown in figure 2.19. The position of the neutral axis c and the consequent moment is given by equations 2.46 and 2.47 respectively.

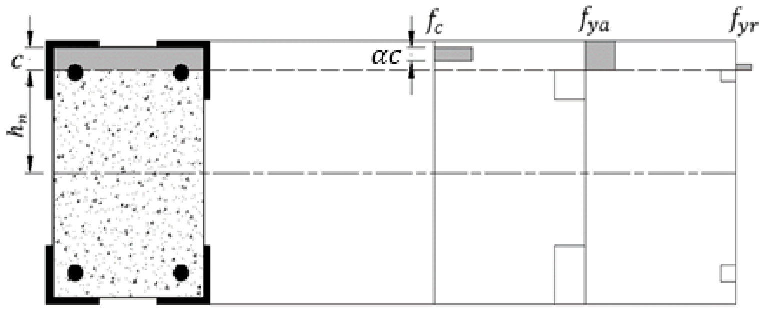


Figure 2.19. Stress distribution for point D case 3 (Al-Sherrawi & Salman, 2017)

$$A_i = d_{bar} \cdot h \cdot i$$

$$h_i = c - concrete\ cover - d_{tie}$$

Where:

A_i is the area of the segment of reinforcing bars (mm^2) under compressive stress and h_i is the height of this segment (mm).

The centre of gravity of this segment can be assumed as h_i.

$$N_D = 0$$

$$f_c \cdot b \cdot \alpha_c + \frac{A_s}{2} f_{ya} + \frac{n}{2} A_i f_{yr} - \left(\frac{Asr}{2} - \frac{n}{2} Ai\right) f_{yr} - \frac{A_{sr}}{2} f_{yr} - \frac{A_s}{2} f_{ya} - 4\left(t\,(L1 - t - c) f_{ya}\right) = 0$$

Where:

n is the number of reinforcing bars.

$$c = \frac{4 \cdot t \cdot (L_1 - t) \cdot f_{ya} + (A_{sr} - n \cdot A_i) \cdot f_{yr}}{f_c \cdot \alpha \cdot b + 4 \cdot t \cdot f_{ya}} \qquad (2.46)$$

$$h_n = \frac{h}{2} - c$$

$$M_D = f_c \cdot b \cdot \alpha \cdot c \cdot \left(\frac{h}{2} - \frac{\alpha c}{2}\right) + Z_a f_{ya} - \left[4t\,(L1 - t - c) fya \left(\frac{h}{2} - c - \left(\frac{L1 - t - c}{2}\right)\right)\right] + \frac{n}{2} \cdot A_i f_{yr} \left(\frac{h}{2} - c + \frac{h_i}{2}\right) - \left[\left(\frac{A_{sr}}{2} - \frac{n}{2} \cdot A_i\right) f_{yr} \left(\frac{h}{2} - c - \left(\frac{d_{bar} - h_i}{2}\right)\right)\right] + \left(A_{sr} f_{yr} \left(\frac{h}{2} - d'\right)\right)$$

(2.47)

Case 4: $0 < c < (L_1 - t)$ and $c \leq (d' - \frac{d_{bar}}{2})$ as shown in figure 2.20. The position of the neutral axis c and the consequent moment is given by equations 2.48 and 2.49 respectively.

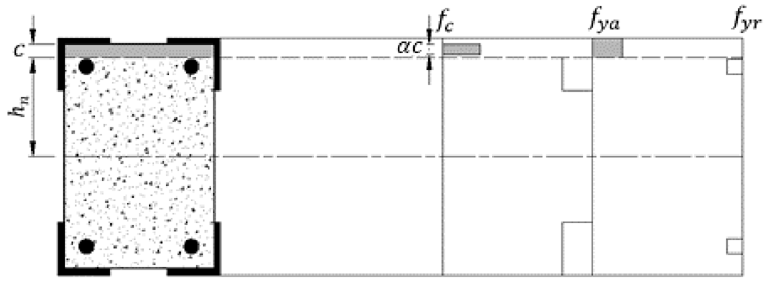


Figure 2.20. Stress distribution for point D case 4 (Al-Sherrawi & Salman, 2017)

$$N_D = 0$$

$$f_c \cdot b \cdot \alpha \cdot c + \frac{A_s}{2} f_{yab} - A_{sr} \cdot f_{yr} - \frac{A_s}{2} f_{ya} - 4\left(t(L1 - t - c)f_{ya}\right) = 0$$

$$c = \frac{4 \cdot t \cdot (L_1 - t) \cdot f_{ya} + A_{sr} \cdot f_{yr}}{f_c \cdot \alpha \cdot b + 4 \cdot t \cdot f_{ya}} \quad (2.48)$$

$$M_D = f_c \cdot b \cdot \alpha \cdot c \cdot \left(\frac{h}{2} - \frac{\alpha c}{2}\right) + Z_a f_{ya} - \left[4t(L1 - t - c)f_{ya}\left(\frac{h}{2} - c - \left(\frac{L_1 - t - c}{2}\right)\right)\right] \quad (2.49)$$

Point B corresponds to a neutral axis location that results in the same flexural capacity as Point D as shown in equation 2.50 and twice the axial load of Point C as seen in equation 2.51.

$$N_B = 2N_C \quad (2.50)$$

$$M_B = M_D \quad (2.51)$$

2.5.2.3. Local analysis of steel jacketed column on Abaqus

The concrete column strengthened with a steel jacket was modelled on Abaqus. A dynamic explicit analysis was run in other to determine the stress distribution and the displacement on the column jacketed section.

The steps for building the model were done according to what was described in section 2.4.4.3. The different elements that constitute the steel jacket such as the strip and steel angle were modelled and assembled. A tie constrain was used as an interaction parameter between the strip and the steel angle. The steel jacket was then joined to the original column for strengthening using a tie constrain. A fixed constrain was applied on the base end of the concrete column and the top end was allowed free for the application of the loads. The analyses were performed and the results were viewed.

2.5.3. Carbon fiber reinforced polymer jacketing of column

CFRP is another jacketing technic that was applied to the column for strengthing. A selection of an appropriate section of the carbon fiber that was required for reinforcing the column section column was made. The verification of the adequacy of the CFRP selected in resisting the incoming load was verified by plotting an interaction diagram for the new carbon fiber concrete section. Then, the stress distribution obtained by the new column section was viewed by making a model of the column on Abaqus.

2.5.3.1. Design of carbon fiber reinforced polymer jacket of column

There is no appropriate design specification for CFRP. So, a section CFRP of a particular grade was chosen from a carbon fiber category reference table. The mechanical properties of the material were noted and the characteristics of the concrete section too were noted.

2.5.3.2. Computation of interaction diagram for CFRP jacket

There was a plot of an mn-interaction diagram for the composite carbon fiber and concrete composite section. Verifications were then made in other to validate the resistance of the CFRP to the excess axial and moment forces. Analysis of RC columns confined with transverse FRP sheets is similar to typical reinforced concrete columns (without CFRP sheets); the only difference is the stress-strain model of concrete in the compressive region (Manie et al., 2017). Longitudinal fibers have a larger impact on improving the loading capacity of tension-controlled columns whereas in the compression-controlled region of behaviour it is better to use transverse CFRP composites under uniaxial bending. Hence the design philosophy for the columns was strengthening with combined longitudinal and transverse CFRP materials in other to properly enhance the strength of the column in cases. The interaction diagram will have the shape of that presented in figure 2.21

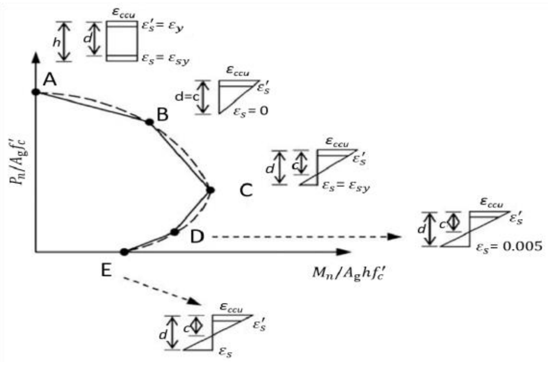


Figure 2.21. Simplified interaction diagram for column strengthened with transverse CFRP (Manie et al., 2017)

- Point A: uniform axial compressive strain in the confined concrete. At point A, the moment is zero. But the axial force that makes up point A is given by equation 2.52

$$P_n(A) = \left[\,0.85f'_{cc}(Ag - Ast) + f_y A_s t\right] \tag{2.52}$$

- Point B: This point corresponds to the state of strain distribution in which maximum compressive strength is ε_{ccu} and strain in the last rebar sheet in the tensile region is equal to zero. ε_{ccu} is the ultimate compressive strength of the confined concrete. In non-confined conditions, ε_{cu} is the ultimate compressive strength of unconfined concrete is used
- Point C: This point represents the state of strain distribution in which simultaneously the maximum ultimate compressive strain of concrete is εccu and the maximum tensile strain in the last rebar sheet is equal to yield stress. This point is the same as the equilibrium (balance) point of the column behaviour.
- Point D: This point corresponds to the state of strain distribution in which maximum compressive strain equals εccu (or εcu in unconfined conditions) and strain in the last longitudinal rebar sheet of the section is equal to 0.005.
- Point E: This point is associated with the conditions for pure bending without axial force.

The nominal axial force and bending moments for the other points B, C, D, and E were calculated using the integration of stress as seen in equations 2.53 and 2.54.

$$P_n(B,C,D,E) = \int_0^{yt}\left(E_c\left(\frac{\varepsilon_{cc}}{y}\right) - \frac{(E_c-E_2)^2}{4f_c'}\left(\frac{\varepsilon cc}{c}y\right)^2\right)\cdot b\cdot d\cdot y \; + \int_{yt}^{c}\left[f'c + E2\left(\frac{\varepsilon ccu}{c}y\right)\right]bdy \; + \Sigma f_{si}A_{si} + \Sigma_f f_i A f_i \tag{2.53}$$

$$M_n(B,C,D,E) = \int_0^{yt}[E_c\left(\frac{\varepsilon_{cc}}{y}\right) - \frac{(E_c-E_2)^2}{4f_c'}\left(\frac{\varepsilon cc}{c}y\right)^2]\cdot(\frac{h}{2} - c - d)bdy \; + \int_{yt}^{c}\left[f'c + E_2\left(\frac{\varepsilon_{ccu}}{c}y\right)\right](\frac{h}{2} - c \; + y)\,bdy \; + \Sigma siAsidsi \; + \Sigma ffiAfidfi \tag{2.54}$$

Where:

c is the distance of the neutral axis from the farthest compressive axis of the section.

A_{si}, f_{si} and d_{si} are respectively area, stress, and distance to the centroid of the areas in i-th sheet of longitudinal rebars. “y” is the integration variable in the compressive region of the section.

The coordinate parameter y_t lies on the cross-section has the neutral axis and can be determined using equation 2.55 According to the stain distribution diagram in figure 2.22

$$y_t = c \times \left(\frac{\varepsilon'_t}{\varepsilon_{ccu}}\right) \tag{2.55}$$

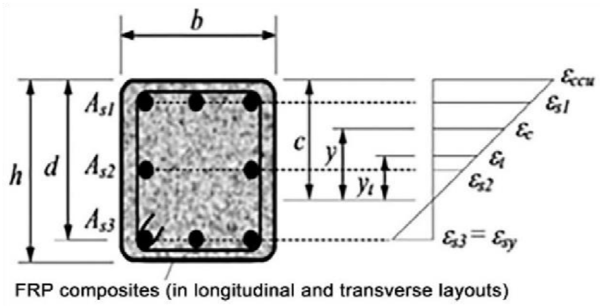


Figure 2.22. Strain distribution diagram for composite CFRP and concrete section (Manie et al., 2017)

The compressive strength fc, the transient strain ε'_t, the slope of the linear section of confined stress-strain curve E_2 as shown in figure 2.23, the maximum compressive strength of the confined concrete f'_{cc}, the CFPR compressive strength f_l and the effective strain ε_{fe} are computed form equation 2.56, 2.57, 2.58, 2.59, 2.60 and 2.61, respectively.

$$f_c = \begin{cases} E_c\varepsilon_c - \frac{(E_c - E_2)^2}{4f'_c}\varepsilon_c^2 & 0 \le \varepsilon c \le \varepsilon' t \\ f'c + Ec\ \varepsilon c & \varepsilon' t \le\ \varepsilon c \le\ \varepsilon ccu \end{cases}$$

(2.56)

$$\varepsilon'_t = 2f'_c(E_c - E_2) \tag{2.57}$$

$$E_2 = \frac{(f'cc - f'c)}{\varepsilon_{ccu}} \tag{2.58}$$

$$f'_{cc} = f'_c + 3.3 \cdot K_a f_1 \tag{2.59}$$

$$f_1 = \frac{2 \cdot n \cdot t \cdot f \cdot E \cdot f\varepsilon_{fe}}{\sqrt{h^2 + b^2}} \tag{2.60}$$

$$\varepsilon_{fe} = 0.586 \cdot \varepsilon_{fu} \tag{2.61}$$

Where:

ε_{fu} is the maximum ultimate design strain of CFRP, f'c maximum compressive strength of unconfined concrete;

K_a and K_b are the geometric effect coefficients and ε_{ccu} is the maximum compressive strain of CFRP confined concrete.

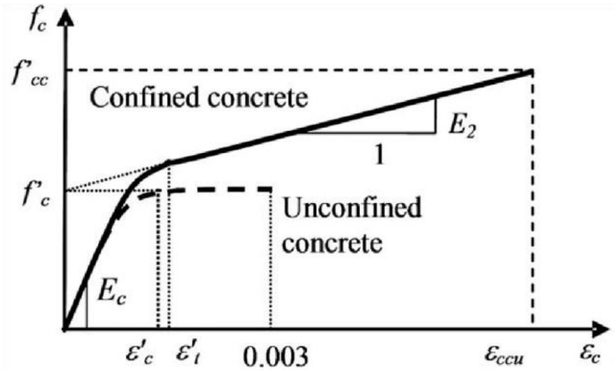


Figure 2.23. Stress-strain curve of unconfined and CFRP-confined concrete based on Lam and Teng model (Manie et al., 2017)

The maximum compressive strain of CFRP was evaluated using equation 2.62.

$$\varepsilon_{ccu} = \varepsilon'_c \cdot \left(1.5 + 1.2 \cdot K \cdot b \frac{f1}{f'c} \left(\frac{\varepsilon fe}{\varepsilon' c}\right) 0.45\right) \tag{2.62}$$

The geometric effect coefficient Ka and Kb was evaluated using equations 2.63 and 2.64 respectively;

$$K_a = \frac{A_e}{A_c} \cdot \left(\frac{h}{b}\right)^2 \tag{2.63}$$

$$K_b = \frac{A_e}{A_c} \cdot \left(\frac{h}{b}\right)^{0.5} \tag{2.64}$$

Where:

A_c is the cross-sectional area of the concrete and Ae is the effective confined area. h and b are the lengths of the sides of the section.

Figure 2.24 shows the effectively confined core for the non-circular section.

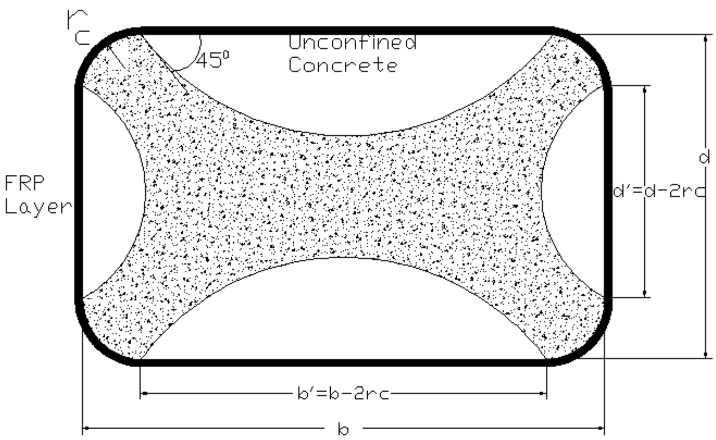


Figure 2.24. Effectively confined core for non-circular section (Marlapalle et al., 2014)

$$A_e = \frac{b'^2 + d'^2}{3} \tag{2.65}$$

$$b' = b - 2 \cdot r_c \tag{2.66}$$

$$d' = d - 2 \cdot r_c \tag{2.67}$$

Where:

r_c is the radius of curvature (25 mm).

Following integration and rearrangement of equations 2.35 and 2.36, using coefficients A, B, C, D, E, F, G, H, and I, equations 2.68 and 2.69 are derived as below. The aforementioned coefficients are separately computed in equations 2.68 − 2.78.

$$P_n(B,C,D,E) = [A(yt)^3 + B(yt)^2 + Cyt + D] + \sum f_{si}A_{si} \tag{2.68}$$

$$M_n(B,C,D,E) = [E(yt)^4 + F(yt)^3 + G(yt)^2 + Hyt + I] + \sum f_{si}A_{si}d_{si} \tag{2.69}$$

$$A = \frac{-b(E_c - E_2)^2}{12f_c'}\left(\frac{\varepsilon_{ccu}}{c}\right)^2 \tag{2.70}$$

$$B = \frac{b(E_c - E_2)}{2}\left(\frac{\varepsilon_{ccu}}{C}\right) \tag{2.71}$$

$$C = -bf_c' \tag{2.72}$$

$$D = bf_c' + \frac{b \cdot c \cdot E_2}{2}\varepsilon_{ccu} \tag{2.73}$$

$$E = \frac{-b(E_c - E_2)^2}{16f_c'}\left(\frac{\varepsilon ccu}{c}\right)^2 \tag{2.74}$$

$$F = b \cdot \left(c - \frac{h}{2}\right) \cdot \frac{-b(Ec - E2)^2}{12fc'} \cdot \left(\frac{\varepsilon ccu}{c}\right)^2 + \frac{b(Ec - E2)}{3}\left(\frac{\varepsilon ccu}{C}\right) \tag{2.75}$$

$$G = -\left(\frac{b}{2}f_c' - b\left(c - \frac{h}{2}\right)\frac{(E_c - E_2)}{2}\left(\frac{\varepsilon ccu}{C}\right)\right) \tag{2.76}$$

$$H = b\,f_c' \cdot \left(c - \frac{h}{2}\right) \tag{2.77}$$

$$I = \frac{bc^2 f_c'}{2} - b\,c\,fc'\left(c - \frac{h}{2}\right) + \frac{b\,c^2 E2}{3}\varepsilon ccu - \frac{b\,c\,E2}{2}\left(c - \frac{h}{2}\right)\varepsilon_{ccu} \tag{2.78}$$

The position of the neutral axis c for the points that constitute the interaction curve was obtained using the formulas in equations 2.79 – 2.81.

For point B,

$$c = d \text{ (d is the effective depth of the section)} \tag{2.79}$$

For point C,

$$c = \frac{\varepsilon_{ccu}}{\varepsilon_{ccu} + \varepsilon_s} \cdot d \qquad 2.80$$

For point D,

$$c = \frac{\varepsilon_{ccu}}{\varepsilon_{ccu} + 0.005} \cdot d \tag{2.81}$$

For point E, we assume the axial force is zero and we compute the position of the neutral axis for this assumption.

2.5.3.3. Local analysis of CFRP jacketing column on Abaqus

The concrete column strength with the CFRP jacket was modelled on Abaqus. A dynamic explicit analysis was run in other to determine the stress distribution and the displacement on the CFRP column jacketed section.

Following the same steps that were described in section 2.4.4.3, the column strengthened by the CFRP jacket was modelled. The fiber section was first modelled, its material property defined and then it was assembled into the original concrete column section. Embedded region constrain was applied for the concrete section and the reinforcing steel bars whereas tie constraints were used between the CFRP and the concrete section. Fixed support constrain was applied on the base end of the concrete column and the top end was allowed free for the application of the loads. The analyses were performed and the results were viewed.

2.6. Comparative study of the different design interventions

A comparative study was made between the different forms of design strengthening interventions on the column. The comparison was done between concrete jacketing, steel jacketing and carbon fiber reinforced polymer jacket. The first comparison had to do with the behaviour of the strengthen column and the second comparison was a cost comparison of the different jacketing methods for the column.

2.6.1. The behaviour of the strengthened column

What is meant by the behaviour of the strengthen column is the way the strengthen column will respond to the incoming loads. Hence, a comparison was made between the different jacketing methods after performing a local analysis on Abaqus. The criteria that constituted this comparison were the mise stresses, the average displacements and the magnitude of the duckling deformation on the column.

2.6.2. Cost comparison

A cost comparison was done between the different jacketing technics. An assessment of the number of columns needing strengthening was made. In other to carry out the cost comparison, first of all, quantifying the amount of material needed by each strengthening method. Then the price of each volume of material was noted. A sum of the material price for the different jacketing methods was done and the results of the cost were compared.

For the concrete jacket section, knowing the dimensions of the jacket, the components that make up the jacket (concrete, main bars reinforcement, shear connectors and stirrup) were quantified and the corresponding price of each component in the city of Yaounde was obtained. On the other hand for the steel and CFRP, the quantity needed for the strengthening was obtained knowing the surface area of the column and the cross-section design of the jacket. The materials price for the steel and CFRP were obtained from production firms in china via online purchase. The shipping fees were also included in the calculation

Conclusion

This chapter aimed at making a chronological presentation of the necessary steps in other to carry out an analysis and design intervention on a RC building. A general site recognition was done through adequate research on the internet. An actual site visit was made in other to come in contact with the actual reality of the state of the build. Questionnaires and interviews were made in other to collect data before the intervention on the building. A structural model of the building was done using the numerical software Sap2000, thereby enabling the actual loads that each pillar will carry to be known. The steps required for the computation of an mn-interaction diagram in other to verify if the existing column can resist incoming load were clearly defined. The stages in the design and verifications of the strengthening of the column with concrete, steel and CFRP jacketing were pointed out in this chapter. The steps required for the verification of the different methods of jacketing through computation of the jacket interaction curve point were explained. This chapter also explained the stages involved in the local analysis of the column using the finite element method software Abaqus. Finally, the way to carry out a comparative study between this method was discussed.

CHAPTER 3 : PRESENTATION OF RESULTS AND INTERPRETATION

Introduction

This third chapter presents the results of the procedure described in chapter two. These results are the actual design of the interventions on the actual building. Before the presentation of the design intervention, the location of the case study is presented and this is done through the general presentation of the site. The site visit permitted a physical description of the site. In this chapter, the data collected are presented. The result of the structural analysis is shown and interpretative comments are made. The result of the design intervention on the column that was described in chapter two is presented and finally, the result of the comparative study is presented and comments are made.

3.1. General presentation of the site

The construction site is found at Mimboman, a residential area in the town of Yaounde, the national capital of Cameroon. This presentation is done considering two characteristics: the physical and the socio-economical characteristics of the site.

3.1.1. Physical characteristics

These characteristics of Mimboman are the geographical location and topography and climate.

3.1.1.1. Geographical location and topography

The city of Yaoundé (see figure 3.1) is located in the south of the Center Region and is 250 km east of the coast of the Gulf of Biafra. This mountain site is broken down into three topographic units inscribed in a rocky base of Precambrian gneiss: the barrier of inselbergs in the northwest dominated by the Mbam Minkom mountains (1,295 m) and Mount Nkolodom (1,221 m) and to the southwest with Mount Eloundem (1,159 m); a set of hills 600 to 700 m high and plateaus; the valleys also called élobis.

A location plan from the National Advanced School of Public Works to the construction site found at Mimboman is seen in figure3.2. The topography of the construction site shows that the building is on a flat land surface.

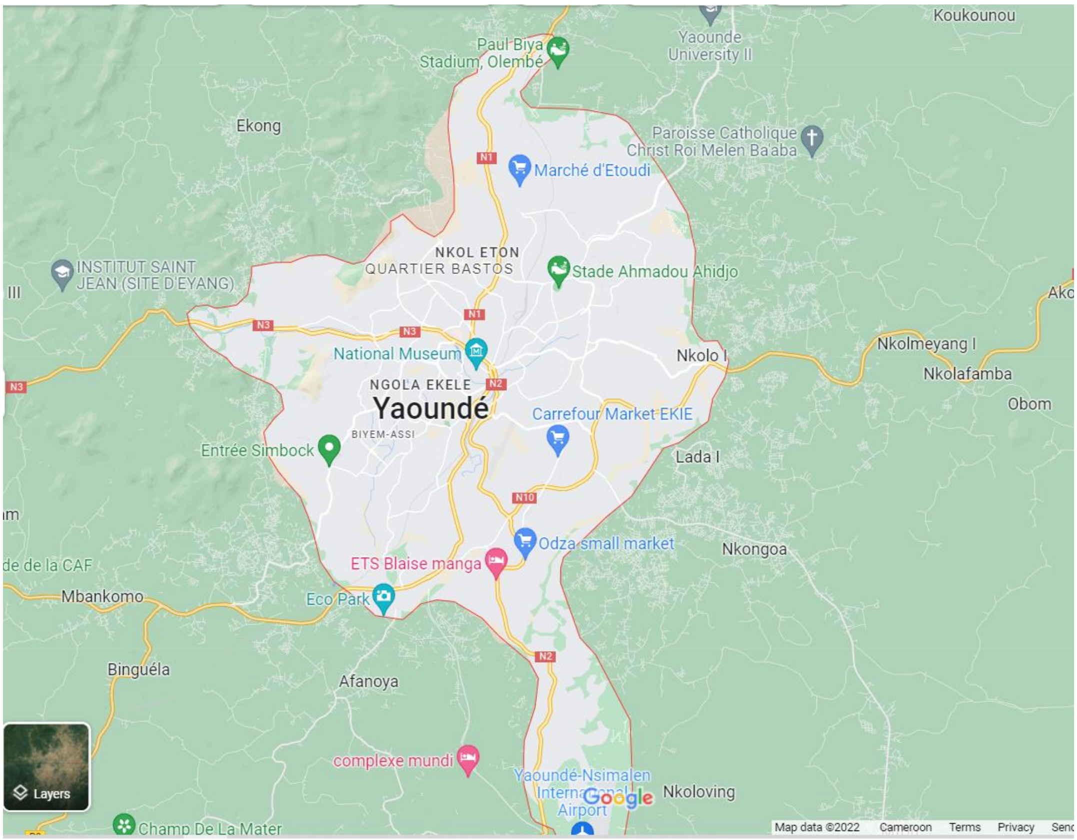


Figure 3.1. The map of Cameroon showing the capital city Yaounde (Google maps, 2022)

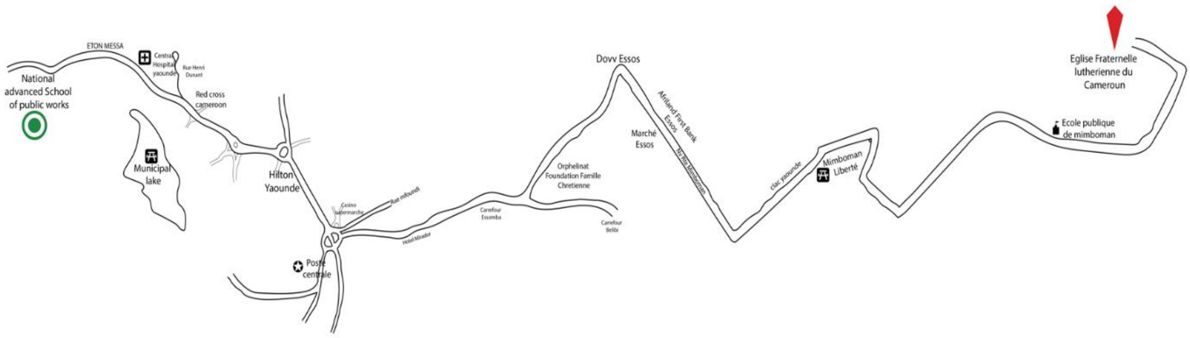


Figure 3.2. location map of the case study in Yaounde (Adobe illustrator, 2022)

3.1.1.2. Climate

The climate of Yaoundé, the capital of Cameroon, is tropical, though tempered by altitude, with a dry season from December to February and a rainy season from March to November. The rains decrease a bit in July and August, although the sky remains often cloudy. The city is located southwest of Cameroon, at 700 meters (2,300 feet) above sea level. In Yaoundé, the average temperature of the coldest month (August) is 23.2 °C (73.8 °F), and that of the warmest month (February) is 25.8 °C (78.5 °F). Here are the average temperatures in table 3.1.

Table 3.1. The average temperature of Yaounde throughout the year from 1991-2020 (Climate to travel, 2020)

Yaoundé - Average temperatures (1991-2020)						
Month	**Min (°C)**	**Max (°C)**	**Mean (°C)**	**Min (°F)**	**Max (°F)**	**Mean (°F)**
January	20	30	24.8	67	86	76.6
February	21	31	25.8	70	88	78.5
March	21	30	25.7	70	87	78.2
April	21	30	25.2	69	86	77.4
May	20	29	24.9	69	85	76.8
June	20	27	23.9	69	81	75
July	20	26	23.3	68	79	73.9
August	20	26	23.2	68	79	73.8
September	20	27	23.7	68	81	74.6
October	20	28	24.1	68	83	75.4
November	20	29	24.5	68	84	76
December	20	29	24.4	67	85	75.9
Year	20.2	28.7	24.4	68.4	83.6	76

In Yaoundé, precipitation amounts to 1540 millimetres (60.6 inches) per year, it is therefore abundant. It ranges from 20 mm (0.8 in) in the driest months (January, December) to 295 mm (11.6 in) in the wettest one (October). Here is the average precipitation in table 3.2.

Table 3.2. Average precipitation in Yaounde (Climate to travel, 2020)

Yaounde - Average precipitation			
Month	**Millimetres**	**Inches**	**Days**
January	20	0.8	3
February	45	1.8	4
March	125	4.9	12
April	170	6.7	14

May	200	7.9	17
June	155	6.1	14
July	75	3	11
August	115	4.5	12
September	230	9.1	20
October	295	11.6	23
November	95	3.7	11
December	20	0.8	3
Year	1540	60.6	144

3.1.2. Socio-economical characteristics

The socio-economical characteristics of the town of Yaounde include the population, the economy and transport.

3.1.2.1. Population

The population of the people living in Yaounde is estimated to be about 4315670 people according to world statistics data. Yaounde is considered the most populated city in Cameroon. It ranks 13th in Africa and 104th in the world in terms of population.

3.1.2.2. Economy

Most of Yaoundé's economy is centred on the administrative structure of the civil service and the diplomatic services. Owing to these high-profile central structures, Yaounde has a higher standard of living and security than the rest of Cameroon. Major industries in Yaoundé include tobacco, dairy products, beer, clay, glass goods and timber. It is also a regional distribution centre for coffee, cocoa, copra, sugar cane and rubber. Residents engage in urban agriculture. The city is estimated to have "50,000 pigs and over a million chickens.

3.1.2.3. Transport

Yaoundé Nsimalen International Airport is a major civilian hub, while nearby Yaoundé Airport is used by the military. Train lines run west to the port city of Douala and north to N'Gaoundéré. Many bus companies operate from the city; particularly in the Nsam and Mvan districts.

3.2. Physical description of the site

In the physical description of the building, there will be a presentation of what was observed on site and the responses obtained after the questionnaire and the interview was made.

3.2.1. Actual observations

The building is a church auditorium with mezzanine for L'Eglise Fratenelle Lutherienne du Cameroun (EFLC) paroisse de mimboman. The building has a surface area of 692.25 m^2. Some cracks highlighted in red in figure 3.3 were observed on the wall hence showing the impact of the collapse on the structure. The actual state of the building (roofless) during the site visit is shown in figure 3.3.

Figure 3.3. The state of the building during the site visit showing cracks (onsite picture, 2022)

3.2.2. Response to questionnaires and interviews

The questionnaires and interviews permitted the obtaining of a description of the initial state of the building and the collapse and how the analysis was made on the building by the enterprise respectively.

3.2.2.1. Building initial state and collapse

The building was initially constructed about 15 years ago. The initial construction was done locally without any proper architectural or structural intervention. Hence, the building did not have an architectural plan or a structural plan. The construction was done based on experience and estimations. There wasn't a proper study of the soil bearing capacity. Initially, when the building was constructed, the roof truss was made of wood. Due to the long span length of the truss elements of about 18 m and the absence of intermediate pillars in the hall, there was a danger of the roof collapsing into the hall due to its weight. Hence the technicians decided to add a mezzanine in the building since it will help to create a connection between the structural element, making them resist more to the weight of the roof and also increasing the seating capacity of the auditorium. Unfortunately, this recommendation wasn't taken into consideration. The hall was constructed without a mezzanine. Since they were wood truss elements that were used for the building, the roofing system was relatively light and hence the structure was able to tolerate that weight. The building was used for some years then, there was a need to cover the roof with a beautiful wood panelled ceiling as shown in figure 3.4. But before that, the structure had already shown some signs of fatigue through cracks observed on the walls. Hence this led to some remedial work on the building. But they were not enough to prevent collapse due to the new panelled ceiling. The ceiling became an additional weight on the building. Hence the roof of the structure that was already in a limited state of stability after about two months of usage collapsed. It happened on a Monday at 4 am. The entire roof fell under the effect of its weight.

Figure 3.4. wooden panelled ceiling, before the collapse (building picture, 2021)

3.2.2.2. Analysis made on the structure

The collapse of the roof brought the need for a global study on the structure. The study began with a geotechnical survey of the soil in other to determine the soil bearing capacity. The soil was studied at three points using a dynamic penetrometer as shown in figure 3.5. An Analysis of the structural elements of the building was made in other to assess the state of the building. The verification of the concrete quality and strength was done using the Schmidt sclerometer (beams, pillars and footing). Finally, the state of the iron reinforcement was verified using a Pachometer.

3.3. Data presentation

The data that are presented are the structural plan and the audit report.

3.3.1. Structural plan

The structural plan of the building showing the beams, pillar and foot is presented in figure 3.5. The inclined mezzanine and the dimensions of the structural elements are shown.

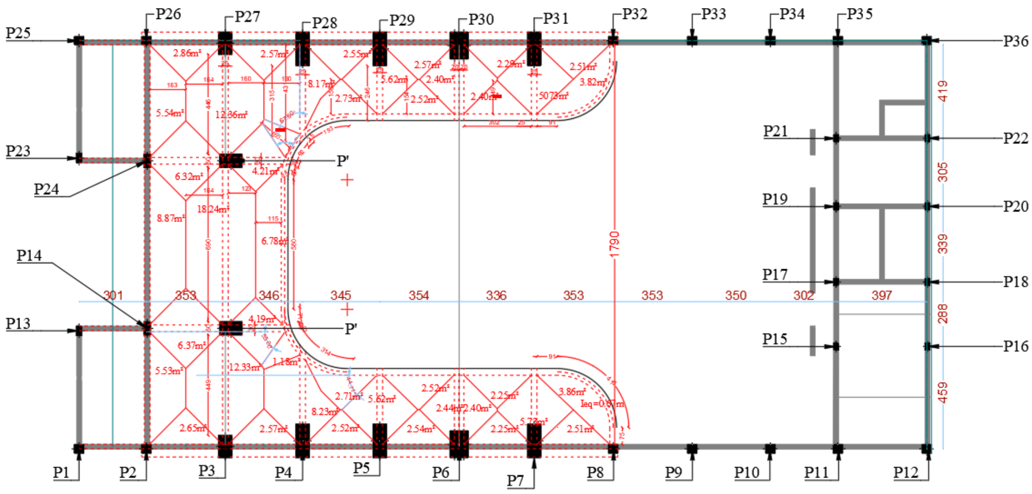


Figure 3.5. Structural plan indicating the pillars of EFLC church auditorium (AutoCAD, 2022)

3.3.2. Audit report

In the audit report, there was the geotechnical data after soil studies, the structural data and the different conclusions obtained after interpretation.

3.3.2.1. Geotechnical data

Using those three test points gotten from the dynamic penetrometer survey (DPS), a check of the water table was made and the result is presented in table 3.3. No water table was found after surveying the 3 points.

Table 3.3. Data on the water table (Geotechnical data, 2020)

Nº of Survey	Date of test	Depth of survey/ TN (m)	Depth of the water table / TN (m)	Comments
DPS 1	14/08/2020	9.80	-	No water table found
DPS 2		2.80		
DPS 3		2.80		

The result of the survey carried out with a dynamic penetrometer for the three points is shown in table 3.4.

Table 3.4. Soil resistance profiles (Geotechnical data, 2020)

DPS 1		
Depth (m)	Qd (kg/cm^2)	Soil Layer
0.0 – 2.40	16 - 80	1st layer
2.40 – 6.80	40 - 132	2nd layer
6.80 – 9.80	21 - 170	3rd layer
9.80	≥ 170	End of test
DPS 2		
Depth (m)	Qd (kg/cm^2)	Soil Layer
0.0 – 2.40	47 - 152	1st layer
2.40 – 2.80	168 - 366	2nd layer
2.80	≥ 366	End of test
DPS 3		
Depth (m)	Qd (kg/cm^2)	Soil Layer
0.0 – 1.60	24 - 86	1st layer
1.60 – 2.80	86 - 366	2nd layer
2.80	≥ 366	End of test

The result of the soil bearing capacity for each survey according to the anchorage depth is given in table 3.5.

Table 3.5. Allowable bearing capacity with respect to anchorage depths (Geotechnical data, 2020)

Anchorage depth (m)	**Allowable bearing capacity (σ_{adm} = Qd / 20 in kg/cm^2) depending on the anchorage depths**			
	DPS 1	DPS 2	DPS 3	Average
1.00	1.69	3.38	1.27	2.11
1.5	0.98	2.55	2.74	2.09
2.0	0.78	2.35	13.71	5.61
2.5	4.75	12.98	16.09	11.27
3.0	6.58	High resistance level	High resistance level	6.58
3.5	6.51	-	-	3.77
4.0	3.77	-	-	2.58
5.0	2.58	-	-	3.05
6.0	3.05	-	-	1.73

7.0	1.73	-	-	3.57
8.0	3.57	-	-	1.04
9.0	1.04	-	-	
10.0	End of test	-	-	

Because of the results of the penetrometer tests, a suggestion was made to take the following bearing capacity, corresponding to the types of precise foundations.

Foundations on isolated footings anchored at a depth of 1.50 m from the level of the current natural ground and working at 2.00 kg/cm2 at Ultimate limit state.

Foundations on isolated footings anchored at a depth of 2.5 m from the level of the current natural ground and working at 4.00 kg/cm2 at Ultimate limit state.

From the geotechnical survey at the 3 points shown in figure 3.6, it was noticed that in other to have a good bearing capacity for the incoming loads, the foundation was to be done at a depth of 2.5 m. hence there was a need for redesigning the footing considering the new depth

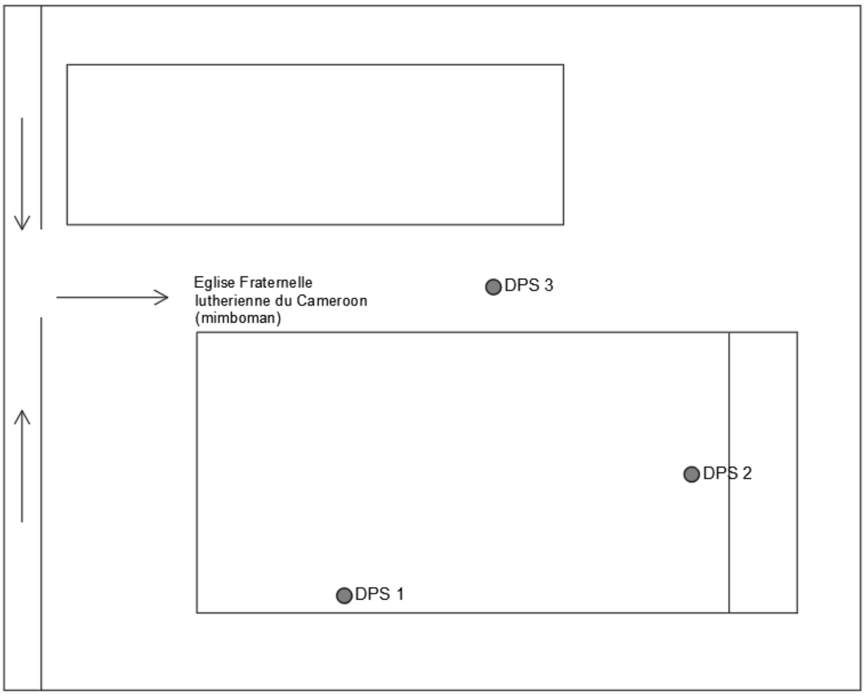


Figure 3.6. The three points of the dynamic penetrometer survey (Geotechnic survey, 2020)

3.3.2.2. Structural data

From the data of the sclerometer, it was noticed that all the structural elements had a resistance of more than 20 MPa and hence they were strong enough to resist the new coming load. Analysis with a pachometer showed that the reinforcement bars were in a good state with no corrosion. Hence, the structural elements were in a good state even though they were not strong enough to resist the new loads on the structure.

3.4. Structural analysis results

Here, there is a presentation of the values of the load that was applied to the structure (the permanent and variable loads) and the numerical model. Also, the stress on the pillars and the model verifications are presented.

3.4.1. Load determination

The actions on the structure have been defined in chapter 2.4.1.2 as well as their methods of evaluation. They all have different characteristic values. These values will be all presented in the tables to facilitate their understanding.

3.4.1.1. Permanent loads

The permanent loads that will act on the structure are in two categories. The entire self-weight of the structure and the non-structural permanent loads.

a. Self-weights

The self-weight of the structural elements like columns and beams will be automatically taken into consideration by the software during the analysis. The weight of the roof will be considered as a load acting on the structure. Also, at the level of the mezzanine, the bleacher will be considered as distributed loads on the slab of the mezzanine and added to that, the self-weight of the slab of a thickness of 15 cm will be applied on the beams of the mezzanine. The values of the self-weights are presented in table 3.6.

Table 3.6. Self-weights on structure

Element	Symbol	Value (kN/m^2)
Roof	G_{rf}	0.6
Slab	G_{slb}	3.75
Bleacher	G_{bl}	4.79

b. Nonstructural permanent loads

The non-structural permanent loads may include, maintenance of roof and ceiling and their values are shown in table 3.7.

Table 3.7. Non-structural permanent loads on the building

Element	Symbol	Value (kN/m^2)
Maintenance on the roof	G_{mrf}	0.4
Wood ceiling	G_{ceil}	1

3.4.1.2. Variable loads

The values for the imposed load and those for the wind loads are presented.

a. Imposed loads

Since the building falls under the use category of C2 and from table 2.3 we obtain the value presented in Table 3.8.

Table 3.8. Impose variable load

Categories of the loaded area	Symbol	Value (kN/m^2)
C2	qk	4.0

b. Wind loads

The wind load was determined using the equations in 2.4.1.2. The value of the parameters and wind load are presented in table 3.9.

Table 3.9. Wind loads on the building

Parameter	Symbol	Value
Directional factor	C_{dir}	1.0
Season factor	C_{season}	1.0
Reference wind	$V_{b,0}$	22 m/s
Basic wind velocity	$\mathbf{V_b}$	**22 m/s**
Roughness length	Z_0	0.05 m
Terrain factor	k_T	0.019
Orography factor	$C_o(z)$	1.0
Roughness factor	$C_r(z)$	0.096
Mean wind	$\mathbf{V_m(z)}$	**2.112 m/s**
Turbulence factor	k1	1.0
Wind turbulence	$\mathbf{I_v}$	**0.197**
Exposure factor	$C_e(z)$	2.2
Basic velocity pressure	q_b	0.3025 kN/m^2
Peak velocity pressure	**qp(z)**	**0.67 kN/m^2**

3.4.1.3. Presentation load influences area

The influence area of the loads that will be acting on our structure, and that will enter each pair of opposite pillars is presented in figure 3.7.

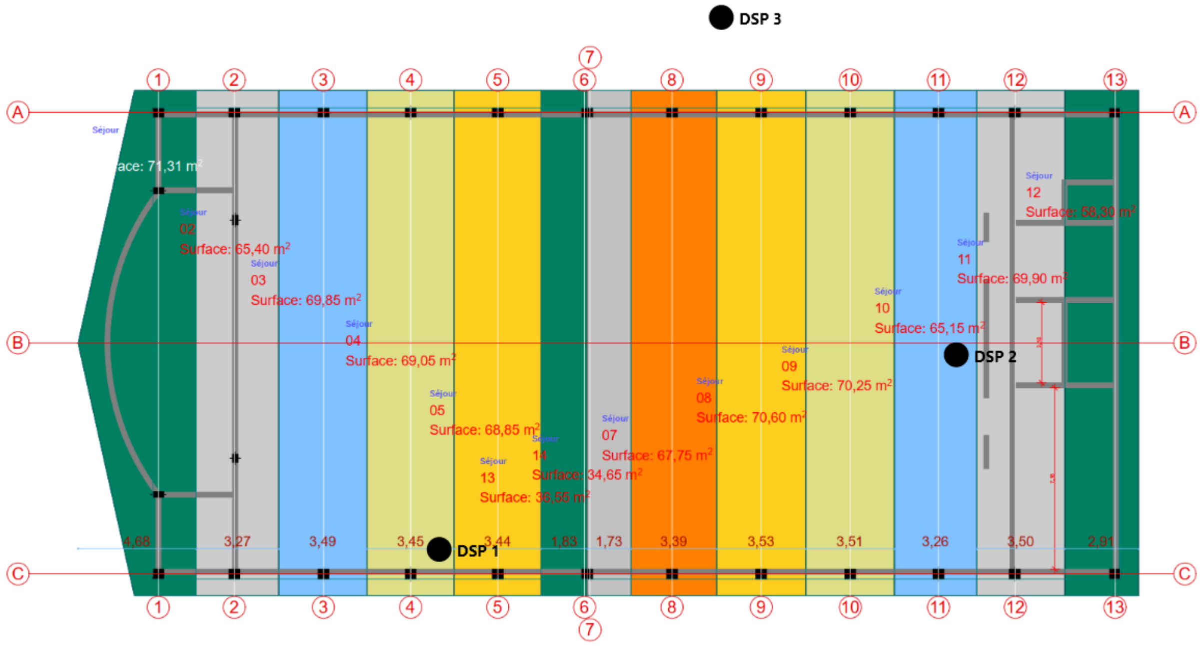


Figure 3.7. Influence area of load acting on each pillar and position of survey points (AutoCAD, 2022)

3.4.2. Presentation of numerical model

Two numerical models of the structure were presented. The first model was that of the building before the mezzanine was constructed as seen in figures 3.8 and figure 3.9 and the second model was the model of the actual building with the mezzanine and is presented in figures 3.10 and figure 3.11.

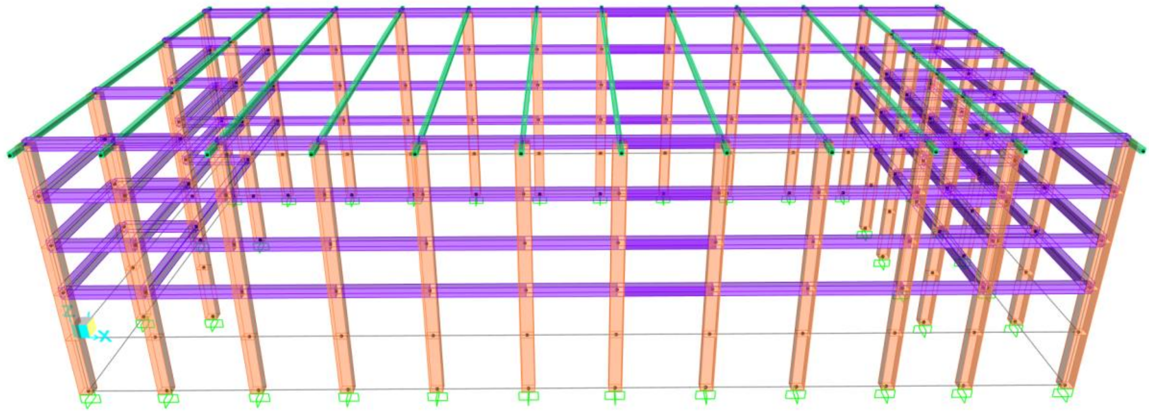


Figure 3.8. 3D model of building without mezzanine, lateral view (Sap2000, 2022)

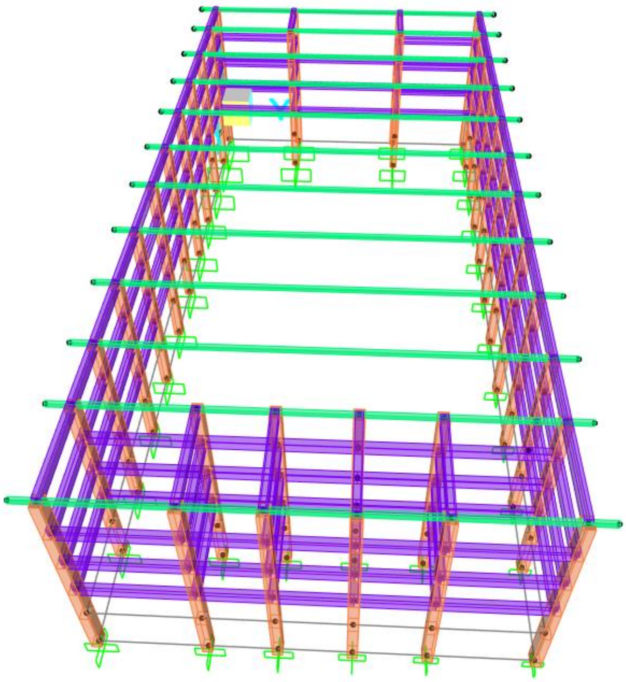


Figure 3.9. 3D model of the building without mezzanine, top view (Sap2000, 2022)

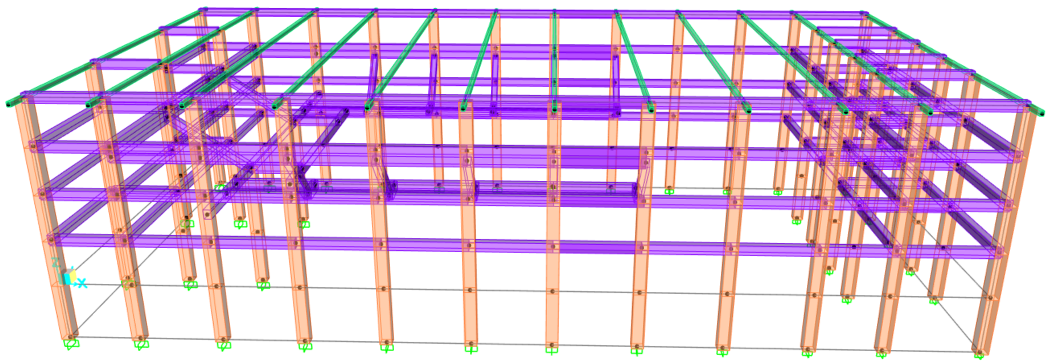


Figure 3.10. 3D model of building with the mezzanine, lateral view (Sap2000, 2022)

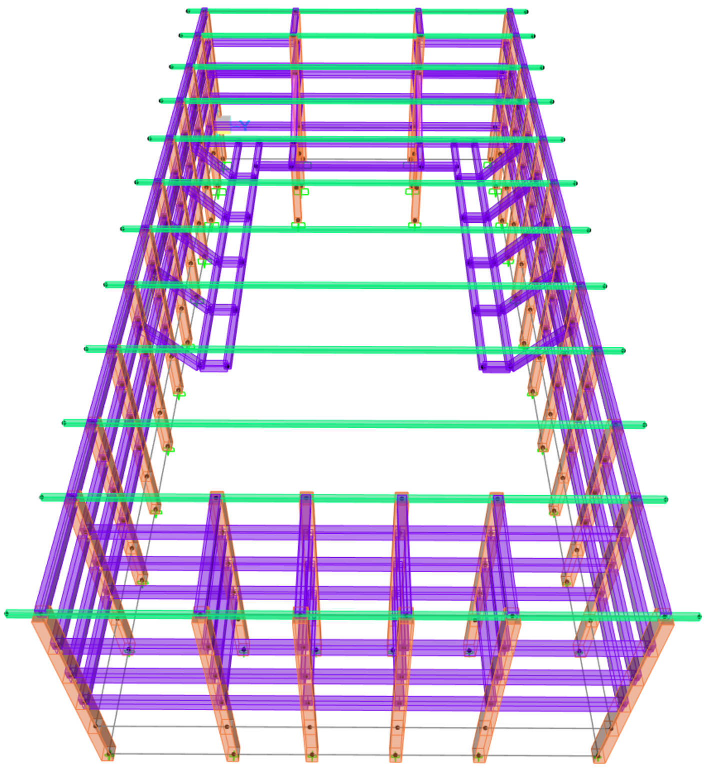


Figure 3.11. 3D model of building with mezzanine, top view (Sap2000, 2022)

3.4.3. Presentation of the stress on the pillar

The focus of the analysis was to determine the stresses on the building and thereby noting the pillar subjected to the greatest stress. The presentation of the maximum axial forces and maximum moment on each pillar will be done in table 3.10.

Table 3.10. Maximum axial forces and maximum moment on each pillar

Pillar	Axial force (kN)	Moment 2-2 (kNm)	Moment 3-3 (kNm)
P’	557.53	62.02	9.8224
P1	168.2	2.29	22.31
P2	214.63	3.65	28.58
P3	281.01	12.82	294.66
P4	564.52	4.47	297.5
P5	**572.02**	**5.16**	**302.74**
P6	559.77	5.58	299.23
P7	550.14	6.71	299.03
P8	416.94	8.4	306.29
P9	249.16	4.66	303.14
P10	223.34	3.83	267.03
P11	193.78	2.04	21.28
P12	139.75	6.04	3.22
P13	211.74	7.57	22.07
P14	442.81	68.56	20.74
P15	185.23	3.03	9.13
P16	157.35	6.06	1.85
P17	168.7	3.18	0.30
P18	146.89	5.92	0.10
P19	170.37	3.16	0.97
P20	148.02	5.93	0.183
P21	184.93	3.03	11.78
P22	157.07	6.05	2.4
P23	211.75	7.57	22.07
P24	442.82	68.56	20.74
P25	168.2	2.3	22.31
P26	214.62	3.65	28.58
P27	281.01	12.81	294.66
P28	564.52	4.47	297.5
P29	572.02	5.16	302.74

P30	559.77	5.57	299.23
P31	550.14	6.71	299.03
P32	416.94	8.4	306.29
P33	249.16	4.66	303.14
P34	223.18	3.88	267.08
P35	191.51	2.05	23
P36	138.28	6.01	3.15

From the result of the stresses on the pillars, the pillar with the highest amount of stress is pillar 5. Hence pillar 5 will be considered for further analysis.

3.4.4. Presentation of model verification

The different verifications that were made on our model are presented. They included a modal analysis and an MN interaction verification of the pillars.

3.4.4.1. Modal presentation

The modal analysis shows that the first mode was a translation in the y direction, the second was a translation in the x direction and the third was torsion in the xy-plane and this is seen in figures 3.10 to 3.12. The period and frequency of vibration of the building are presented in table 3.11. Hence this verifies the regularity of the building. The regularity of the structure is dependent on the symmetrical and compact shape of the structure. The importance of regularity of the building is for avoiding unpredictable stress concentration that can cause local collapses and modification of the dynamic behaviour.

Table 3.11. Period and frequency of vibration of the building

Mode	Period (s)	Frequency (s^{-1})
1	0.29337	3.40869
2	0.24573	4.06955
3	0.23029	4.3424

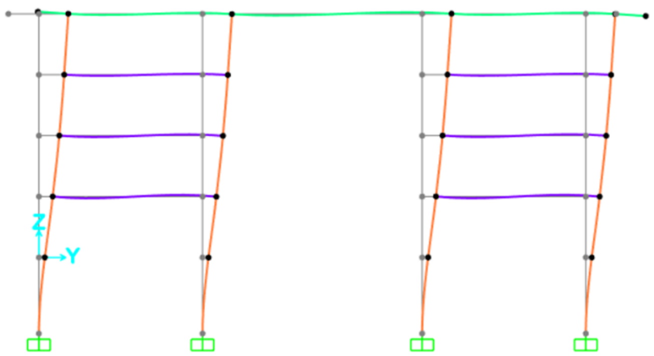


Figure 3.12. Mode 1, translation in the y direction (Sap2000, 2022)

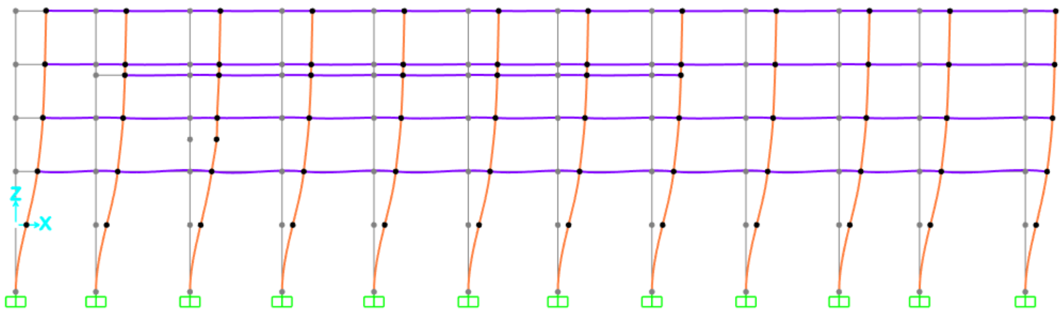


Figure 3.13. Mode 2, translation in the x direction (Sap2000, 2022)

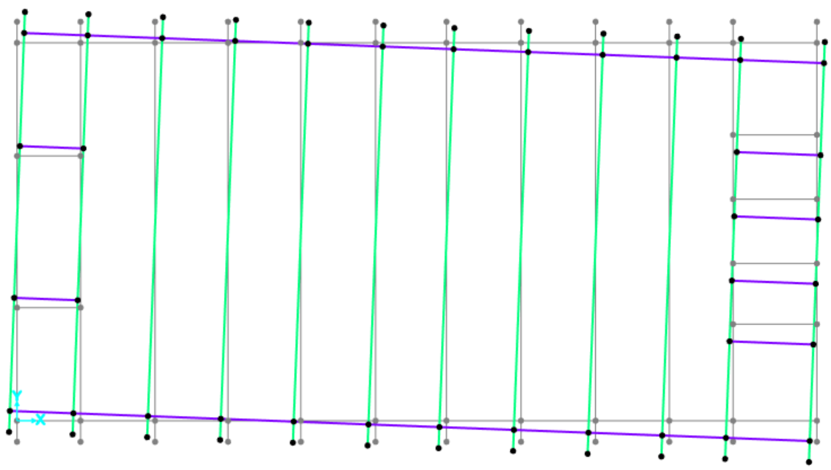


Figure 3.14. Mode 3, rotation in the xy plane (Sap2000, 2022)

3.4.4.2. Pillar interaction diagram

The points that make up the curve of the column interaction diagram considering moments 3-3 are presented in table 3.12. A section of the initial column is shown in figure 3.15. The interaction curve, plotted in excel is shown in figure 3.16. In that same light, the table showing the points for the interaction curve considering moments 2-2 and the actual plot of the curve in MS office excel are presented in table 3.13 and figure 3.18 respectively. In other to obtain the MN-interaction curve for moments 2-2, the rotated section of the initial column was considered as shown in figure 3.17. The interaction diagrams show zero moments at the extremity corresponding to pure compression and pure tension which agrees with the physical situation.

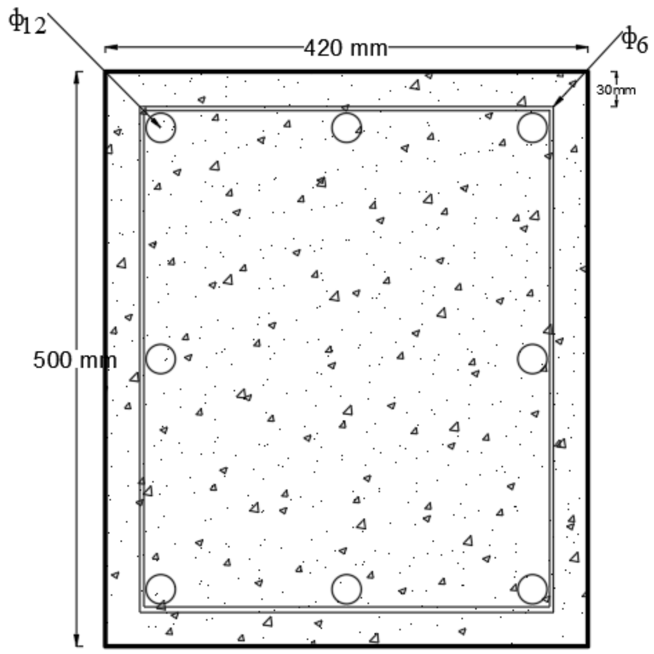


Figure 3.15. Initial concrete column section for moment 3-3 (AutoCAD, 2022)

Table 3.12. Points for column interaction curve considering moment 3-3

Point	Axial (kN)	Moment (kNm)
1	2684.47	0
2	1415.79	223.5
3	748.63	196.61
4	0	70.49
5	-314.68	0

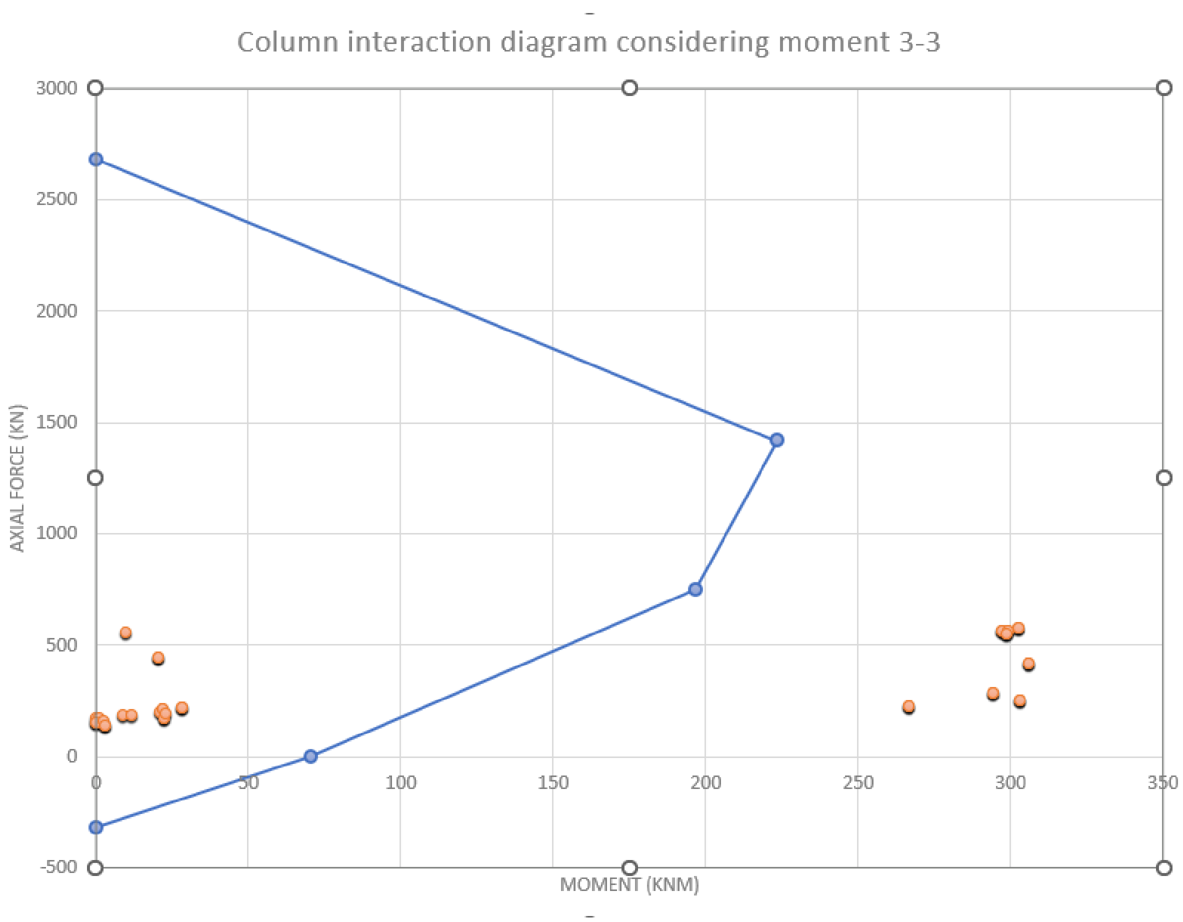


Figure 3.16. MN-interaction diagram of column for moments 3-3 with a plot of the maximum stresses on the pillars (MS Office Excel, 2022)

Table 3.13. Points for column interaction curve considering moment 2-2

Point	Axial (kN)	Moment (kNm)
1	2684.5	0
2	1395.29	194.924
3	823.98	169.96
4	0	58.67
5	-314.68	0

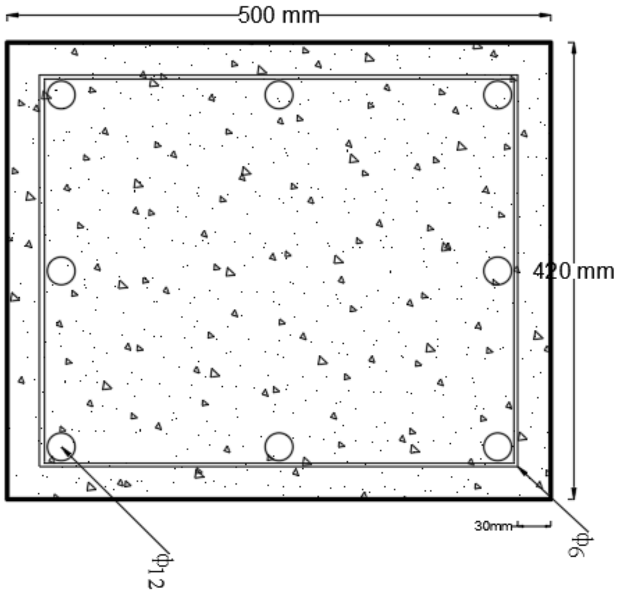


Figure 3.17. Rotation of initial concrete column section to obtain moment 2-2 (AutoCAD, 2022)

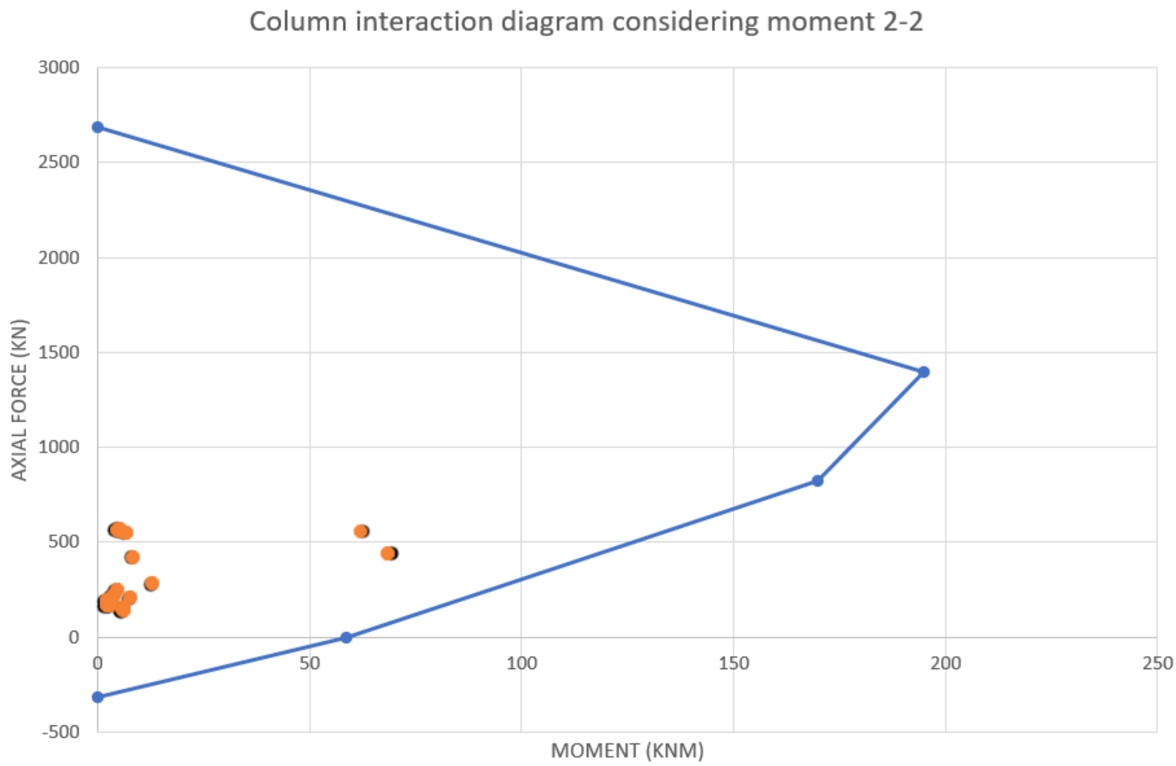


Figure 3.18. MN-interaction diagram of column for moments 2-2 with a plot of the maximum stresses on the pillars (MS Office Excel, 2022)

From the results displayed in figure 3.16, we see that in the plot of the stresses on each pillar, some points are found in the interaction curve and others are found outside. This shows that they are two categories of pillars, those pillars that have design strength greater than the stresses they experience (inside the interaction curve), and those that experience stresses greater than that they had been designed to withstand. Hence, this second category of pillars is not safe. So, they will be a need of strengthening the pillars in other to make them withstand the incoming moments and forces.

On the other hand, the results displayed in figure 3.18 shows that the cross-section of the initial column is enough to resist the axial forces and the moment 2-2 acting on our pillar.

Therefore, the strengthening will be done regarding the high values of moments 3-3. The pillar with the greatest amount of stress (P5) will be considered as a design reference for the strengthening intervention.

3.4.4.3. Column verification in Abaqus

The elasticity and plasticity parameters of concrete are presented in table3.14. The Concrete behaviour and damage properties are seen in table 3.15. Steel reinforcement properties of the main bar and stirrups are presented in table 3.16. The column modelled on Abaqus is presented in figure 3.19

Table 3.14. Density, elasticity and plasticity parameter of concrete C25/30 (Hafezolghorani et al., 2017)

Density	
Density ρ	2.4E-06 kg/mm3
Elastic parameter	
Parameter	**value**
Young's modulus (E)	21200 MPa
Poisson ratio	0.2
Plasticity parameter	
Parameter	**value**
Dilation angle	31
Eccentricity	0.1
f_{bo} / f_{co}	1.16
K	0.67
Viscocity parameter	0

Table 3.15. Concrete behaviour and damage properties C25/30 (Hafezolghorani et al., 2017)

Concrete compressive behaviour		**Concrete compression damage**	
Yield stress (MPa)	Inelastic strain	Damage parameter C	Inelastic strain
10.2	0	0	0
12.8	7.74E-05	0	7.74E-05
15	0.000173585	0	0.000173585
16.8	0.000288679	0	0.000288679
18.2	0.000422642	0	0.000422642
19.2	0.000575472	0	0.000575472
19.8	0.00074717	0	0.00074717
20	0.000937736	0	0.000937736
19.8	0.00114717	0.01	0.00114717
19.2	0.001375472	0.04	0.001375472
18.2	0.001622642	0.09	0.001622642
16.8	0.001888679	0.16	0.001888679
15	0.002173585	0.25	0.002173585
12.8	0.002477358	0.36	0.002477358
10.2	0.0028	0.49	0.0028
7.2	0.003141509	0.64	0.003141509
3.8	0.003501887	0.81	0.003501887
Concrete tensile behaviour		**Concrete tension damage**	
Yield stress (MPa)	Cracking strain	Damage parameter T	Cracking strain
2	0	0	0
0.02	0.000943396	0.99	0.000943396

Table 3.16. Steel reinforcement properties B400C

Density	
Density ρ	7.8E-06 kg/mm3
Elastic parameter	
Parameter	**value**
Young's modulus (E)	210000 MPa
Poisson ratio	0.3
Plasticity parameter	
Yield stress	**Plastic strain**
280	0
370	0.1

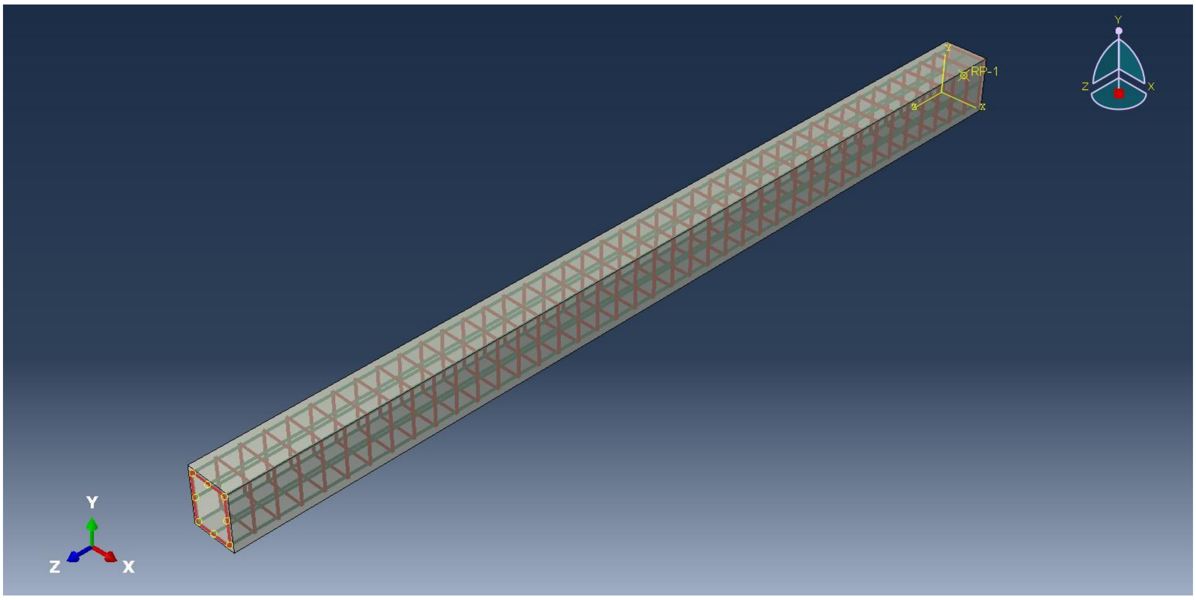


Figure 3.19. The initial concrete column model (Abaqus, 2022)

After the analysis was performed on Abaqus, because of the large amount of the load notably the maximum moment (302.74 kNm) and its corresponding axial load (572.02 kN) obtained previously from the global analysis, the analysis of the software was aborted. A message indicating the error in the analysis was sent as presented in figure 3.20 independent of the type of analysis done. This shows that the concrete column section can not resist the incoming load. The loads are too high in comparison to the column section. Therefore, that validates the fact that the plot of that point that represents the maximum load case was found outside the interaction curve as previously presented in figure 3.16.

3.5. Intervention on column

Three different strengthening jacketing methods were studied in other to strengthen our pillar P5. These different strengthening methods may be used as alternative retrofitting of technics for our pillar. These are concrete jacketing, steel jacketing and carbon fiber reinforced polymer.

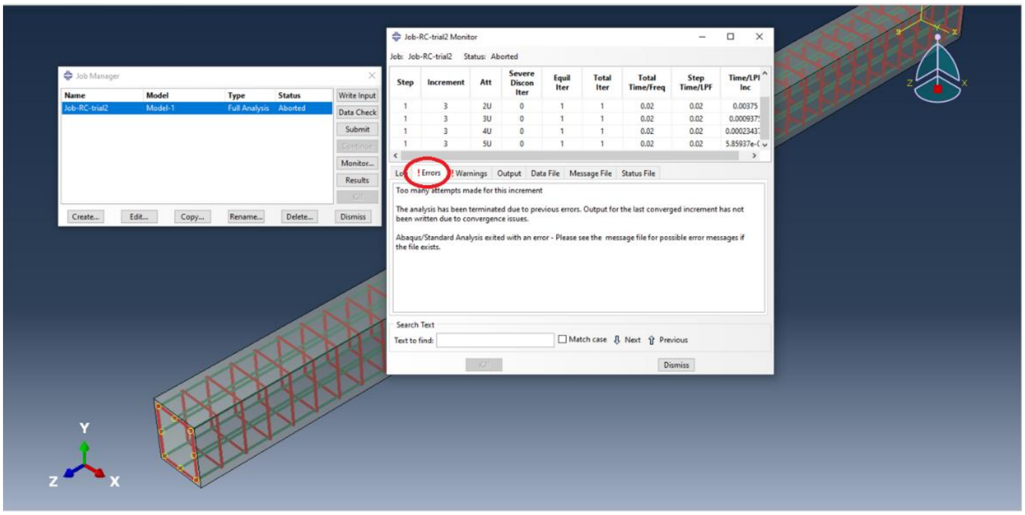


Figure 3.20. Message indicating error in analysis (Abaqus, 2022)

3.5.1. Concrete jacketing of pillar

In this section, there will be a presentation of the design computation of the concrete jacket, an mn-interaction diagram for the new concrete jacket section and the stress distribution and the displacement that occurred throughout the loading of our column.

3.5.1.1. Design computation of the concrete jacket

The design specifications of the different elements of the concrete pillar are presented in table 3.17.

Table 3.17. Design specification of the parameter used in the concrete jacketing

Parameter	Symbol	Value
Characteristic strength of concrete	f_{ck}	25 N/mm^2
Characteristic strength of steel	f_{yk}	400 N/mm^2
Design strength of concrete	f_{cd}	14.17 N/mm^2
Design strength of steel	f_{yd}	347.83 N/mm^2
Depth of the original pillar section	h	500 mm
Width of the original pillar section	b	420 mm
Axial force	N	572.02 kN
Moment	M	302.74 kNm
Concrete cover	Cc	30 mm
Steel reinforcement in the initial pillar	Φ12	8 bars
Area of steel provided in the original section	A_{sp}	904.78 mm^2

The design of the concrete jacket was done following the methodology presented in 2.5.1.1.

$N_u = 0.4 * 14.17\ Nmm^2 * \left((420 * 500) - 904.789\ mm^2\right) + 0.67 * 347.83\ N\ mm^2 * 0$

$= 1185.15\ kN > N = 572.02\ kN$ Hence, our pillar can resist to axial load.

But, checking for the moment and using the interaction diagram for the rectangular section,

$$\frac{C_c}{h} = \frac{30}{500} = 0.06$$

$$\frac{N}{fcd * b * h} = \frac{572.02 * 10^3 N}{14.17 * 420 * 500} = 0.2$$

$$\frac{M}{fcd * b * h^2} = \frac{302.74 * 10^6 Nmm^2}{14.17 * 420 * 500^2} = 0.2$$

From the interaction diagram, we get the value of ω = 0.36

Hence the area of steel required are;

$$A_s = \frac{\omega * b * h * fcd}{f_{yd}} = \frac{0.36 * 420\ mm * 500\ mm * 14.17\ Nmm^2}{347.83\ Nmm^2} = 3079.8\ mm2$$

$$A_s = 3079.8\ mm2 > A_{sp} = 904.78\ mm2$$

The additional steel needed $A_s' = 3079.8 - 904.78 = 2175.02\ mm2$

So, we provide $\mathbf{12\ \phi 16}$ as reinforcement for our concrete jacket.

The minimum thickness of the jacket to be provided is 80 mm hence we provide 100 mm.

The diameter of literal ties = ¼ of the diameter of the largest longitudinal bar.

$\boldsymbol{\phi ties = 1/4 * 16 = 4\ mm}$ but we provide 8 mm as diameter.

Hence the revised jacketed section of the column will be 620 mm*700 mm.
The jacketed section for the column is given in figure 3.21

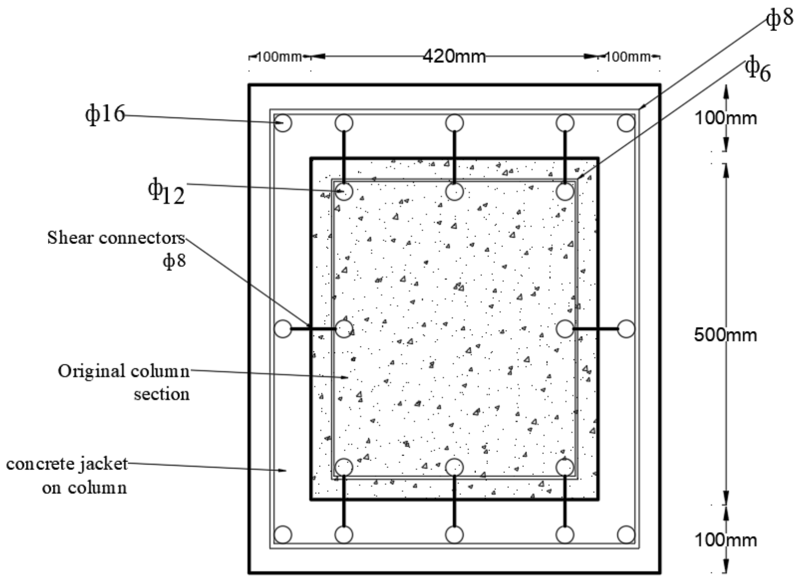


Figure 3.21. The section of the concrete jacketed column (AutoCAD, 2022)

3.5.1.2. MN interaction diagram for concrete jacketed column

The points computed for the plot of the interaction diagram for the concrete jacket column are presented in table 3.18. The concrete jacket interaction curve is presented on the same diagram as that of the original column interaction diagram for moments 3-3 in figure 3.22.

Table 3.18. Points for column interaction curve for concrete jacket

Point	Axial (kN)	Moment (kNm)
1	6035.09	0
2	3039.67	798.11
3	1753.81	690.15
4	0	449.30
5	-1153.91	0

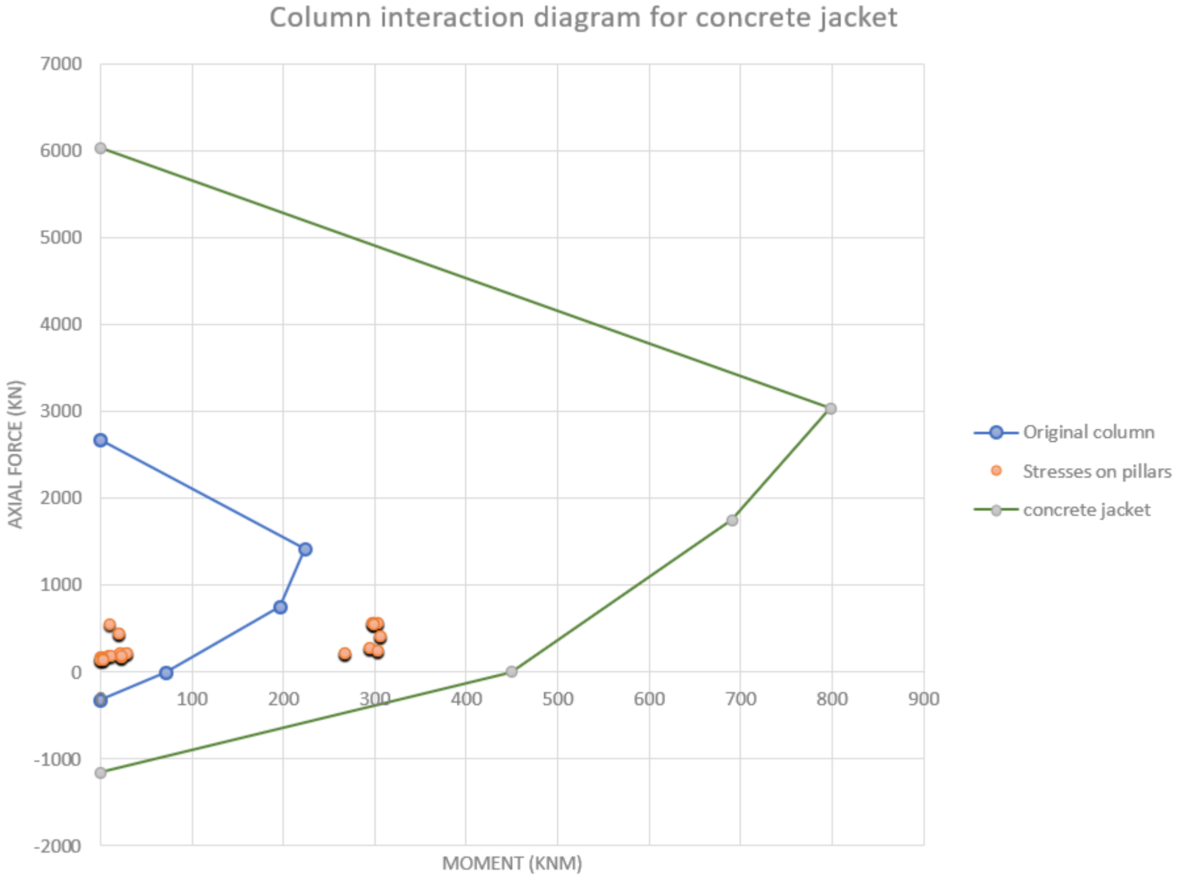


Figure 3.22. Column interaction diagram for concrete jacketed section with a plot of the maximum stresses on the pillars (MS Office Excel, 2022)

3.5.1.3. Concrete jacket model of column on Abaqus

The numerical model for the local analysis that was done on Abaqus is presented in figures 3.23 and figure 3.24. The same behaviour and damage property of the original concrete column as specified in table 3.15 above is used for the concrete jacket.

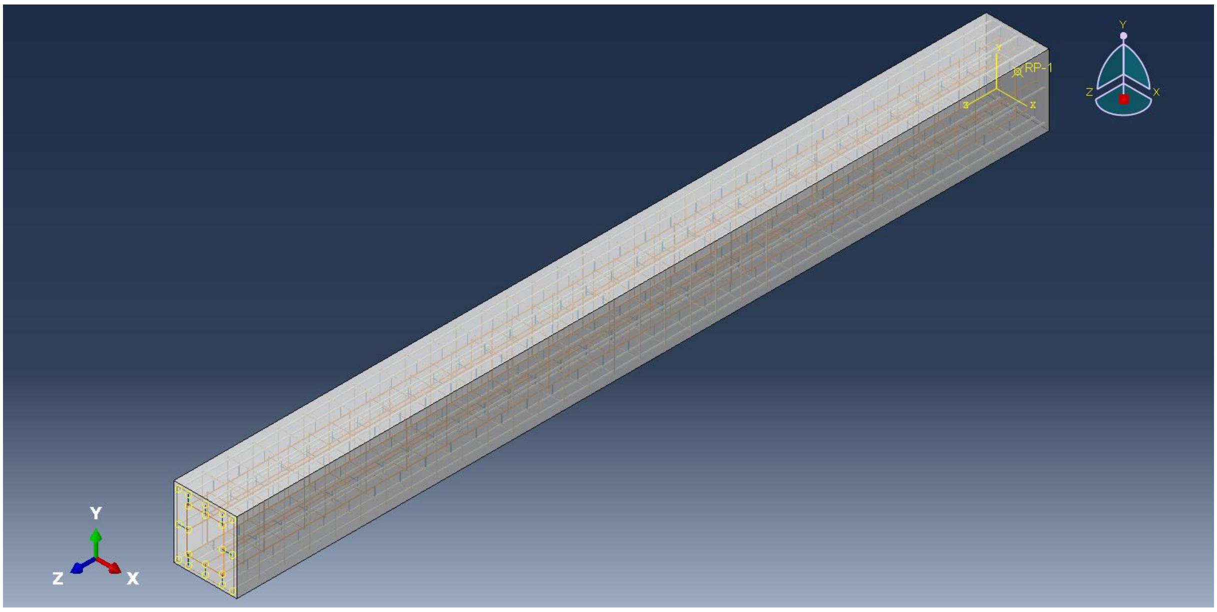


Figure 3.23. Concrete jacketed column section (Abaqus, 2022)

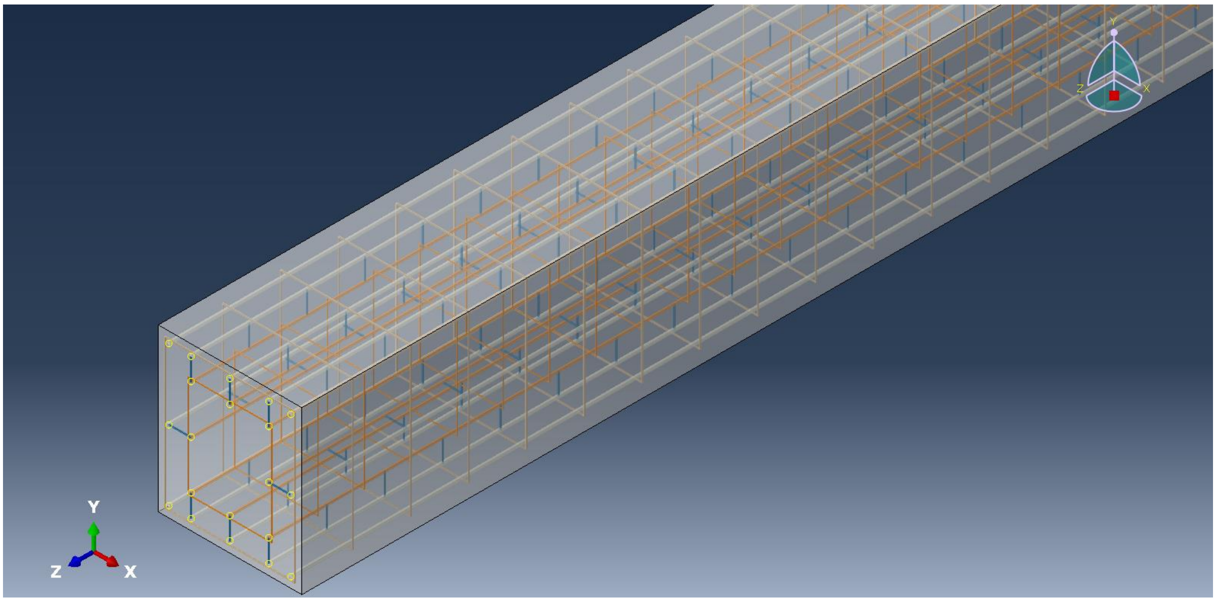


Figure 3.24. More detailed view of concrete jacket steel bar reinforcement (Abaqus, 2022)

Von Mises stress (S, Mise) is a value used to determine if a given material will yield or fracture. It is mostly used for ductile materials, such as metals. The von mise yield criterion states that if the von mises stress of a material under load is equal to or greater than the yield limit of the same material under simple tension then the material will yield. The mise stress distribution that occurred in the column is presented in figure 3.25

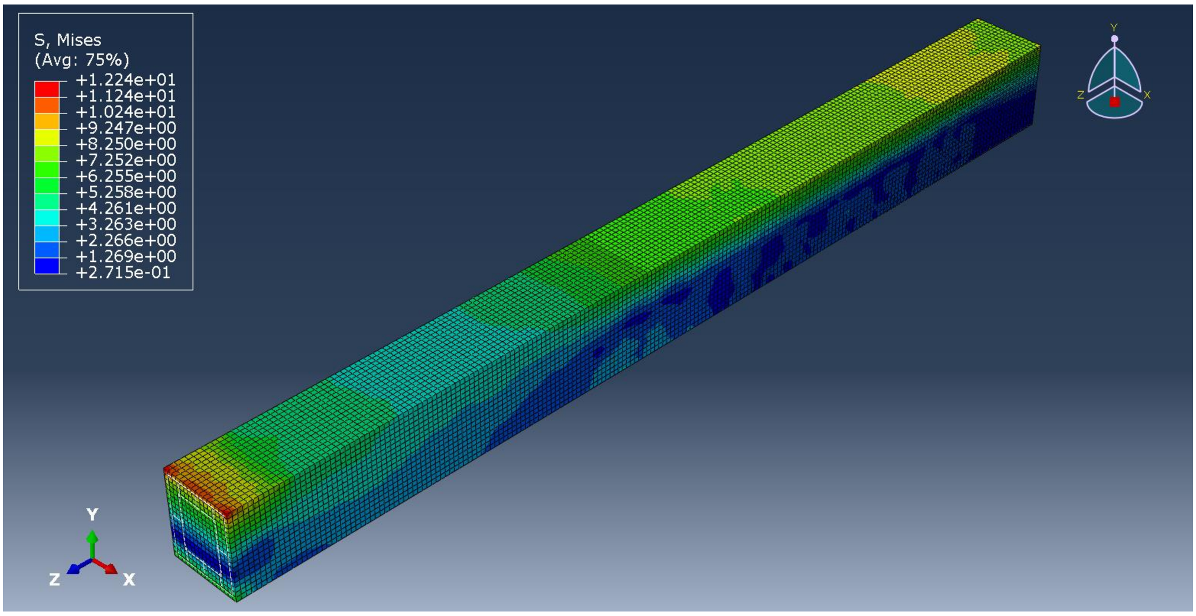


Figure 3.25. Mise stress distribution on concrete jacketed column (Abaqus, 2022)

Hence, it is seen from the column section that the maximum mise stress on the concrete jacketed section is 12.24 MPa and it is less than the yield stress of the steel reinforcement which is 400 MPa and that of the concrete characteristic strength of 25 MPa. So, the steel section resists the incoming loads without yielding.

The resultant displacements in mm that occurred due to the loading of the jacketed column are shown in figure 3.26. The displacement in the x-direction, y-direction and z-direction are presented in figures A.2, A.3 and A.4 respectively in the annex section.

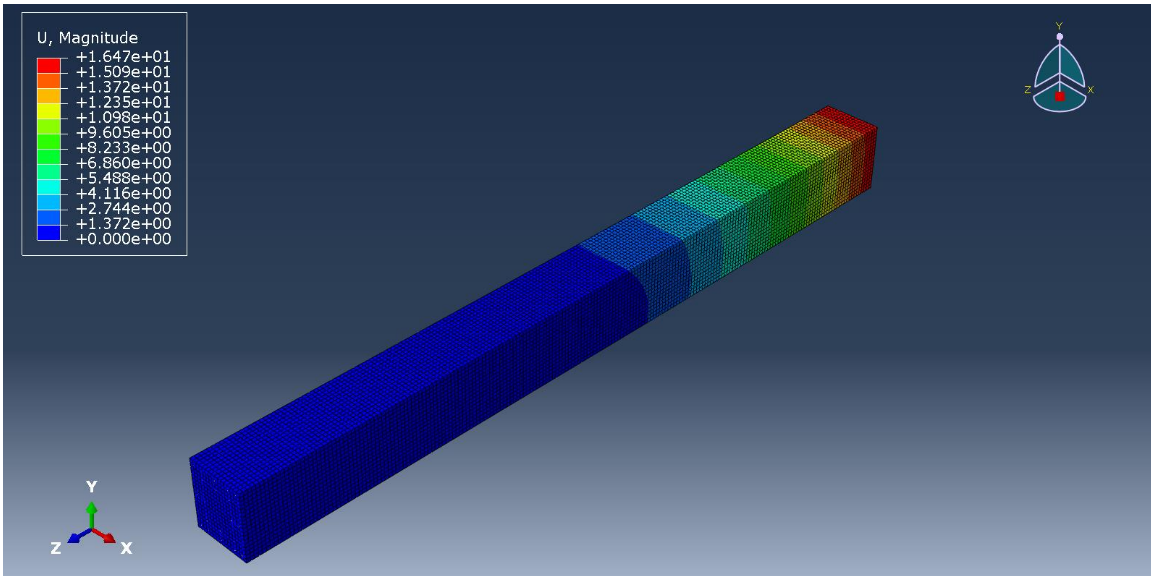


Figure 3.26. The resultant displacement on the concrete jacketed column (Abaqus, 2022)

A global and detailed view of vectors highlights of resultant displacement of the concrete jacketed column is presented in figures 3.27 and 3.28 respectively.

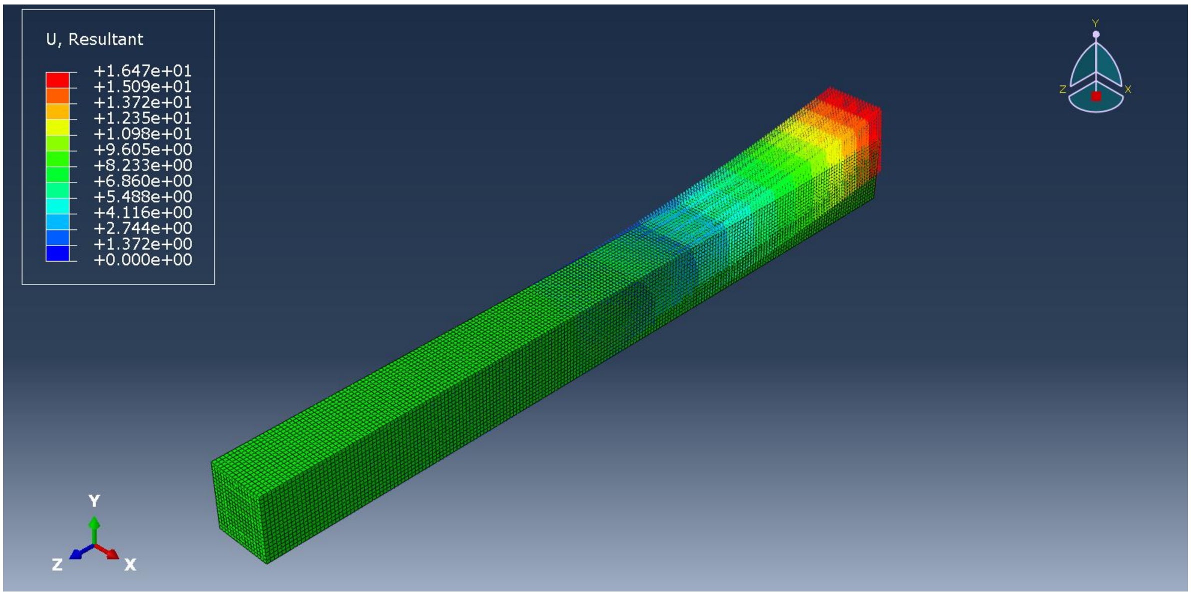


Figure 3.27. Global view of vectors highlights of resultant displacement of the concrete jacketed column (Abaqus, 2022)

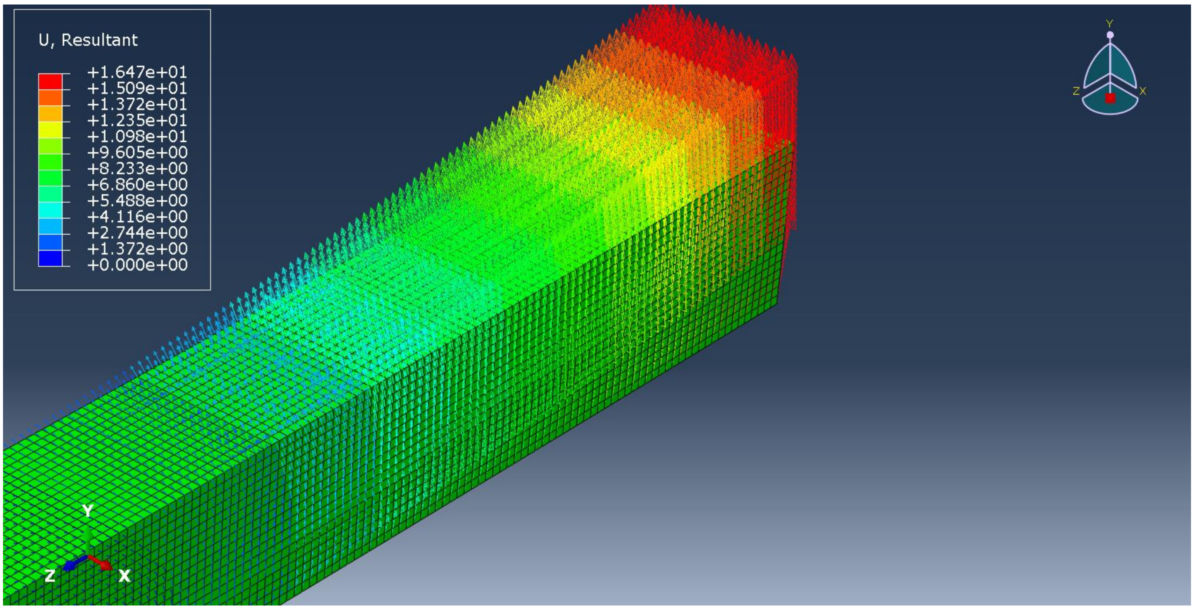


Figure 3.28. Detailed view of vectors highlights of resultant displacement of the concrete jacketed column (Abaqus, 2022)

Hence, it is seen from figure 3.26 above that the maximum displacement that the column will experience is 16.47 mm. Therefore, the column resists well to the incoming loads with very small displacements.

3.5.2. Steel jacket of column

Here, there is the presentation of the choice of the steel jacket reinforcement, the steel jacket interaction diagram used for verification and the model with the stress distribution made from the local analysis on Abaqus.

3.5.2.1. Choice of Steel jacket column section

The choice of the steel jacket section is gotten from table 3.19. The steel jacket section is going to be a steel angle of dimension 4 L 80 × 8 mm and strips 70 × 7 mm at 250 mm spacing between trips and with a yield strength of 275 MPa. A section of the column reinforced with a steel jacket is shown in figure 3.29.

Table 3.19. The characteristics of the specimens analyzed in the parametric study (Al-Sherrawi & Salman, 2017)

Specimen	**Cross-section (mm)**	**Steel angle (mm)**	**Longitudinal bars**	**(MPa) $f'c$**	**(MPa) fya**	**(MPa) fyr**
1	260 × 260	4 L 60 × 6	4 φ 12 mm	12	275	500
2	260 × 260	4 L 50 × 5	4 φ 12 mm	12	275	500
3	260 × 260	4 L 70 × 7	4 φ 12 mm	12	275	500
4	260 × 260	**4 L 80 × 8**	4 φ 12 mm	12	275	500
5	260 × 260	4 L 60 × 6	4 φ 12 mm	12	355	500
6	260 × 260	4 L 60 × 6	4 φ 12 mm	20	275	500
7	260 × 260	4 L 60 × 6	4 φ 12 mm	30	275	500
8	260 × 260	4 L 60 × 6	4 φ 10 mm	12	275	500
9	260 × 260	4 L 60 × 6	4 φ 16 mm	12	275	500
10	260 × 260	4 L 60 × 6	4 φ 20 mm	12	275	500

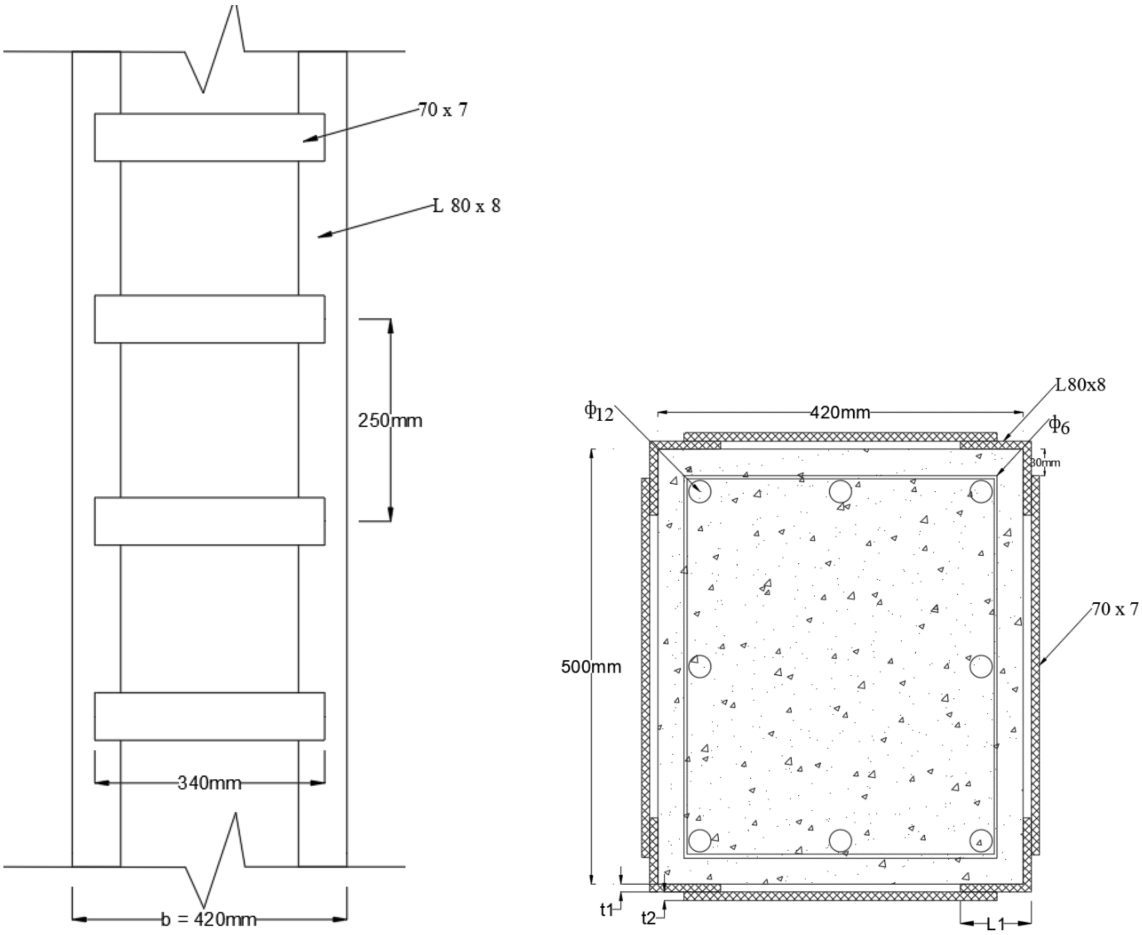


Figure 3.29. Lateral view and cross-section of column reinforced with steel jacket (AutoCAD, 2022)

3.5.2.2. Interaction diagram of column with steel jacket

The points that constitute the interaction curve are presented in table 3.20. The plot of the new steel jacket interaction curve on the original column interaction graph is presented in figure 3.30.

Table 3.20. Points for column interaction curve for column with steel jacket

Point	Axial (kN)	Moment (kNm)
A	3431.2	0
B	2975.7	400.06
C	1487.85	530.6
D	0	400.06
E	-1722.7	0

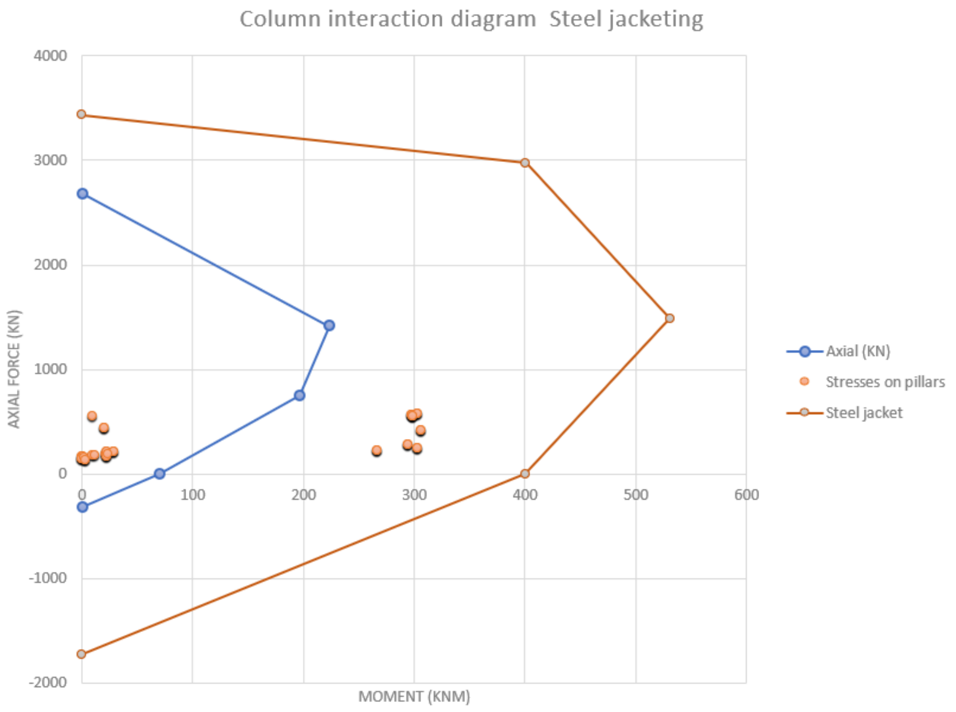


Figure 3.30. Column interaction diagram for steel jacketed section with a plot of the maximum stresses on the pillars (MS Office Excel, 2022)

3.5.2.3. Steel jacket model of column on Abaqus

The numerical model of the steel jacket column that was done on Abaqus is presented in figures 3.31, 3.32 and 3.33. S275 steel was used. The properties of the steel used for the jacket are presented in table 3.21.

Table 3.21. Steel reinforcement properties S275

Density	
Density ρ	7.8E-06 kg/mm3
Elastic parameter	
Parameter	**value**
Young's modulus (E)	210000 MPa
Poisson ratio	0.3
Plasticity parameter	
Yield stress	**Plastic strain**
275	0
410	0.14

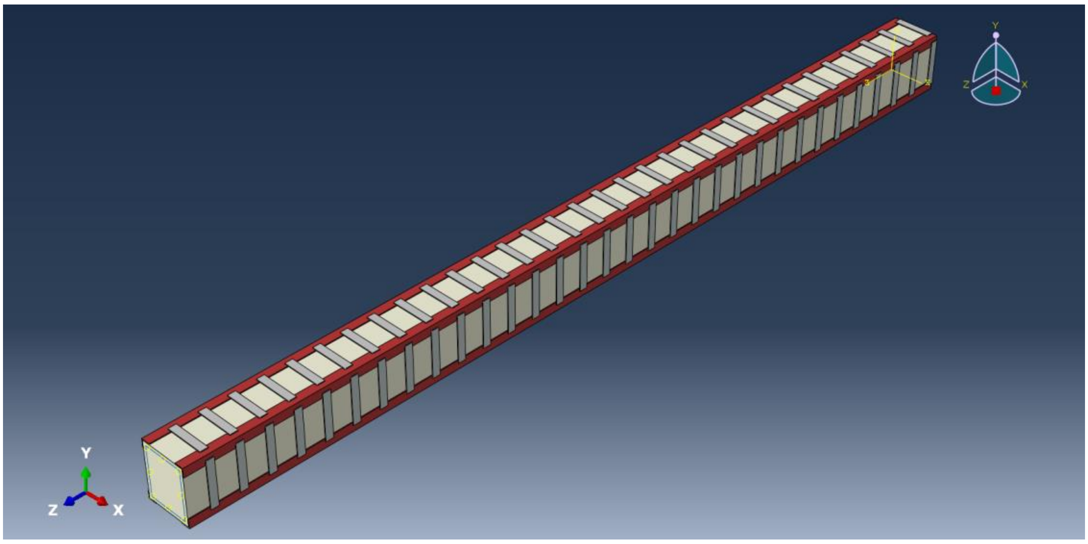


Figure 3.31. Steel jacketed column section (Abaqus, 2022)

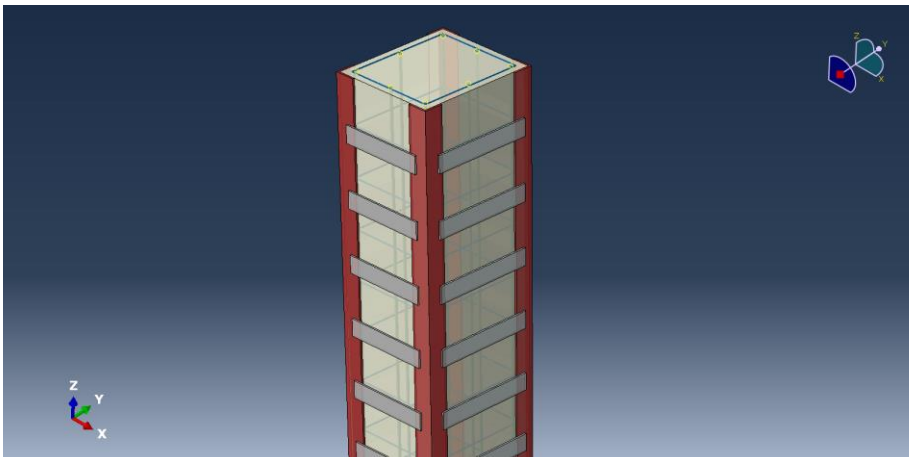


Figure 3.32. Steel jacketed column showing column bar reinforcement (Abaqus, 2022)

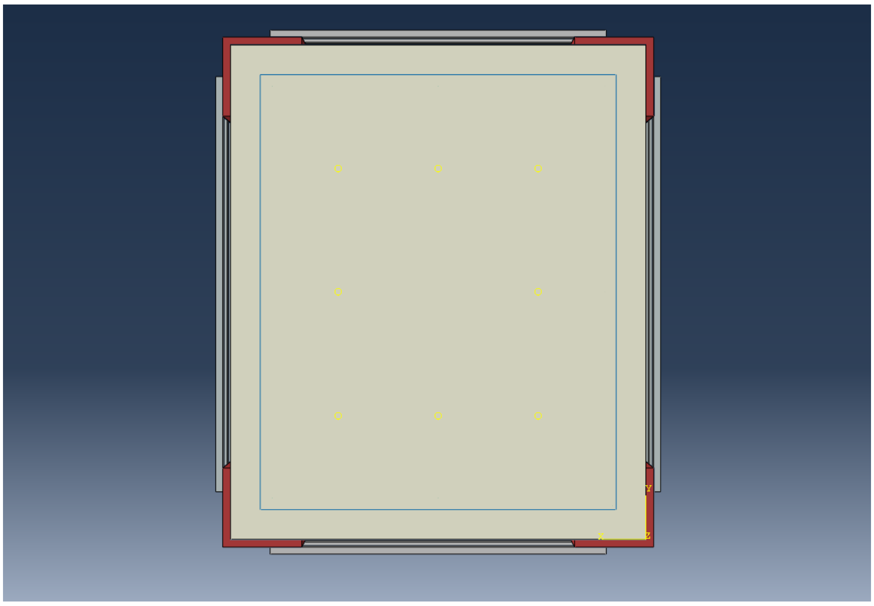


Figure 3.33. Cross section of column with steel jacket (Abaqus, 2022)

The mise stress distribution that occurred in the steel jacketed column is presented in figure 3.34. The maximum mise stress is 211.4 MPa and it is inferior to the yield stress of the steel jacket which is 275 MPa. Hence, the steel jacket will resist the incoming loads perfectly without yielding.

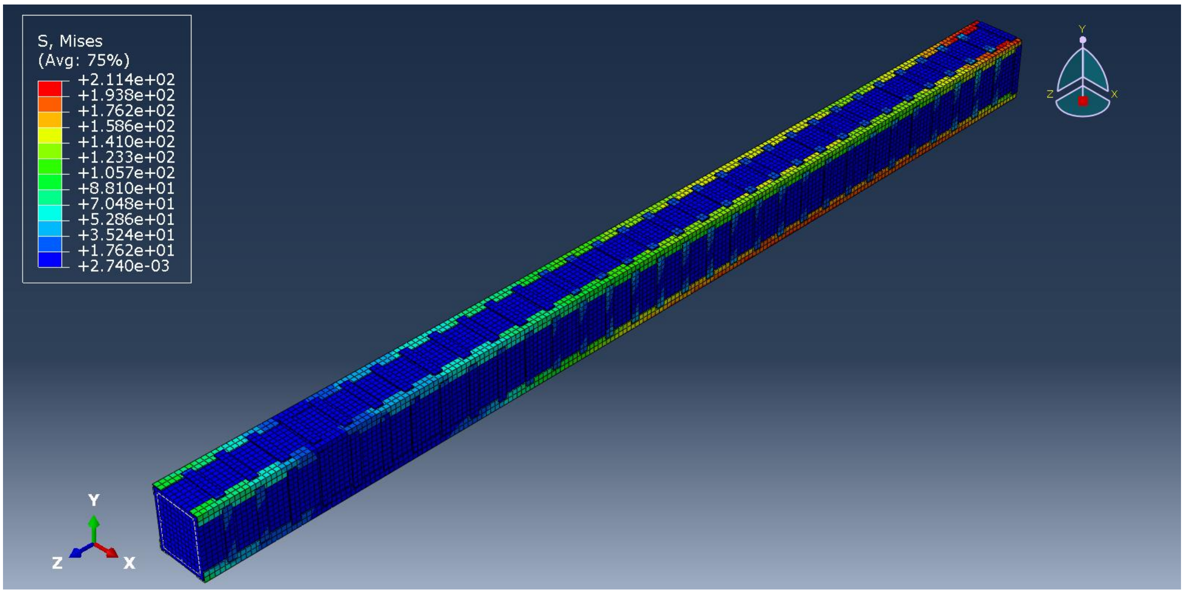


Figure 3.34. Mise stress distribution on the steel jacketed column section (Abaqus, 2022)

The resultant displacements that occurred due to the loading of the jacketed column are shown in figures 3.35 and 3.36. The maximum displacement on the column is 34.21 mm. Hence the column experience a relatively low amount of displacement. Hence we see that the steel jacket experiences a greater displacement and stress than the above concrete jacket.

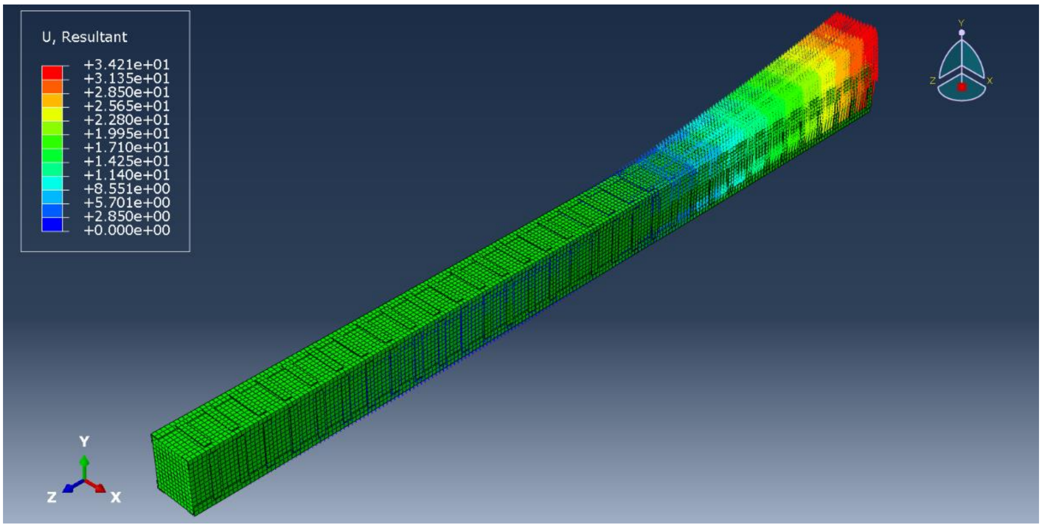


Figure 3.35. Global view of vectors highlights of resultant displacement of the steel jacket column (Abaqus, 2022)

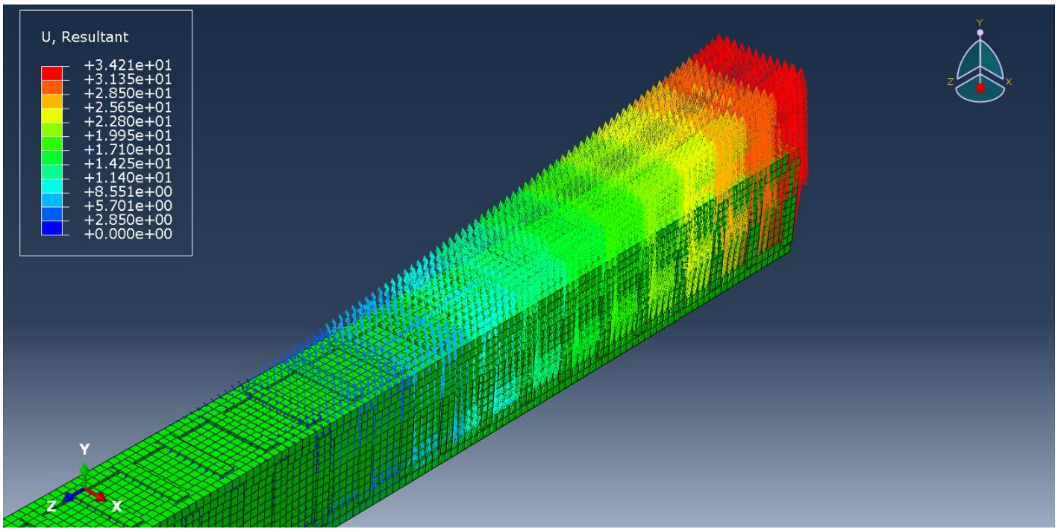


Figure 3.36. Detailed view of vectors highlights of resultant displacement of the steel jacket column (Abaqus, 2022)

3.5.3. Carbon fiber reinforced polymer jacket of column

The choice of the CFRP to be used, the interaction diagram for the verification of the new section made for the initial column and the CFRP and the presentation of the stress distribution on the column are discussed under this topic.

3.5.3.1. Choice of carbon fiber reinforced polymer

The choice of the CFRP is obtained from the table in figure 3.37. A CFRP of grade T300 is chosen for our design. The design parameters of the CFPR are presented in table 3.22. The characteristics of the original column are the same as those presented in table 3.14. A section of the column reinforced with CFRP is shown in figure 3.38.

Carbon Fiber Category Reference

Grade	Tensile Strength (MPa)	Tensile Strength (kgf/mm²)	Tensile Modulus (Gpa)	Tensile Modulus (kgf/mm²)	Density (g/cm²)	Equivalent tensile breaking strength @ thickness(mm)
T300/T300B	3530	360	230	23500	1.76	4.99
T400HB	4410	450	250	25500	1.8	4
T700SC	4900	500	230	23500	1.8	3.6
T700HB	5490	560	294	30000	1.81	3.21
T800SC	5880	600	294	30000	1.8	3

Remark: Equivalent tensile breaking strength means the carbon fiber sheet only thickness and materials grade are different, the others are same, all above is only for reference.

Figure 3.37. Carbon fiber category reference (Jinjiuyi, 2020)

Table 3.22. Design parameter of CFRP T300

Parameter	Symbol	Value
Tensile Strength	f_f	3530 MPa
Tensile Modulus	E_f	230000 MPa
Density	ρ	1.76 g/cm^3
Thickness	t_f	4.99 mm
Failure Strain	ε_{fu}	0.014
Poisson's ratio	ν	0.20

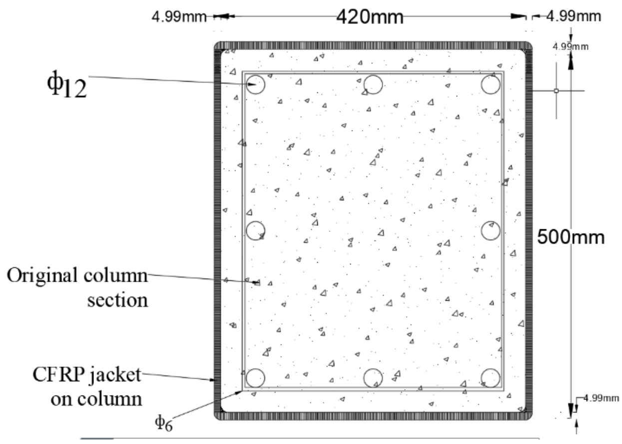


Figure 3.38. The section of column with CFRP jacket (AutoCAD, 2022)

3.5.3.2. Interaction diagram of column with CFRP jacket

The parameters useful for the computation of the points that makes up the interaction diagram were computed using the formulas presented in section 2.3.5.2. The computation of these parameters was done using an excel spreadsheet and the values are presented in table 3.23. The points that make up the interaction diagram depend on the assumption made for the position of the neutral axis of each section. The values of the neutral axis for each point are computed using excel and presented in table 3.24 and the value of the axial and moment that makes up the individual points of the interaction curve are also shown in table 3.25. The interaction curve of the CFRP-concrete section is presented in figure 3.39.

Table 3.23. Parameters used for the computation of the interaction curve points (Excel, 2022)

Parameters	Symbol	Value	Units
Height of concrete section	h	500	mm
Width of the concrete section	b	420	mm
Area of section	Ag	210000	mm^2
Area of concrete	Ac	209095.22	mm^2
Yield strength of bars	fy	347.83	Mpa
Elastic modulus of concrete	Ec	31000	Mpa
Concrete compressive strength	f'c	14.167	Mpa
Area of reinforcement	Ast	904.78	mm^2
Thickness of CFRP	tf	4.99	mm
Number of turns of CFRP	n	1	
Ultimate strain of concrete	ε'c	0.0035	
Effective height	h'	450	mm
Effective width	b'	370	mm
Radius of curvature	r	25	mm
Effective area of concrete	Ae	113133.333	mm^2
Ultimate strain of CFRP	εfu	0.014	
Elastic modulus of CFRP	Ef	230000	Mpa
Compressive strength of CFRP	f1	28.8386835	Mpa
Effective strain of fiber	εfe	0.008204	
Maximum compressive strain of confined concrete	εccu	0.11015894	
Geometric effect A	Ka	0.76681024	
Geometric effect B	Kb	0.59034629	
Maximum compressive strength of confined concrete	f'cc	89.3539131	Mpa
The slope of linear section of confined stress-strain	E2	682.531224	Mpa
The transient strain	ε't	0.00093458	

Table 3.24. The position of the neutral from the top of the mixed cross-section (Excel, 2022)

Point	Value of neutral axis	Unit
B	458	mm
C	449.98	mm
D	438.08	mm
E	41.67	mm

Table 3.25. The points that constitute the individual points of the interaction curve

Point	Parameters		Pn (kN)	Mn (kNm)
A			15880.95	0
B	A	-131.37	8426.4	122.0
	B	1531.32		
	C	-5950.14		
	D	7237427.44		
	E	-98.52		
	F	-11475154.02		
	G	315539.88		
	H	1237629.12		
	I	761092236.8		
C	A	-136.09	7181.9	199.7
	B	1558.61		
	C	-5950.14		
	D	7110797.64		
	E	-102.067		
	F	-11429452.84		
	G	308716.69		
	H	1189909.0		
	I	777494989.9		
D	A	-143.58	4994.2	275.0
	B	1600.95		
	C	-5950.14		
	D	6922905.54		

	E	-107.69		
	F	-11341217.36		
	G	298132.14		
	H	1119102.33		
	I	799880401.5		
	A	-15869.69		
	B	16830.94		
	C	-5950.14		
	D	663888.26		
E	E	-11902.27	0	321.7
	F	1388587022		
	G	-3509365.54		
	H	-1239592.67		
	I	212165474.5		

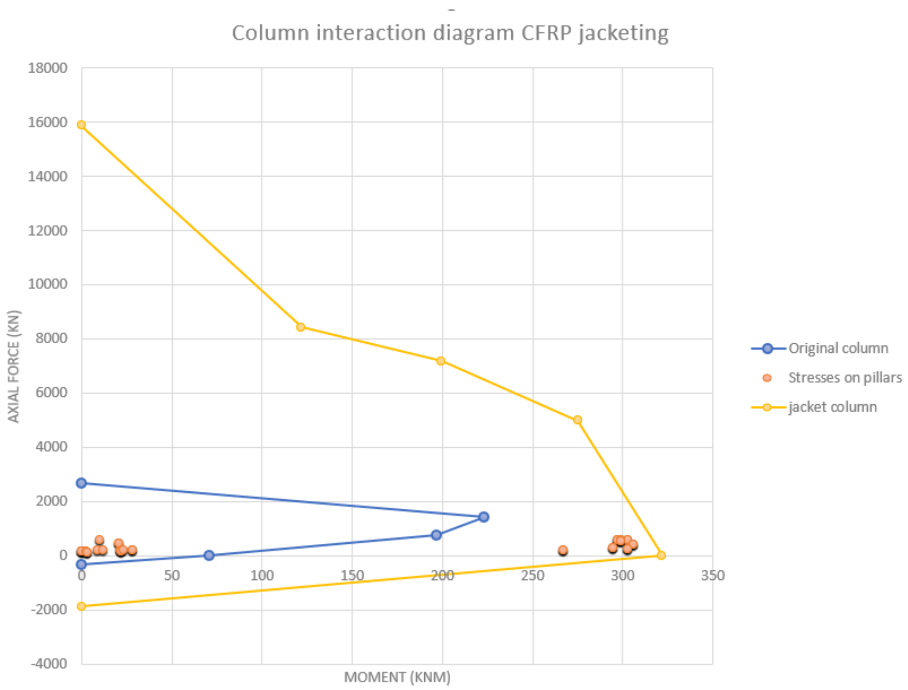


Figure 3.39. Column interaction diagram for CFRP jacket with a plot of the maximum stresses on the pillars (MS Office Excel, 2022)

3.5.3.3. CFRP model of column on Abaqus

The CFRP column jacket modelled with Abaqus is presented in figure 3.40 and figure3.41 respectively. The properties of the CFRP used are those defined previously in section 3.5.3.1.

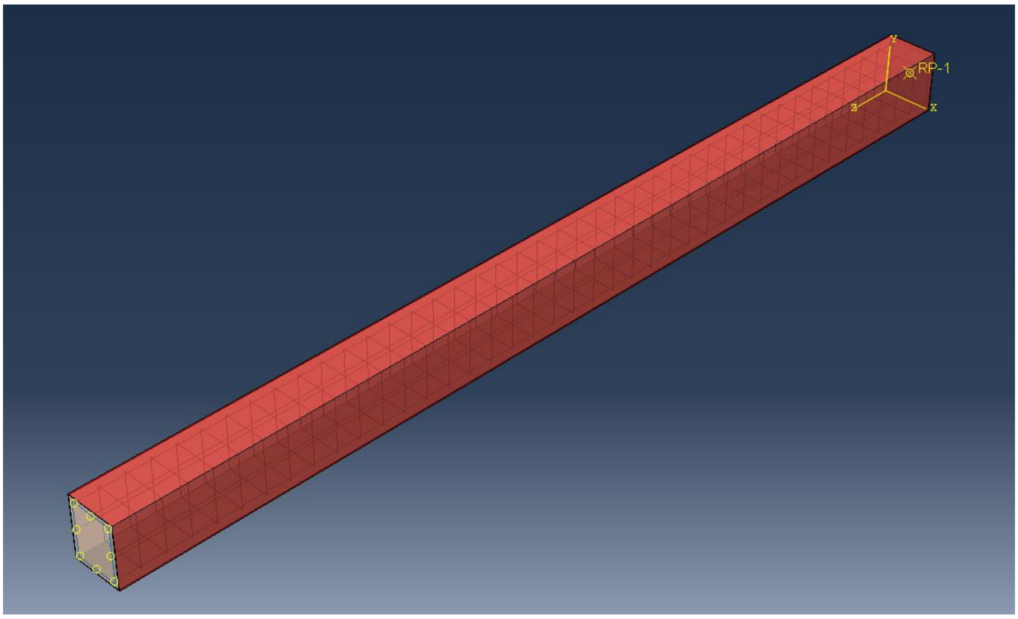


Figure 3.40. Concrete column jacket with CFRP (Abaqus, 2022)

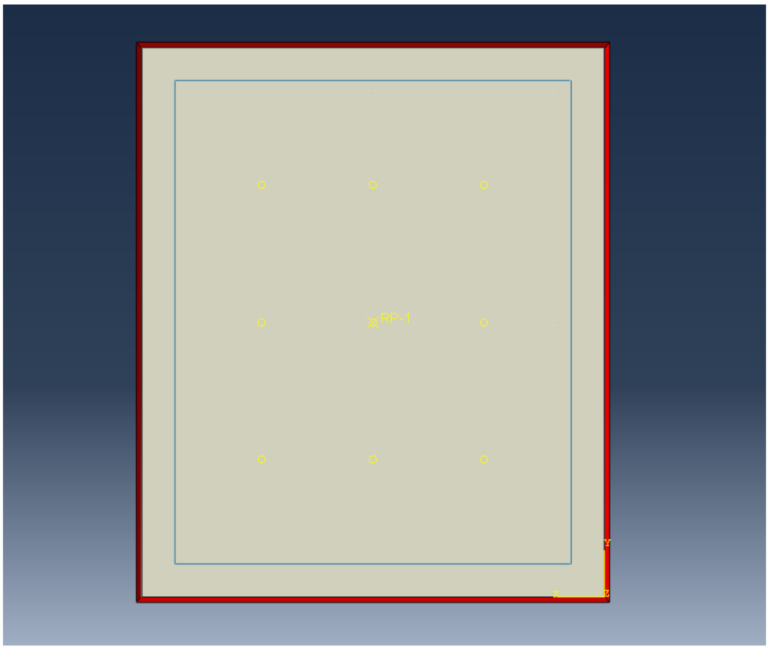


Figure 3.41. Cross section of concrete column jacket with CFRP (Abaqus, 2022)

The mise stress distribution that occurred in the CFRP jacketed column is presented in figure 3.42. The maximum mise stress on the column section with CFRP is 79.40 MPa and this value is inferior to the maximum compressive strength of carbon fiber confined concrete which is 89.35 MPa obtained from table 3.23. Hence, the CFRP jacked column will resist the incoming loads.

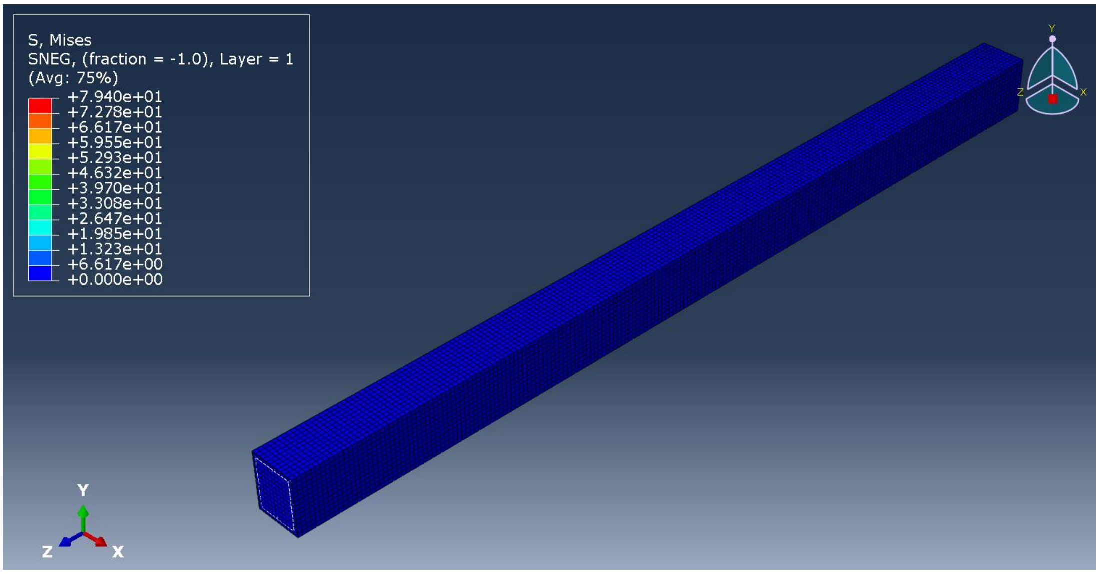


Figure 3.42. Mise stress distribution on CFRP jacketed column section (Abaqus, 2022)

The resultant displacements that occurred due to the loading of the jacketed column are shown in figures 3.43 and 3.44. The maximum displacement on the column is 0.76 mm. Hence the column experiences a very small amount of displacement.

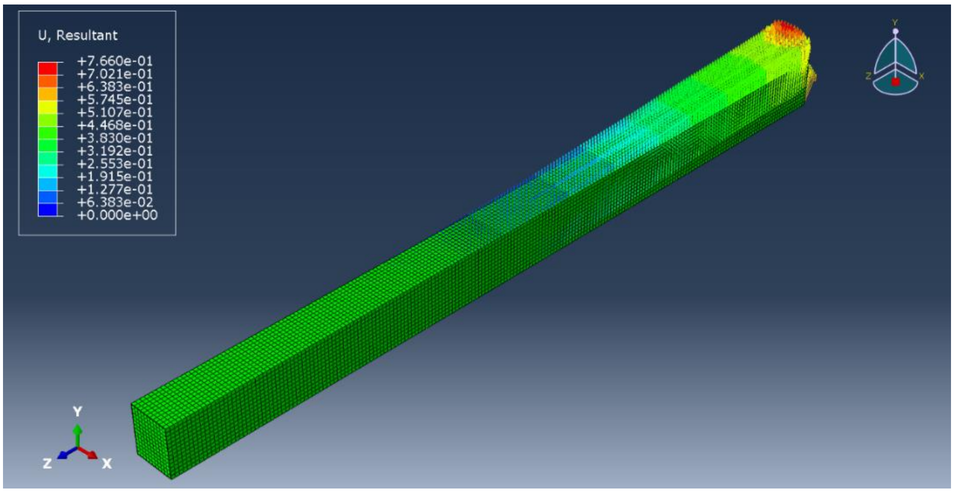


Figure 3.43. Global view of vectors highlights of resultant displacement of the CFRP jacket column (Abaqus, 2022)

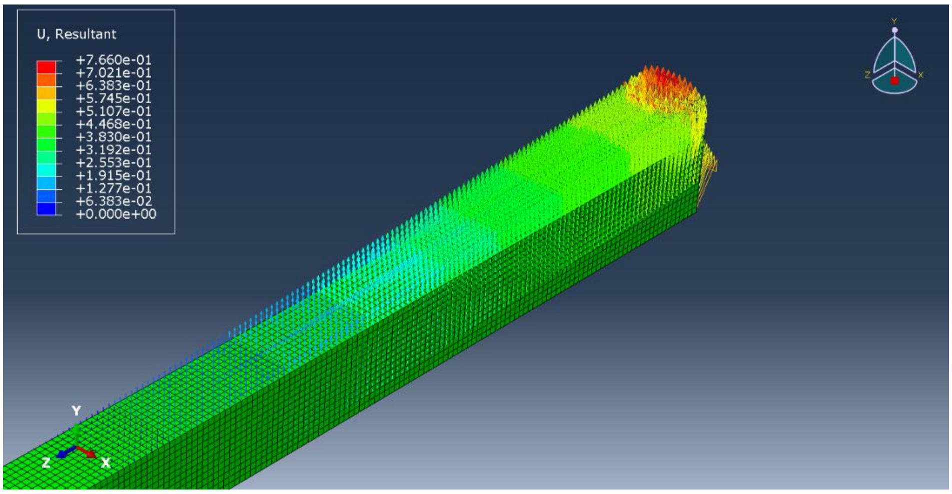


Figure 3.44. Detail view of vectors highlights of resultant displacement of the CFRP jacket column (Abaqus, 2022)

3.6. Results of comparative study

The comparative study involved a study of the behaviour of the strengthen column and also a comparison of the cost for each jacketing method.

3.6.1. The behaviour of the strengthened column

Since the pillar is fixed at the base, the column will have its maximum deflection at the top. The value of the maximum resultant displacement at the top is the same as the maximum deflection the column will experience due to the applied load. The comparison of the mise stress and the deflection on the column considering each jacketing technic is presented in table 3.26. The shaded cell of the table shows the jacketing technic with the greatest value of the comparison parameter. The jacketing methods comparison considering the mise stress and the deflection are presented on a bar chart in figure 3.45 and figure 3.46 respectively.

Table 3.26. The behaviour of the column with different jacketing strengthening

	Concrete jacket	**Steel jacket**	**CFRP jacket**
Mise stress (MPa)	12.24	**211.4**	79.40
Maximum deflection (mm)	16.47	**34.21**	0.76

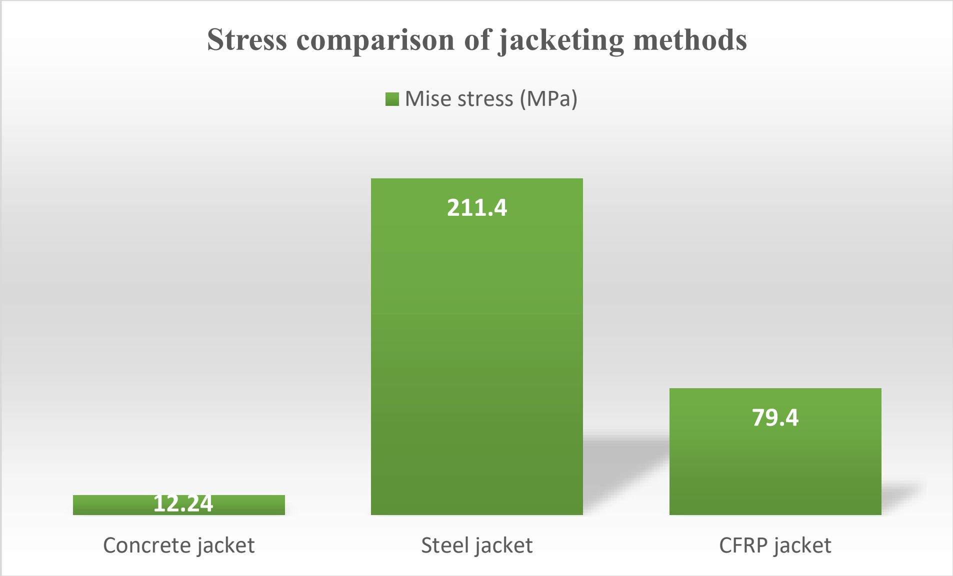


Figure 3.45. Stress comparison of jacketing methods (MS Office Excel, 2022)

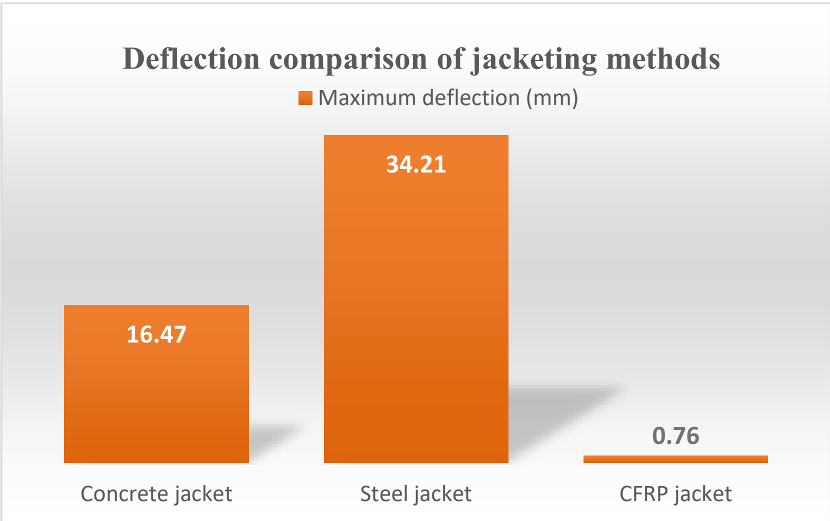


Figure 3.46. Deflection comparison of jacketing methods (MS Office Excel, 2022)

The steel jacket will experience the greatest mise stress and deflection. Even though the steel jacket experiences the greatest amount of stress, it also has a high-stress resistance, in this case, its yield stress is 275 MPa which is still above the experienced stress of 211.4 MPa. The concrete jacketed column experienced the smallest amount of stress. The concrete with characteristic strength of 25MPa can resist twice as much stress as that which was experienced (12.24 MPa) and the concrete bar reinforcement with a yield strength of 400 MPa can resist more than 30 times the experienced stress. Hence it's the recommended method in terms of stress comparison.

Deflection will be an important parameter to be considered especially in an area where the building will be experiencing lateral loads such as those due to earthquakes. In this case, a steel jacket will be less recommended and CFRP will be advantageous since it experiences the smallest amount of deflection.

Hence, a steel jacket though providing a good strengthening of the column will be the less recommended solution in contrast to the other strengthening technics in the cases of deflection and stresses experienced by the column.

3.6.2. Cost comparison result

From the plot of the axial load and the moment 3-3 on an mn interaction diagram in figure 3.16, is noticed that all the pillars with a moment 3-3 greater than 250 kNm were out of the interaction curve and needed strengthening. They were 16 columns in total that needed strengthening. Table 3.27 present the cost analysis of the different strengthening technics. Figure 3.47 presents the cost comparison on a bar chart.

Table 3.27. Cost analysis of the different jacketing technics

Concrete jacketing		**Steel jacketing**		**CFRP jacketing**	
Parameters	**values**	**Parameters**	**Values**	**Parameters**	**Values**
Column section	0.42 m x 0.5 m	Column section	0.42 m x 0.5 m	Column section	0.42 m x 0.5 m
Length of column	8 m	Length of column	8 m	Length of column	8 m
The volume of concrete jacket	1.792 m^3	Volume of steel angle on column	0.041 m^3	Area of contact column surface	14.72 m^2
Price of 1m^3 of concrete	52000 FCFA	Volume of stips on column	0.024 m^3	Cost 1.2 m^2 of fiber	28000 FCFA
Cost of concrete jacket of column	90000 FCFA	Total volume of S275 for strengthening	0.065 m^3	Cost required fiber area	343500 FCFA
Numbers of main bars (ϕ16)	12	Density of steel S275	7850 kg/m^3	Transport fees	79500 FCFA

Length of main bars	96 m	Total mass of jacket	510.25 kg		
Cost of main reforcment for jacket	75500 FCFA	Costs of 1kg of steel S275	560 FCFA		
Number of stirrups (ϕ8) in column	256	Transport fees	20500 FCFA		
Cost of stirrups for jacket	21000 FCFA				
Number of shear connectors	32				
Cost of connectors for jacket	6500 FCFA				
Cost of formwork	44500 FCFA				
Total cost of a pillar jacket	**237500 FCFA**	Total cost of a pillar jacket	**306000 FCFA**	Total cost of a pillar jacket	**423000 FCFA**
Number of pillars for strengthening	16	Number of pillars for strengthening	16	Number of pillars for strengthening	16
Total cost of intervention	**3800000 FCFA**	Total cost of intervention	**4896000 FCFA**	Total cost of intervention	**6768000 FCFA**

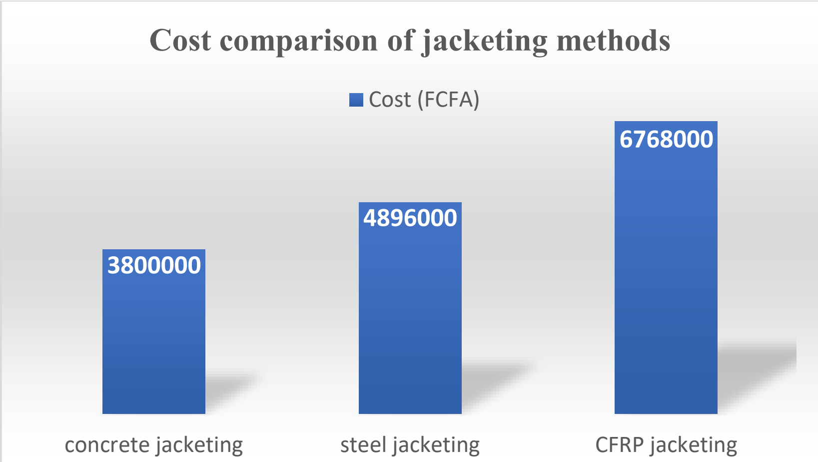


Figure 3.47. Cost comparison of jacketing methods (MS Office Excel, 2022)

The cost comparison results show that the cheapest strengthening method is that with the concrete jacket followed by the steel jacket and lastly the CFRP jacket. Steel jacketing and CFRP jacketing are jacketing methods that need certain expertise and knowledge in the domain. Moreover, CFRP is still a relatively new material and hence not well known by many engineers and technicians. Concert jacketing despite its bulky appearance is chosen because it is the most cost-efficient method with about 43.9 % save on the cost with regards to CFRP. Furthermore, the concrete jacket uses construction technics that are well known by engineers in Cameroon. Also, in terms of stresses, the concrete jacket experienced the smallest amount of stress and ranks second in terms of displacement. So, the recommended strengthening jacketing method for the column of l'EFLC paroisse de Mimboman is concrete jacketing.

Conclusion

This chapter aimed at presenting the results of the methodology used in carrying out the analysis and design of the intervention on a building. The general site presentation showed that the building was located in the residential area of Mimboman in the town of Yaounde. The physical description gave the climate and topography and the climate of the area. The presentation of the data revealed that the initiation of remedial work on the church building was provoked by the drastic collapse of the roof of the building. The results of the structural analysis were also presented and discussed. This led to an assessment of the loads on the pillars of the building. The interaction curve shows that the pillars 16 pillars need reinforcement. The different interventions made on the column were presented. The column with the largest load is used to design the other. The design of the concrete, steel and CFRP jacket was presented and the plot of their respective mn-interaction curves to show that all new design sections resist the incoming loads. The presentation of the model on Abaqus and the behaviour of the column under the effect of the loads were observed. The results of the comparative study showed that the concrete jacketing method is the most cost-efficient strengthening jacketing technic and it experienced the smallest amount of stress.

GENERAL CONCLUSION

This work aimed to carry out an analysis of an existing reinforced concrete building in Cameroon thereby determining the need for an intervention and designing the proper intervention suitable for the building. The building was L'Eglise Fratenelle Lutherienne du Cameroun paroisse de mimboman. After the collapse of the roof, a demand was made in other to increase the capacity of the building by including a mezzanine. This created a need for strengthening the columns of the building. Hence the design of a strengthening intervention was needed. In this study, there different alternative jacketing strengthening of the column was studied notable, including concrete jacket, steel jacket and CFRP jacket.

In other to know the incoming load on the structure, a structural model was belted using the numerical software Sap2000. The pillar experiencing the maximum load was pillar 5 with a moment of 302.74 kNm and an axial force of 572.02 N. This pillar was later on used in verifying the resistance of the original concrete section. This verification was done by plotting an MN-interaction diagram and it was noticed that 16 pillars could not resist the incoming load acting on the building. Hence this validated the need for an intervention in the column section. Steel and CFRP sections were chosen and verification was made later on. The three different jacket strengthening of the new jacket column were verified by plotting an MN-interaction diagram. This proved that the section of the concrete jacket designed and that of steel and CFRP chose were sufficient enough to resist the incoming loads. A more detailed analysis was done on the FEM software Abaqus to view the stresses and displacements in the concrete column. And it was noticed that the steel jacket with a stress of 211.4 MPa and a deflection of 34.21 mm had the greatest amount of stress and deflection, hence it was the least recommended method in terms of behaviour comparison of the jacketing methods. The concrete jacket experienced the smallest amount of stress (12.24 MPa) and ranked second in terms of deflection in a low seismic area like in this case study.

Finally, a cost analysis was made in other to know which of the method will be more cost-efficient in Cameroon. The steel for the jacket and the CFRP were gotten from China and shipped to Cameroon. This analysis revealed that concrete jacketing was the cheapest of all the methods with an implementation cost of 3,800,000 FCFA followed by steel jacket with a cost of 4,896,000 FCFA and lastly CFRP with a cost of 6,768,000 FCFA. Hence using concrete jacketing there is a money-saving of 43.9 % in contrast to the cost of CFRP and similarly, using steel jacketing there is a cost saving of 27.66 % in contrast to CFRP. Hence the recommended jacketing technic for the strengthening of the column was concrete jacketing.

Throughout this study, there were some difficulties in data collection since not all the appropriate data could be gotten from the enterprise. Before this study was done, some of the data such as the results of the assessment of the existing column steel reinforcement strength were not available. That is the reason why in carrying out the concrete jacketing, the existing steel bar strength was neglected.

As a perspective, since they are not clearly defined norms for the computation of the exact amount and appropriate section of steel jacket and CFRP to be used for strengthening of columns, further research could be done in that light thereby improving this work. Research could be carried out to evaluate the performance of other strengthening technics for the building such as the addition of shear walls and steel bracing in contrast to the jacketing technics. Further studies could be done in other to evaluate the strengthening performance of other Fiber reinforced polymers like glass (GFRP) and aramid (AFRP) which could be used as a cheaper replacement for carbon fiber in column jacketing.

BIBLIOGRAPHY

Michel Mbessa, (2005) Traite de genie civil. Volume 1 : Matériaux, Technical note. 2005 Edition. 172p

Mehdi Setareh • Robert Darvas, (2017) Concrete Structures Second Edition, Springer International Publishing Switzerland 2017

Mohamed Chemrouk (2015), The deteriorations of reinforced concrete and the option of high performances reinforced concrete. University of Sciences and Technology Houari Boumediene, Algiers, Algeria. 2015 The Authors. Published by Elsevier.

Shashikant Srivastava DESIGN OF CONCRETE STRUCTURES, Assistant Professor Department of Civil engineering Faculty of Engineering and Technology Rama University Uttar Pradesh, Kanpur(India)

American Concrete Institute(ACI) committee 437, (2019). Strength Evaluation of Existing Concrete Buildings Reported by ACI Committee 437 32(12)

Jagan M. Gudimettla (2016) Resistivity Tests for Concrete—Recent Field Experience, Article in Aci Materials Journal · August 2016

Silvia Santini , Angelo Forte and Lorena Sguerri (2020), Article The Structural Diagnosis of Existing RC Buildings: The Role of Nondestructive Tests in the Case of Low Concrete Strength, Department of Architecture, Roma Tre University, Largo G. Marzi 10, 00153 Rome, Italy. Received: 28 October 2020; Accepted: 6 November 2020; Published: 12 November 2020. Infrastructures 2020, 5, 100 18(1)

Saim Raza, Muhammad K. I. Khan, Scott J. Menegon , Hing-Ho Tsang and John L. Wilson (4 June 2019) Strengthening and Repair of Reinforced Concrete Columns by Jacketing: State-of-the-Art Review

Wewin Inge, Audrey, Sofie Nugroho, Helen Njo (2018), Strengthening method of concrete structure, Wewin Inge et al 2018 IOP Conf. Ser.: Earth Environ. Sci. 126 012051.

Grainger Editorial Staff (2019) Preventive Maintenance Planning for Aging Buildings, 6/17/19.

Shri. Pravin B. Waghmare (2011), MATERIALS AND JACKETING TECHNIQUE FOR RETROFITTING OF STRUCTURES, Research Article, International Journal of Advanced Engineering Research and Studies.

Qasem Khalaf (February 2015), Comparative Study for Strengthening Techniques of RC Beams Using Concrete Jackets and Steel Plates(thesis) Supervised by Prof. Mohamed Ziara, 148p (5-7).

CGTI (2011). 110208 CGTI Fiche EUROCODES AFRICA 2.qxp 8/02/11 14 :10 page 1

Japan Society of Civil Engineers (March 2018), Guidelines for structural intervention of existing concrete structures using cement-based materials, Koichi MAEKAWA, Chairman.

British Standards Institution. (2002). Eurocode 1: Actions on structures : Part 1.2 General actions : actions on structures exposed to fire. BSI.

Noel, E. O. P., Marcelin, M. P., & Bekoa, A. (2014). Crustal Structure and Seismogenic Zone of Cameroon: Integrated Seismic, Geological and Geophysical Data. Open Journal of Earthquake Research, 03(04), 152–161. https://doi.org/10.4236/ojer.2014.34015

Vedprakash C. Marlapalle, P. J. Salunke, N. G. Gore, *(July 2014),* *Analysis & Design of R.C.C. Jacketing for Buildings*

Runsheng, W., & Yang, Z. (2019). Simulation of Stress Distribution and Deformation Analysis for Cantilever I-beams Using ABAQUS Software. *IOP Conference Series: Materials Science and Engineering*, *626*(1). https://doi.org/10.1088/1757-899X/626/1/012010

Brooker, O., Ceng, B., Mistructe, M., Harris, A. J., Harrison, F. T., Ceng, D., Narayanan, R. S., Webster, F. R., & Fistructe, C. (2006). *A J Bond MA MSc DIC PhD MICE CEng How to Design Concrete Structures using Eurocode 2*. www.concretecentre.com

Manie, S., Jami, E., & Azarian, Z. (2017). Simplified design of FRP-confined square RC columns under bi-axial bending. *Buildings*, *7*(3).

Colin R.Gagg, (2014) Cement and concrete as an engineering material: An historic appraisal and case study analysis. Accepted 5 February 2014.

RCM (2006). REINFORCEMENT FOR CONCRETE MATERIALS AND APPLICATIONS ACI Education Bulletin E2-00 (Reapproved 2006). Developed by Committee E-701, Materials for Concrete Construction.

Kimberly Mae Cruz, (2008) Chapter 1: Introduction to Reinforced Introduction to Reinforced Concrete Design

Marlapalle, V. C., Salunke, P. J., & Gore, N. G. (2014). Analysis & Design of FRP Jacketing for Buildings. https://www.researchgate.net/publication/332683608, (31)

Al-Sherrawi, M. H., & Salman, H. M. (2017). Construction of N-M Interaction Diagram for Reinforced Concrete Columns Strengthened with Steel Jackets Using Plastic Stress Distribution Method. *Civil Engineering Journal*, *3*(10), 929. https://doi.org/10.28991/cej-030926

Shallan, O., Sakr, T., Khater, M., & Ismail, A. (2022). Interaction diagram for RC column strengthened by steel angles and strips. Frattura Ed Integrita Strutturale, 16(60), 1–12. https://doi.org/10.3221/IGF-ESIS.60.01

Hafezolghorani, M., Hejazi, F., Vaghei, R., Jaafar, M. S. bin, & Karimzade, K. (2017). Simplified damage plasticity model for concrete. *Structural Engineering International*, *27*(1), 68–78. https://doi.org/10.2749/101686616X1081

WEBOGRAPHY

CF2018manual.pdf.http://2j123n2urkl31z24fo1mojn1.wpengine.netdnacdn.com/wpcontent/uploads/2014/08/CF2018manual.pdf (27/01/2022, 6:02 pm)

Britannica, (2020). The Editors of Encyclopaedia. "reinforced concrete". Encyclopedia Britannica, 7 Feb. 2020, https://www.britannica.com/technology/reinforced-concrete. Accessed 13 April 2022

Action wall Ties LTD (2022) https://actionwallties.co.uk/customer-stories/rebuild-reinforce-collapsed-bay-window-in-sw-london (2/5/2022).

Jinjiuyi (2020). Published on 19 july 2020. https://www.jinjiuyi.net/news/difference-between-t300-t700-and-t800-carbon-fiber.html

Boonedam **(2019)** https://www.boonedam.com/focus-areas. (26/07/2022, 6:24 pm)

https://shyamsteel.com/blogs/different-type-of-steel-reinforcement-bars/#:~:text=In%20RCC%20(Reinforced%20Concrete%20Structure,in%20pounds%20per%20square%20inch). (08/02/2022, 9:30 am)

https://theconstructor.org/concrete/reinforced-concrete-uses-benefits-advantages/35976/. (10/02/2022, 2:02 pm)

https://theconstructor.org/concrete/construction-defects-concrete-structures/8472/ . (13/02/2022, 9:06 am)

https://theconstructor.org/structural-engg/slab-beam-column-footing-construction/24934/ (20/2/2022,10:49 am)

ANNEX

Table A.1. Recommended values of factors for buildings (EN1990, 2002)

Action	0	1	2
Imposed loads in buildings, category (see EN 1991-1-1)			
Category A: domestic, residential areas	0,7	0,5	0,3
Category B: office areas	0,7	0,5	0,3
Category C: congregation areas	0,7	0,7	0,6
Category D: shopping areas	0,7	0,7	0,6
Category E: storage areas	1,0	0,9	0,8
Category F: traffic area	0,7	0,7	0,6
Vehicle weight 30kN	0,7	0,5	0,3
Category G: traffic area, 30kN < vehicle weight 160kN Category H: roofs	0	0	0
Snow loads on buildings (see EN 1991-1-3)* Finland, Iceland, Norway, Sweden Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l. Remainder of CEN Member States, for sites located at altitude H 1000 m a.s.l.	0,70 0,70 0,50	0,50 0,50 0,20	0,20 0,20 0
Wind loads on buildings (see EN 1991-1-4)	0,6	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0
NOTE The values may be set by the National annex. * For countries not mentioned below, see relevant local conditions.			

Table A.2. Design values of actions (EN1990, 2002)

Persistent and transient design situations	**Permanent actions**	**Leading variable action (*)**	**Accompanying variable actions**	
Unfavourable	Favourable	Main (if any)	Others	
(Eq. 2.14)	Gj,sup*G*kj,sup	Gj,inf*G*kj,inf	Q,1 Qk,1	Q,i0,i*Q*k,i

(*) Variable actions are those considered in Table A1.1

NOTE 1 The values may be set by the National annex. The recommended set of values for are :

Gj,sup = 1,10

Gj,inf = 0,90

Q,1 = 1,50 where unfavourable (0 where favourable)

Q,i = 1,50 where unfavourable (0 where favourable)

NOTE 2 In cases where the verification of static equilibrium also involves the resistance of structural members, as an alternative to two separate verifications based on Tables A1.2(A) and A1.2(B), a combined verification, based on Table A1.2(A), may be adopted, if allowed by the National annex, with

the following set of recommended values. The recommended values may be altered by the National annex.

Gj,sup = 1,35

Gj,inf = 1,15

Q,1 = 1,50 where unfavourable (0 where favourable)

Q,i = 1,50 where unfavourable (0 where favourable)

provided that applying Gj,inf = 1,00 both to the favourable part and to the unfavourable part of permanent

actions does not give a more unfavourable effect.

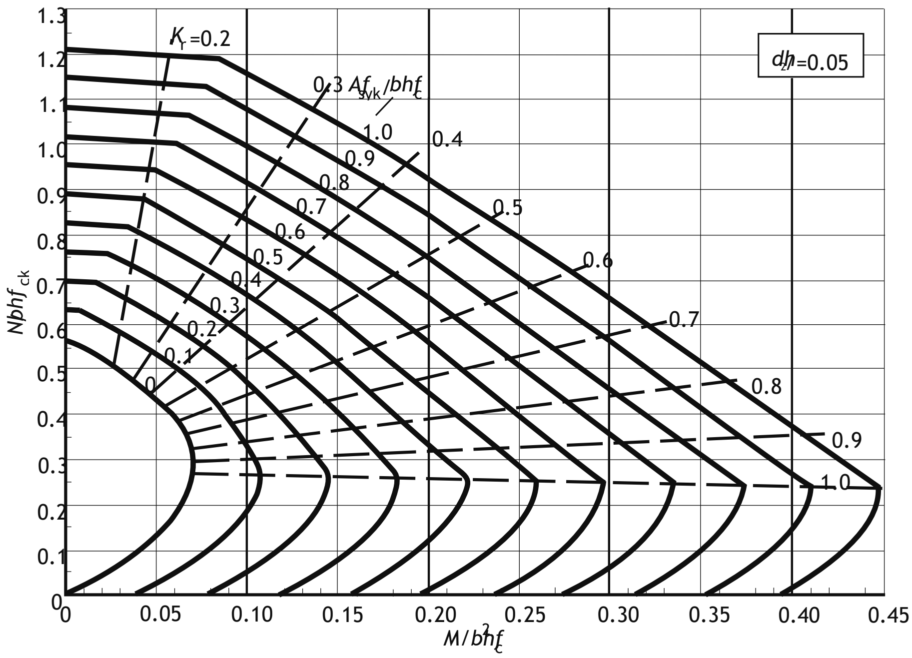


Figure A.1. Column design interaction diagram for rectangular columns d2 /h = 0.05 (Brooker et al., 2006)

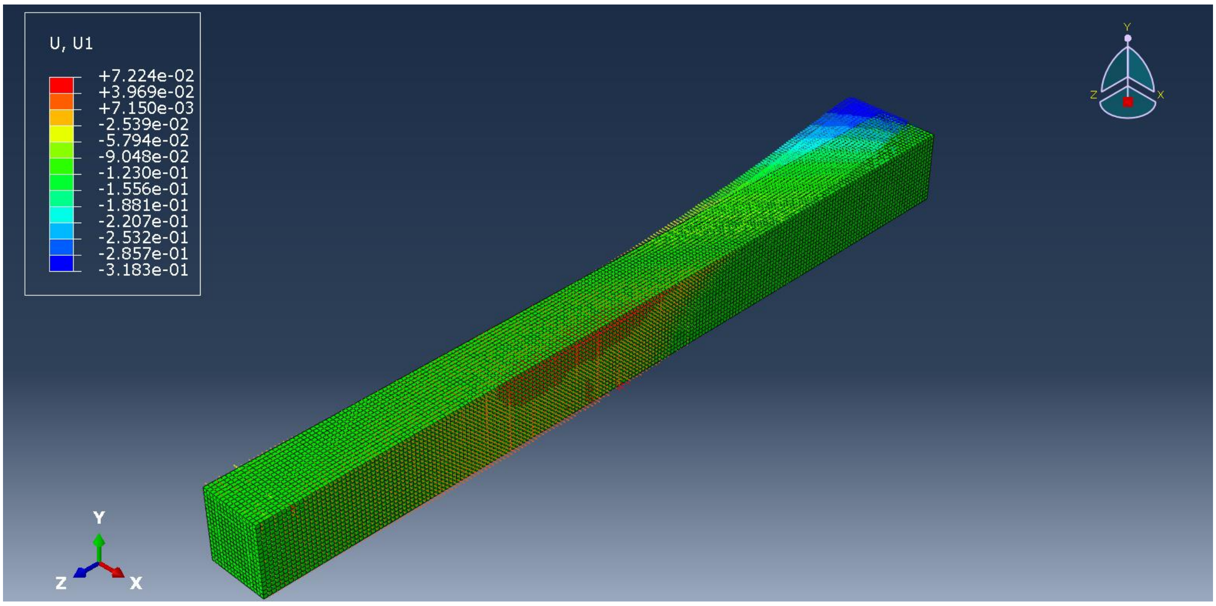


Figure A.2. Displacement in the x-direction the of concrete jacketed column (Abaqus, 2022)

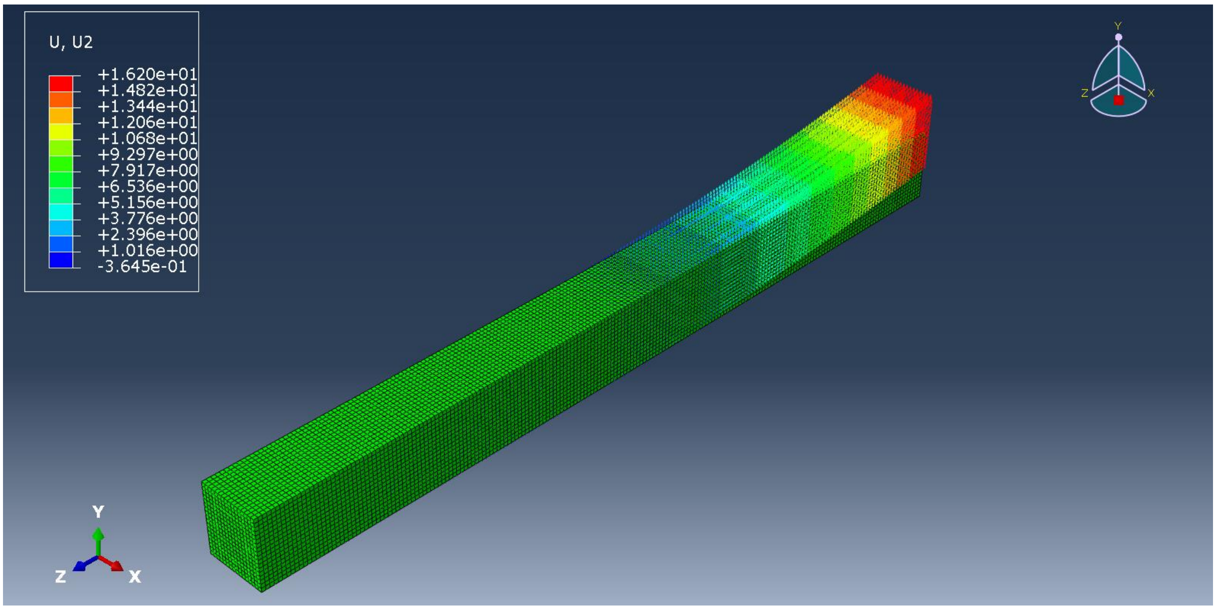


Figure A.3. Displacement in the y-direction the of concrete jacketed column (Abaqus, 2022)

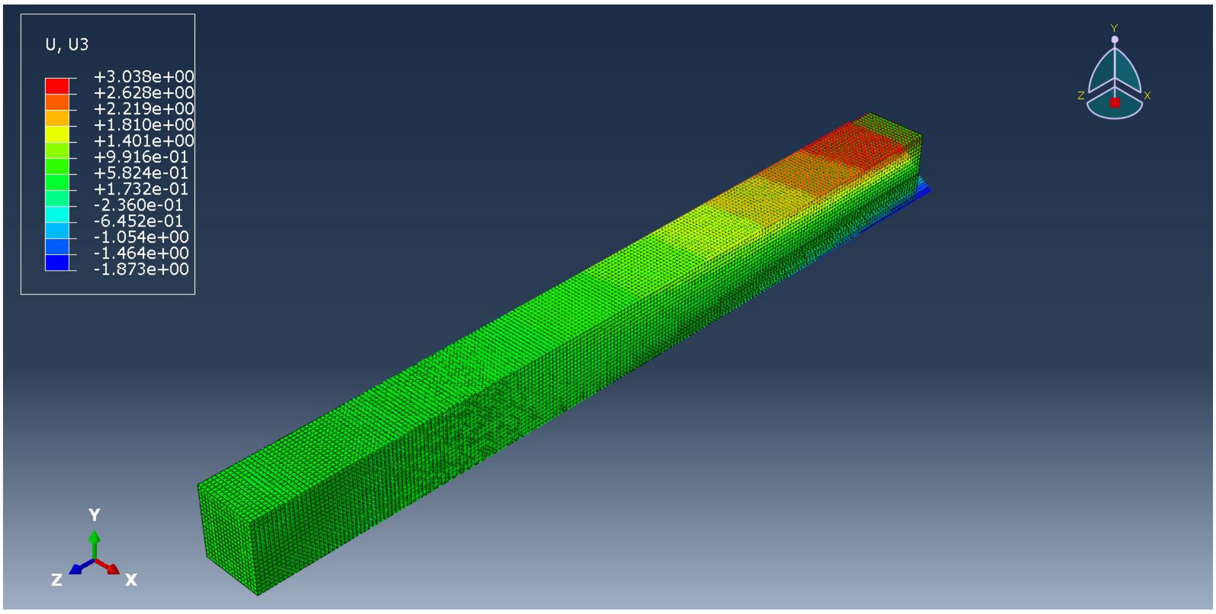


Figure A.4. Displacement in the z-direction the of concrete jacketed column (Abaqus, 2022)

Figure A.5. The collapse of the roof of EFLC Mimboman (03/08/2020, 9:45 am)