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FINITE ELEMENT METHOD ANALYSIS APPLIED TO THE STUDY OF THE DUCTILITY OF JOINTS IN REINFORCED CONCRETE STRUCTURES. CASE STUDY: BLOCK H BUILDING NASPW

A thesis submitted in partial fulfilment of the requirements for the degree

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DEDICATION

This work is dedicated to the

NCHINDUM's family.

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GLOSSARY

LIST OF ABBREVIATIONS

ACI	American Concrete Institute
ASCE	American Society of Civil Engine
BS	British Standard
DCH	Ductility Class High
DCM	Ductility Class Medium
DCL	Ductility Class Low
DMRF	Ductile Moment Resisting Frame
EC	Eurocode
EN	European Nations
FE	Finite Element
FEA	Finite Element Analysis
FEMA	Federal Emergency Agency
IS	Indian Standard
NASPW	National Advanced School of Public Works
NZS	New Zealand Standard
RC	Reinforced Concrete
SMRF	Special Moment Resisting Frame
SAP	Structural Analysis Program
SLS	Service Limit State
ULS	Ultimate Limit State

LIST OF SYMBOLS

A	Area of the cross section
A_c	Area of the concrete cross section
A_{min}	Minimum section area
A_{net}	Net area of the cross section
A_s	Area of the steel reinforcement section
A_{sw}	Cross sectional area of the shear reinforcement
A_{jv}	Area of vertical shear reinforcement
A_g	Gross area of concrete

A_{st}	Total are of stirrups
A_{jh}	Area of horizontal shear reinforcements
A_{sh}	Area of horizontal steel reinforcement
A_{s1}	Area of steel at the top of beam section
A_{s2}	Area of steel section at the bottom of beam section
b	Width of bottom flange of beam
b_c	Width of column
b_w	Width of the beam
$C_{min,b}$	Minimum cover due to bond requirement
$C_{min,dur}$	Minimum cover due to environmental conditions
C_{min}	Minimum concrete cover
d	Effective depth section
d_t	Concrete tensile damage parameter
d_c	Concrete compressive damage parameter
f_{ywd}	Design yield strength of stirrups
f_{yv}	Yield strength of vertical shear reinforcements
f_v	Unconfined compressive strength of concrete
f_h	Horizontal confinement effect due to steel hoops
f_{yh}	Yield stress of horizontal shear reinforcements
f_h	Horizontal confinement effect due to steel hoops
f_c'	Unconfined compressive strength of concrete
f_{sy}	Yield strength of stirrup
f_{cd}	Design value of concrete compressive strength
f_{ctm}	mean value of tensile strength of concrete
f_{yd}	Design value of yield strength of steel
G	Shear Modulus
G_{1k}	Structural load of the building
G_{2k}	Non-structural load applied on the building
H	Total height of the building
I	Moment of inertia
h	Storey height
h_c	cross sectional depth of the column
M_{Ed}	Bending moment at support

M_{Rd}	Resisting moment
M_{Sd}	Soliciting bending moment
k	Concrete damage parameter
V_{jhd}	Design horizontal shear force
M_u	Ultimate beam moment
V_{jh}^{Rd}	Horizontal joint shear capacity
V_{jh}^{u}	Maximum induced horizontal shear force in the joint
N_{Ed}	Design axial compression force
$N_{Pl,Rd}$	Design tension resistance of a steel section
N_{Rd}	Minimum area section of the column
N_{Sd}	Axial load compute using the recovery area of the column
$N_{u,Rd}$	Designed ultimate resistance
$S_{cl,max}$	Minimum spacing of the transverse reinforcement
V_{Ed}	Acting shear
$V_{Rd,C}$	Design shear resistance of the member without shear reinforcement
c	Concrete cover
$s_{l,max}$	Maximum longitudinal spacing
$s_{t,max}$	Maximum transversal spacing
x	Neutral Axis Position
α	Angle between shear reinforcement and the beam axis perpendicular to the shear force
α_{cw}	Coefficient taking account the state of stress in the compression cord
$\Delta C_{dur,add}$	Add reduction of minimum cover for use of additional protection
$\Delta C_{dur,st}$	Reduction of minimum cover for use of stainless steel
$\Delta C_{dur,\gamma}$	Additive safety element
$\emptyset_{l,min}$	Minimum diameter of the longitudinal bars
φ_{ef}	Effective creep ratio
γ	Specific weight
γ_c	Partial factor for concrete
γ_s	Partial safety factor for steel
Ψ_E	Combination coefficient for variable action
λ	Slenderness
λ_{lim}	Limit value of slenderness

ν	Poisson's ratio
V_j	Joint horizontal shear force
ν_n	Nominal joint shear stress
τ_c	Shear stress in concrete

ABSTRACT

The main objective of this work was to determine the extent to which a beam-column joint can withstand deformation in response to static loading. That is, to determine the ductility of the beam-column joint. To achieve this goal, a literature review was carried out to highlight the different ductility relationships and how each of them can be determine. The methodology adopted consisted first in a site recognition and the collection of the geometric data of the building. After, the case study which is a three-storey building was modelled using SAP 2000 (Structural Analysis Program) version 22 under static loading condition. Horizontal and vertical structural elements were designed according to Eurocode 2 to satisfy the safety requirements and the intersecting point of the designed horizontal and vertical elements was taken as the joint under study. Nonlinear finite element analysis (FEA) of the reinforced concrete beam-column joint was performed in order to investigate the applicability of the FEM analysis to study the mechanical behaviour in terms of stresses and cracking. The analysis was performed on the interior joint under static loading where the longer span of the principal beam was given a certain displacement. A 3D finite element model capable of approximately modelling the concrete stress-strain behaviour, tensile cracking and compressive damage of concrete and indirect modelling of concrete-reinforcement bond was used. In order to stimulate the nonlinear behaviour of the concrete material, the concrete damage plasticity was applied to the numerical model as a distributed plasticity over the whole geometry. The results of the reaction forces at the column base and the displacement along the principal beam were gotten from the stimulation and were plotted in order to determine the ductility of the joint under study.

Key words: Finite Element Method, Ductility, Joints.

RÉSUMÉ

L'objectif principal de ce travail était de déterminer dans quelle mesure un joint poutre-poteau peut résister à la déformation selon la charge statique appliquée. En d'autres termes, il s'agit de déterminer la ductilité de l'assemblage poutre-poteau. Pour atteindre cet objectif, une revue de la littérature a été effectuée afin de mettre en évidence les différentes relations de ductilité et la façon dont chacune d'entre elles est utilisée. La méthodologie adoptée consistait d'abord en une reconnaissance du site et la collecte des données géométriques du bâtiment. Ensuite, le cas d'étude, un bâtiment de trois étages, a été modélisée à l'aide du logiciel SAP 2000 (Structural Analysis Program), version 22, dans des conditions de chargement statique. Les éléments structurels horizontaux et verticaux ont été conçus conformément à l'Eurocode 2 pour repondre aux exigences de sécurité et le point d'intersection des éléments horizontaux et verticaux conçus a été considéré comme le joint dans l'étude. L'analyse non-linéaire des éléments finis (AEF) du joint poutre-poteau en béton armé a été réalisée afin d'étudier l'applicabilité de la MEF pour étudier le comportement mécanique en termes de contraintes et de fissures. L'analyse a été effectuée sur le joint intérieur sous chargement statique où la plus grande portée de la poutre principale a reçu un certain déplacement. Un modèle d'éléments finis 3D capable de modéliser approximativement le comportement de contrainte-déformation du béton, la fissuration en traction et l'endommagement en compression du béton et de modéliser indirectement la liaison béton-armature. Afin de stimuler le comportement non linéaire du matériau en béton, la plasticité du béton endommagé a été appliquée au modèle numérique comme une plasticité distributive sur toute la géométrie. Les résultats des forces de réaction à la base du poteau et le déplacement le long de la poutre principale ont été obtenus à partir de la stimulation et ont été tracés afin de déterminer la ductilité de l'assemblage étudié.

Mots clés : Méthode des éléments finis, Ductilité, Nœuds.

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GENERAL INTRODUCTION

Existing reinforced concrete (RC) structures that were designed according to pre-1970's codes often constructive deficiencies, which not only results in deficient lateral load resistance, but also in insufficient energy dissipation, rapid strength deterioration and improper hinging mechanisms during lateral actions such as earthquakes most especially in the joints, leading to excessive drifts and ultimately to structural collapse.

Non-ductile detailing is generally manifested through deficient joint shear resistance, deficient column shear capacity, deficient column's main reinforcement lap splices, deficient anchorage of beam positive reinforcement at the beam-column joint, and deficient beam shear resistance.

In particular, recent lateral actions in the world most especially in Cameroon have demonstrated that RC beam-column joints that have been constructed based on pre-1970's design codes may initiate and cause total collapse of structures. Hence, understanding the response of beam-column joints in reinforced concrete building frames during loading is crucial to the development of an overall efficient and safe structure.

Numerous numerical studies have been carried out on RC joints using FEM analysis by Masi et al. (2009), Mahini and Ronagh (2006) with the help of FEM softwares such as ANSYS, ABAQUS and DIANA which were all concentrated on simulating flexural behaviour of beams columns adjacent to the joint region and not focused on the ductile behaviour of RC connections, while shear strength and ductile behaviour of joints control the overall response of RC beam-column joints subjected to lateral actions.

This study intends to investigate the extent to which a structure can resist deformation prior to collapse when subjected to static loading with the help of finite element method. To attain this objective, the work is divided in three chapters. The first chapter is about the state of the art and will permit to master the basic concepts related to ductility, joints and finite element analysis. The second chapter entitled methodology will present the steps adopted to achieve the objective of this work. Finally, at chapter three the results of the static design and finite element analysis model will be presented and conclusion made.

CHAPTER 1: LITERATURE REVIEW

Introduction

Ductility being an essential property of structures responding inelastically during severe shaking and joints being the most critical regions in reinforced concrete frames designed for inelastic response to severe seismic attack. It is important to analyse the ductile behaviour of joints numerically through finite element method in order to understand the extent to which damage can occur under severe attack like seismic action or wind action on tall buildings before construction. This chapter aims at outlining the finite element method used, giving an in-depth review on ductility and joints, and also the different component constituting reinforced concrete, with the different design method in reinforced concrete buildings

1.1. Reinforced concrete

Reinforced concrete is a strong durable building material which can be formed into many varied shapes and sizes, ranging from a simple rectangular beam or column to a slender curved dome or shell. Its utility and versatility are achieved by combining the best features of concrete and steel. In other words, reinforced concrete is a composite material of steel bars embedded in a hardened concrete matrix. Concrete assisted by steel, carries the compressive forces while steel resists tensile forces. Concrete itself is a composite material which consist of dry mix cement, coarse and fine aggregates. Water is added and reacts with the cement which hardens and binds the aggregate into the concrete matrix. The concrete matrix sticks or bonds onto the reinforcing bars. (Prab et al, 2014)

1.1.1. Concrete materials

1.1.1.1. Cement

Cement is a binder, a substance used for construction that sets, hardens and adheres to other materials to bind them together (Draeger 2020). It is the main constituent of the concrete.

1.1.1.2. Water

Water is the key ingredient, which when mixed with cement, forms a paste that binds the aggregate together. The water causes the hardening of concrete through hydration. The role of water is important because the water to cement ratio is the most critical factor in the production of "perfect" concrete. Water also helps to obtain an appropriate consistency of concrete or mortar; it is also essential for the curing of concrete or mortar in the process of hardening (Kokoszka, 2019).

1.1.1.3. Aggregates

Aggregates are mixtures of various sizes of stone or rock particles in contact with each other. They are typically combinations of gravel and crushed granite, but may also include blast furnace slag, or recycled concrete fragments. Particles with a diameter greater than 4.75 mm are usually classified as coarse aggregate, while smaller particles are called fine aggregate (McNally, 1998). They can either be naturally occurring or recycled aggregates.

1.1.1.4. Admixtures

Admixtures are added to a mixture of cement, water and aggregates in small quantities to increase the concrete durability, fix the concrete behaviour and control the setting or hardening. They come in to improve deficiencies during concrete formulation so that the concrete obtained is of good quality. They are of different types, depending on the purpose of their use. The effectiveness of an admixture depends on the type and amount of cement, water content, mixing time, slump and temperatures of concrete and air.

1.1.2. Properties of concrete

Concrete is a composite material which exists in two phases. The matrix made up of the cement paste and the other phase made up of coarse and fine aggregates. The quality and quantity of these concrete products determine its properties in both the fresh and hardened state. For instance, the water cement ratio in concrete, the curing age, and aggregate quality are some of the few elements that influence the properties of concrete in its hardened state. The strength and properties of concrete also depend on the period of curing.

1.1.2.1. Stress-strain relationship in compression

Stress-strain relations are characterized by a design strength and ultimate strain. Concrete design compressive strength f_{cd} is evaluated from the cylindrical characteristic compressive strength by equation 1.1.

$$f_{cd} = \frac{\alpha_{cc} f_{ck}}{\gamma_c} \qquad \text{(Eq1.1)}$$

Where α_{cc} accounts for the duration of load application and it is generally assumed as 0.85, the safety factor coefficient γ_c is equal to 1.5.

The maximum strain ε_{cu} is generally taken to be 3.5% which represent a good estimation of non-confined concrete that is low stirrups reinforcement ratio. The reference stress-strain relation is described by a parabola-rectangular design relation as shown in figure. 1.1

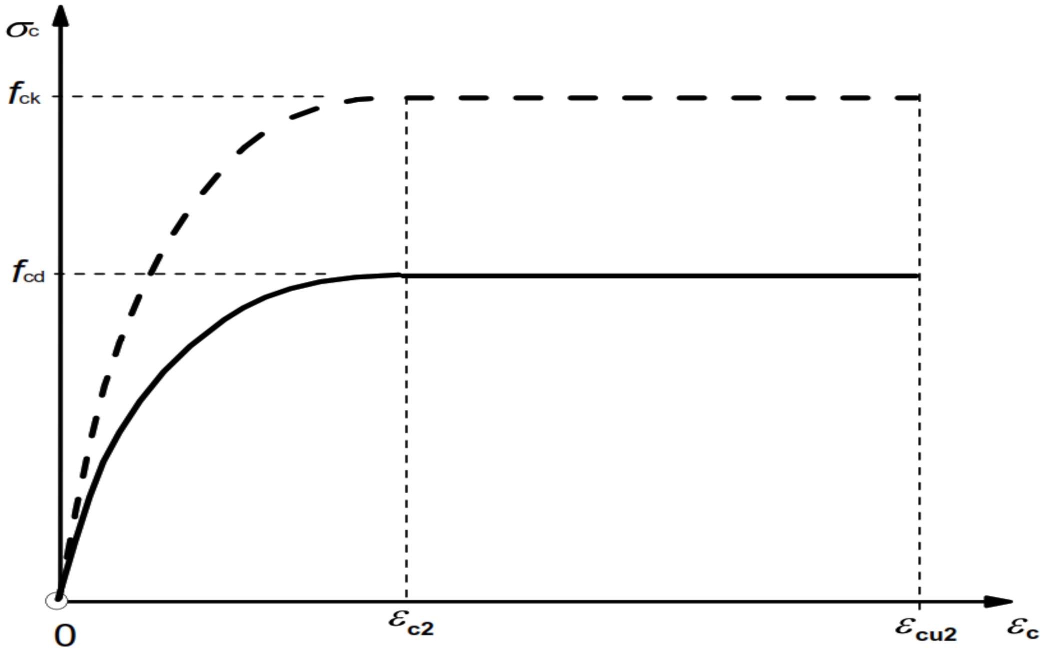


Figure 1. 1. Parabola-rectangular stress-strain relation. (Eurocode 2)

1.1.2.2. Workability

The American Concrete Institute (ACI) defines workability as the property of freshly mixed concrete or mortar which determines the ease and homogeneity with which it can be mixed, placed, consolidated and finished. According to ASTM C 125-93, workability is the property determining the effort required to manipulate a freshly mixed quantity of concrete with minimum loss of homogeneity.

There is no test which can measure the workability of concrete directly. However, there are tests used to determine the workability of fresh concrete through empirical methods such as the flow table test, the ball penetration test and the slump test. The one which is most often used is the slump test because it is easily done. Table 1.1 shows the concrete consistency defined according to the slump while Figure 1.2 shows the slump test

Table 1. 1. Concrete consistency level according to UNI EN - 260

Consistency level	Class	Slump (mm)
Dry	S1	10-40
Plastic	S2	50-90
Quasi-fluid	S3	100-150
Fluid	S4	160-210
Super-fluid	S5	>220

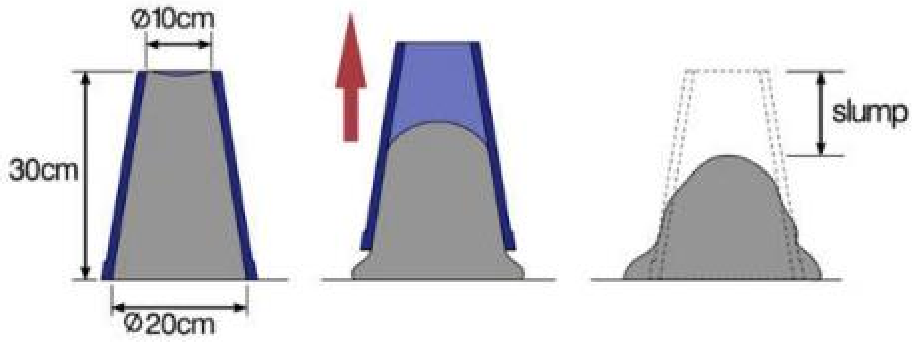


Figure 1. 2. Slump test (Jaime Gonzalez, 2018)

1.1.2.3. Mechanical resistance

Concrete suitability for use depends on its ability to carry loads and accept stresses up to certain values. Hence its formulation and curing must be done under controlled conditions to ensure that the concrete has the required resistance characteristics. The resistances can be measured after period of three days, seven days, twenty-eight days or even ninety-one days depending on what the concrete will be used for.

1.1.2.4. Compressive strength

The compressive strength is the ability to carry loads on its surface without any crack or deflection. Concrete works more in compression than in tension. Research has shown that the compressive strength of concrete is approximately ten times the tensile strength. For Normal Strength Concrete (NSC), the compressive strength varies from 20MPa to 40MPa whereas a compressive strength greater than 60MPa is High Performance concrete (HPC). Concretes having compressive strengths greater than 90MPa are called Ultra High-Performance Concretes (UHPC) (Liew et al, 2015).

1.1.2.5. Tensile strength

This refers to the ability of concrete to resist loads in tension. Concrete being a brittle material, breaks under the influence of loads it cannot carry. Concrete resists less in tension and this type of failure mechanism is avoided as much as possible because it occurs without prior notice.

1.1.2.6. Modulus of elasticity

Modulus of Elasticity of Concrete can be defined as the slope of the line drawn from stress of zero to a compressive stress of $0.45f'_c$. As concrete is a heterogeneous material, the strength of concrete is dependent on the relative proportion and modulus of elasticity of the aggregate.

1.1.2.7. Creep and relaxation

Creep is the concrete tendency to move slowly and to deform permanently under the influence of prolonged stresses. It results as a long-term exposure to high levels of stresses which are below the yield strength of the material. This phenomenon is sensitive to heat and it increases linearly with temperature. The rate of deformation is a function of the material properties, exposure temperature and the applied structural load.

On the contrary, relaxation is referred to concrete tendency to have a decrease in the strength under the influence of a constant strain that is below the strain corresponding to the yield strength of the material as shown in figure1.3.

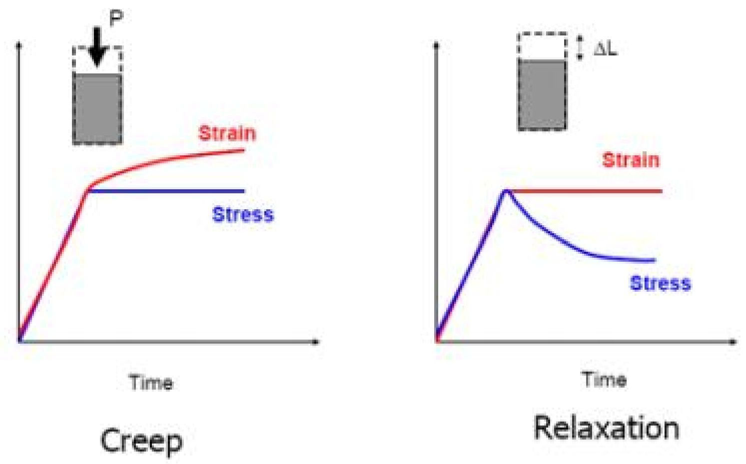


Figure 1. 3. Creep and relaxation stress-strain diagrams (D'Antino et al, 2016)

1.1.2.8. Shrinkage

Shrinkage is a phenomenon which appears in concrete as a result of water expiration from the concrete to the surrounding air, which leads to crack formation. The more rapid is the concrete mixture drying process, the more likely it is that shrinkage cracks will develop. Total shrinkage strain is composed by drying shrinkage and autogenous shrinkage. Drying shrinkage strain is a slow process while autogenous shrinkage develops in conservative systems, where no moisture movement to and from the paste is permitted that is, related to humidity exchanges. The total shrinkage is obtained by applying the relation in equation 2

$$\varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{ca} \quad \text{(Eq.1.2)}$$

1.1.2.9. Durability

Concrete permeability makes it susceptible to chemical attacks from foreign substances in its surrounding. For the concrete to therefore last till its nominal life, its composition has to be considered according to the environment in which it will be used.

1.1.3. Use of concrete

Concrete is very economical and sustainable for construction projects and it is used in the realization of Road pavements, Architectural structures, foundations, roads and bridges, walls and footings for gates, fences and poles, Mid-rise and high-rise buildings.

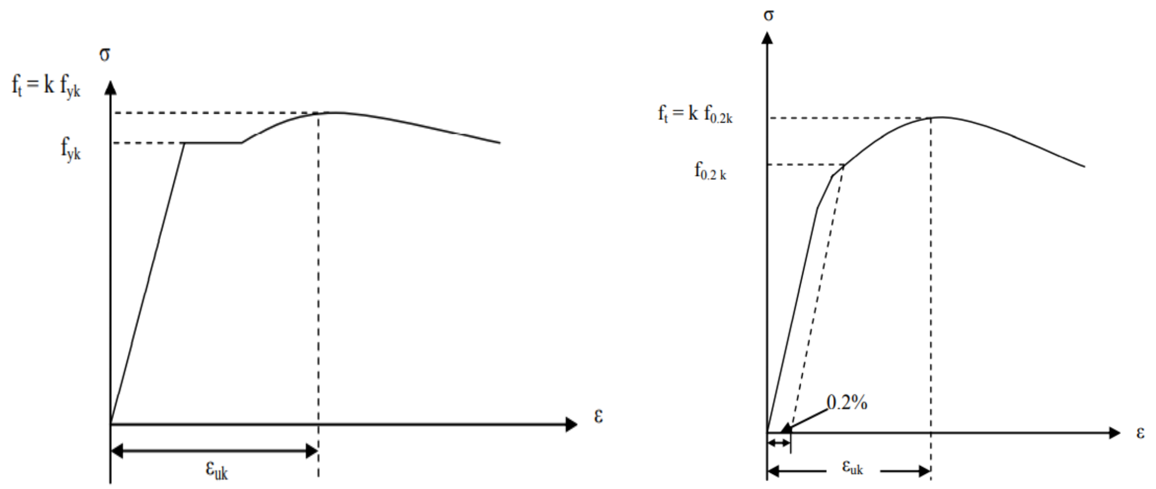
1.1.4. Reinforcement

Reinforcing bars are produced as hot rolled or cold worked high yield steel bars. They have characteristic yield strength f_{yk} of 400 to 600 MPa under 3 classes A, B and C representing low ductility class, normal ductility class and high ductility class respectively as shown on table 1.2. The steel fabric is made from cold drawn steel wires welded to form a mesh. High yield bars are produced as deformed bars with transverse ribs to improve bond with concrete.

Figure 1.4 shows the stress–strain curves for reinforcing bars. Hot rolled bars have a definite yield point. A defined proof stress at a strain of 0.2% is recorded for the cold worked bars. The value of Young's modulus Es for steel is 200GPa and that of the mean density is 7850kg/m^3. The maximum breaking stress is k times the characteristic stress fyk. The design stress fyd = fyk/γs, where γs = 1.15. According to Eurocode 2, the main parameters that define reinforcing steel behavior are yield strength (f_{yk} or $f_{0,2k}$), maximum actual yield strength ($f_{y,max}$), tensile strength (f_t), ductility (ε_{uk} and f_t/f_{yk}), bendability, bond characteristics, section size and tolerances.

Table 1. 2. Properties of reinforcing bars

Product form	Bars and de-coiled rods			Wire fabrics		
Class	A	B	C	A	B	C
f_{yk} or $f_{0.2k}$	400-600 MPa					
k, minimum	≥ 1.05	≥ 1.08	≥ 1.15 < 1.35	≥ 1.05	≥ 1.08	≥ 1.15 < 1.35
$\varepsilon_{uk} \times 10^2$	≥ 2.5	≥ 5.0	≥ 7.5	≥ 2.5	≥ 5.0	≥ 7.5



(a) (b)

Figure 1. 4. Stress-strain curve for (a) hot rolled steel reinforcing bars and (b) cold worked steel reinforcing bars. (Eurocode 2)

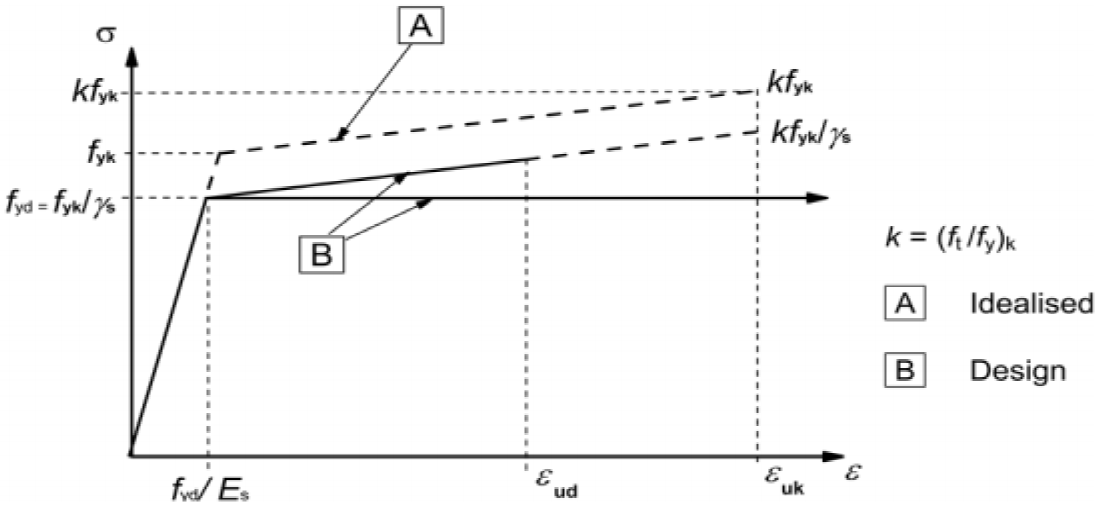


Figure 1. 5. Idealized and design stress-strain diagrams for reinforcing steel (Eurocode 2)

1.2. Design method in reinforced concrete building

The design of an engineering structure must ensure that under the worst loadings the structure is safe and during normal working conditions the deformation of the members does not detract from the appearance, durability or performance of the structure. Despite the difficulty in assessing the precise loading and variations in the strength of the concrete and steel, these requirements have to be met. Three basic methods using factors of safety to achieve safe, workable structures have been developed over many years which are the permissible stress method, the load factor method and the limit state method.

1.2.1. The permissible stress method

This is a method in which ultimate strengths of the materials are divided by a factor of safety to provide design stresses which are usually within the elastic range. This method has proved to be a simple and useful method but it does have some serious inconsistencies and is generally no longer in use. Because it is based on an elastic stress distribution, it is not really applicable to a semi-plastic material such as concrete, nor is it suitable when the deformations are not proportional to the load, as in slender columns. It has also been found to be unsafe when dealing with the stability of structures subject to overturning forces.

1.2.2. The load factor method

This is a method in which the working loads are multiplied by a factor of safety. In this method, the ultimate strength of the materials is used in the calculations. As this method does not apply factors of safety to the material stresses, it cannot directly take account of the variability of the materials and also it cannot be used to calculate the deflections or cracking at working loads. Again, this is a design method that has now been effectively superseded by modern limit state design methods.

1.2.3. The limit state method

It is a method which multiplies the working loads by partial factors of safety and also divides the materials' ultimate strengths by further partial factors of safety. This method of design, now widely adopted across Europe and many other parts of the world, overcomes many of the disadvantages of the previous two methods. It does so by applying partial factors of safety, both to the loads and to the material strengths, and the magnitude of the factors may be varied so that they may be used either with the plastic conditions in the ultimate state or with the more elastic stress range at working loads. This flexibility is particularly important if full benefits are to be obtained from development of improved concrete and steel properties.

The purpose of design using this method is to achieve acceptable probabilities that a structure will not become unfit for its intended use, that is it will not reach a limit state. Thus, any way in which a structure may cease to be fit for use will constitute a limit state and the design aim is to avoid any such condition being reached during the expected life of the structure. The two principal types of limit state are the ultimate limit state and the serviceability limit state.

1.2.3.1. The ultimate limit state (ULS)

This requires that the structure must be able to withstand, with an adequate factor of safety against collapse, the loads for which it is designed to ensure the safety of the building occupants and/or the safety of the structure itself. The possibility of buckling or overturning must also be taken into account, as much as the possibility of accidental damage.

BS EN 1990 Eurocode describes four ultimate limit states which are

- EQU: Loss of static equilibrium of the structure.
- STR: Internal failure or excessive deformation of the structure.
- GEO: Failure or excessive deformation of the ground.
- FAT: Fatigue failure of the structure.

1.2.3.2. Serviceability limit state (SLS)

It is a limit state design which ensures that a structure is comfortable and useable. This includes vibrations and deflections, as well as cracking and durability. These are the conditions that are not strength-based but still may render the structure unsuitable for it intended use, that is it may cause occupant discomfort under routine conditions. It also involves limits to non-structural issues such as acoustics and heat transmission. SLS requirements tend to be less rigid than strength-based limit states as the safety of the structure is not in question. A structure must remain functional for its intended use subject to routine loading in order to satisfy SLS criterion.

1.3. Ductility

Ductility is the ability of a structure or a selected structural component to deform beyond the elastic limits without excessive strength or stiffness degradation (Pauley et al, 1992). It is also the structural property that will need to be relied on in most buildings if satisfactory behavior under damage control and survival limit state is to be achieved. For economical resistance against strong earthquakes most structures must behave inelastically. Resistance earthquake design of RC building structure should ensure whole property resistance earthquake of structure, according to (Men and Qiu, 1998). Ductility demand of resistance earthquake of RC frame building structure is:

- Strong column and soft beam of RC frame
- Moment regulate in beam end of RC frame
- Shear-pressure ratio in beam of RC frame
- Strong shear and soft curve in column of RC frame
- Strong connect and soft member of RC frame point
- Shear-pressure ratio in column of RC frame
- Shear-pressure ratio in point of RC frame

• Axial pressure ratio in column of RC frame

• Axial force increases for frame supported column

• Moment increase for column base in base story of column

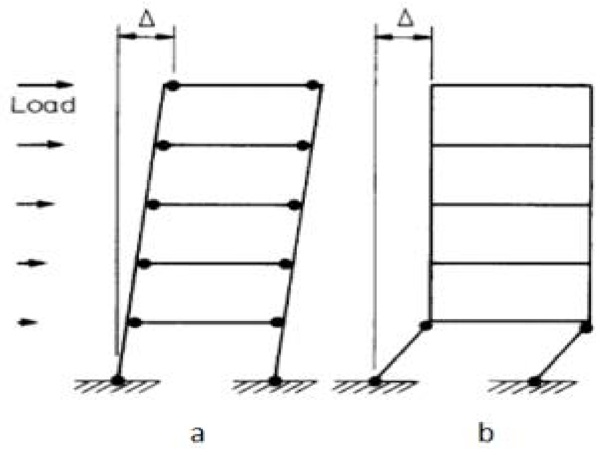


Figure 1. 6. Sway mechanisms in laterally loaded multistory frames. (Paulay, 1983)

1.3.1. Ductility relationships

Ductility being defined as the ratio of the total imposed displacements Δ at any instant to that at the onset of yield Δ_y. Using the idealization of figure 1.7, a comparison of ductile and brittle failure responses is illustrated and a general definition of ductility is given by equation 1.3. It is now necessary to trace briefly specific sources of ductility and establish the relationship between different kinds of ductility. As the term ductility is not specific enough and because misunderstandings in this respect are not uncommon, the various ways of quantifying ductility are reviewed here in some detail.

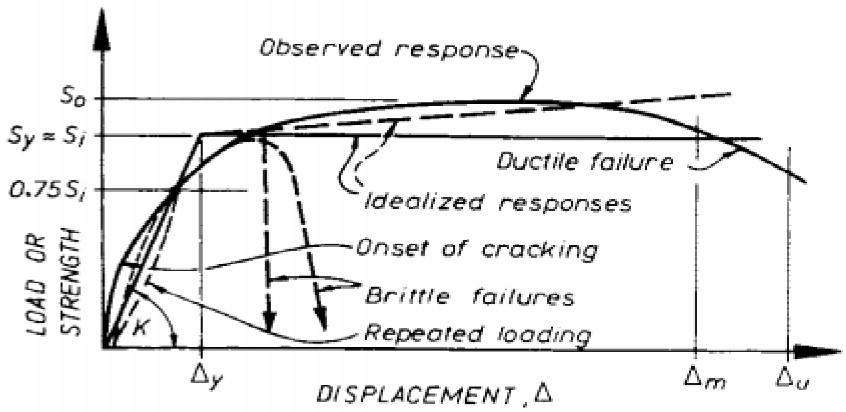


Figure 1. 7. Typical load -displacement relationship for a reinforced concrete element (Pauley et al, 1992)

$$\mu = \Delta / \Delta_y \qquad \text{(Eq1.3)}$$

The displacements Δ_y and Δ in equation 1.3 and figure 1.8 may represent strain, curvature, rotation or deflection.

Strain ductility: The fundamental source of ductility is the ability of the constituent materials to sustain plastic strains without significant reduction of stress. By similarity to the response shown in figure 1.8, strain ductility is defined as

$$\mu_\varepsilon = \varepsilon / \varepsilon_y \qquad \text{(Eq1.4)}$$

Where ε is the total strain imposed and ε_y is the yield strain.

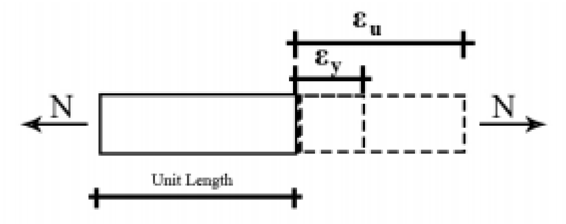


Figure 1. 8. Strain ductility (Mahmoud, 2017)

Curvature ductility: It is the ratio between the curvature angle (rotation angle per unit length) with a maximum angle of curvature of the melting of a structural element due to bending force.

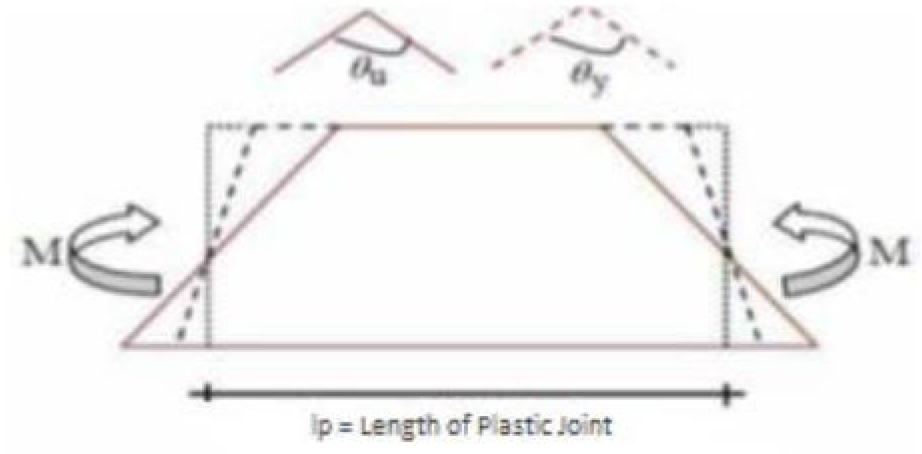


Figure 1. 9. Curvature ductility (Mahmoud, 2017)

Rotational ductility is the ratio between the maximum rotation angle of the rotation angle of melting;

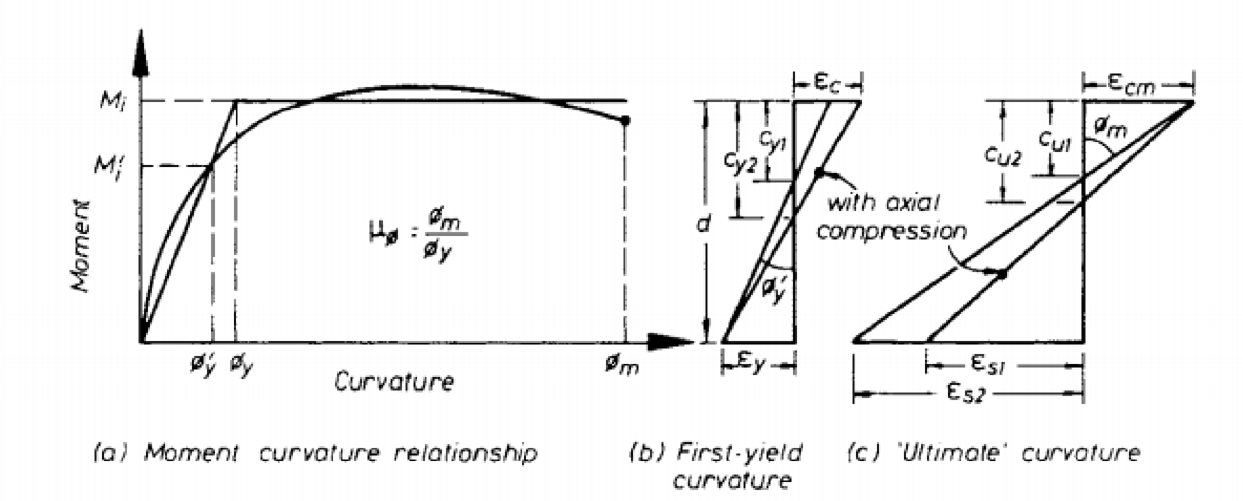


Figure 1. 10. Rotation ductility (Pauley et al, 1992)

Displacement ductility is the ratio between the maximum structural displacements in the lateral direction of the movement of the structure while melting

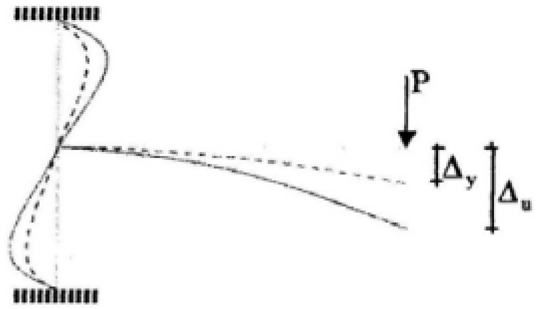


Figure 1. 11. Displacement ductility (Mahmoud, 2017)

1.3.2. Ductility classification according to code provisions

Currently, all seismic design codes take into account the effect of inelastic energy dissipation by reducing the design seismic force by a ‘response reduction factor’ (also called ‘behavior factor’). Values of response reduction factor are provided for different ductility classes of buildings. In addition to ductility, the response reduction factor also takes into account the effect of overstrength. The New Zealand seismic code (NZS 1170.5, 2004) considers a separate structural performance factor in addition to the ductility factor, which represents the combined effect of the limited number of cycles having peak amplitude, overstrength, redundancy, and over-capacity due to damping in secondary components and in the foundation. Further, only the New Zealand seismic (NZS 1170.5, 2004) considers the effect of period on the relationship between ductility and the response reduction factor. All other codes provide constant response reduction factors for a particular construction type, irrespective of the period of vibration. The EUA code (ASCE 7-10, 2010) classifies RC frame buildings into three ductility classes: Ordinary Moment Resisting Frame (OMRF), Intermediate Moment Resisting Frames (IMRF) and Special Moment Resisting Frames (SMRF). The European code, Eurocode 8 (EN 1998-1, 2004) classifies the building ductility as Ductility Class Low (DCL), Ductility Class Medium (DCM) and Ductility Class High (DCH). NZS 1170.5 classifies structures into three ductility classes, namely Ductile Structures (DS), for which the structural ductility factor is greater than 1.25 but less than 6, Structures of Limited Ductility (SLD), which is a subset of DS with structural ductility factor between 1.25 and 3, and Nominal Ductile Structures (NDS), for which the ductility factor is between 1 and 1.25. (IS 1893 - Part 1, 2002) classifies RC frame buildings as Ordinary Moment Resisting Frames (OMRF) and Special Moment Resisting Frames (SMRF) (Khose et al, 2012).

Seismic design codes either provide guidelines for the design and detailing of RC buildings for different ductility classes or refer to complimentary design codes. These provisions, as summarized in table 1.3, consist of four requirements:

Table 1. 3. Overview of ductile detailing requirements for RC frame buildings in different seismic design codes (Khose et al, 2012)

Ductile Detailing Criteria		ASCE 7[1)]			Eurocode 8			NZS 1170.5[2)]			IS 1893[3)]	
		OMRF	IMRF	SMRF	DCL	DCM	DCH	NDS	SLD	DS	OMRF	SMRF
Capacity Design	Strong Column Weak Beam	○	○	●	○	●	●	○	●	●	○	○
	Capacity Shear for Column	○	●	●	○	●	●	○	●	●	○	●
	Capacity Shear for Beam	○	●	●	○	●	●	○	●	●	○	●
Special Confinement Reinforcement	Column	○	●	●	○	●	●	○	●	●	○	●
	Beam	○	●	●	○	●	●	○	●	●	○	●
Special Anchorage Requirement	Interior Joint	○	○	●	○	●	●	●	●	●	○	●
	Exterior Joint	○	○	●	○	●	●	●	●	●	○	●
Joint Shear Design		○	○	●	○	○	●	●	●	●	○	○

1) Ductile detailing as per (ACI, 2008)
2) Ductile detailing as per (NZS 3101:Part 1., 2006) and (NZS 1170.5, 2004)
3) Ductile detailing of OMRF and SMRF as per (IS 465, 2000) and (IS 13920, 1993), respectively.

○ provision is not available
● provision is available

It is evident from Table 1.3 that it is not possible to have a one-to-one parity between different ductility classes of various codes. However, three broad categories of ductility can be considered, as shown in Table 1.4, where each category includes building classes with similar ductility provisions. Figure 1.12 shows the response reduction/behavior factors for different ductility classes of RC frames, according to different codes

Table 1. 4. Different ductility categories of RC frame buildings (Khose et al, 2012)

Category	Ductility classes			
	ASCE 7	Eurocode 8	NZS 1170.5	IS 1893
Low dissipative structures	OMRF	DCL	NDS	OMRF
Medium dissipative structures	IMRF	DCM	SLD	SMRF
High dissipative structures	SMRF	DCH	DS	-

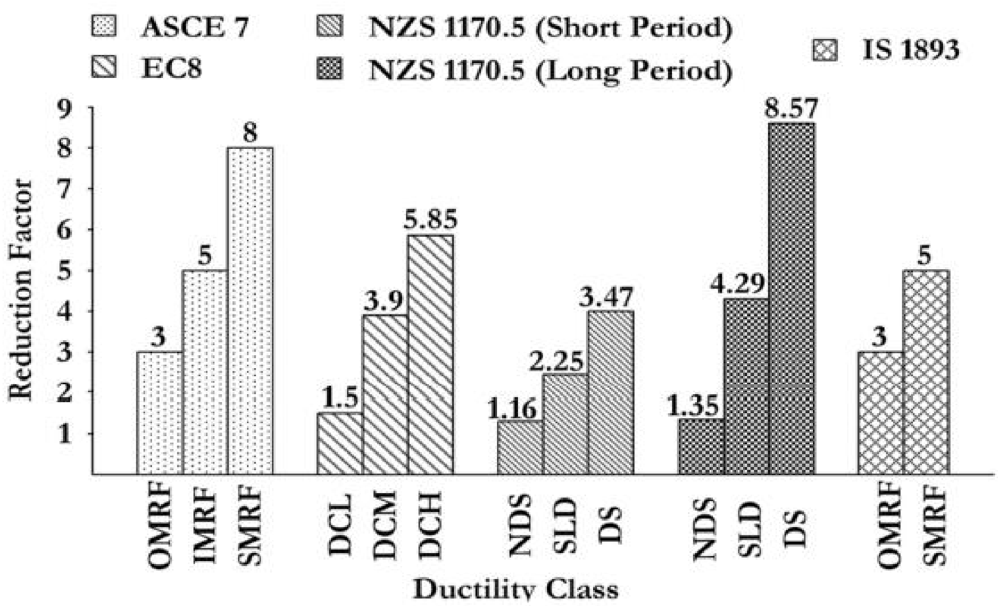


Figure 1. 12. Comparison of reduction/behavior factors recommended in different national codes (Khose et al)

In this work, our focus will be based the classification of ductility according to Eurocode.

1.3.3. Energy dissipation and ductility classification based on Eurocode

Concrete buildings can be designed for low dissipation capacity and low ductility, by applying only the rules of EN 1992-1-1:2004 for the seismic design situation, and neglecting the specific provisions given in EN 1998. Such buildings are termed ductility class low (L), while resistant concrete buildings other than those of ductility class low are designed to provide energy dissipation capacity and an overall ductile behaviour. Overall ductile behaviour is ensured if the ductility demand involves globally a large volume of the structure spread to different elements and locations of all its storeys. Concrete buildings designed in accordance with earthquake resistant, are classified in two ductility classes DCM (medium ductility) and DCH (high ductility), depending on their hysteretic dissipation capacity.

1.3.3.1. Ductility class low (DCL)

Buildings of DCL are designed only for strength and not for ductility, except certain minimum conditions for the ductility of reinforcing steel. They have to follow just the dimensioning and detailing rules specified in Eurocode 2 for non-seismic actions. Although they are expected to stay elastic under the combination of the design seismic action and the concurrent gravity loads, they use a behavior factor value of q = 1.5. DCL buildings are not cost-effective for moderate or high seismicity.

1.3.3.2. Ductility class medium (DCM)

Buildings of DCM have q-factor values higher than the default value of 1.5 used for DCL and lesser q-factor value than those of DCH and are considered as due to overstrength alone. However, unlike DCL, DCM does not systematically require more steel than DCH. The total quantities of materials are essentially the same but the level of detailing is not as complex as that of DCH that is, it is easier to design and implement, thus ensuring a better performance in moderate earthquakes. The design procedure is detailed in section 1.3.3.2.a. and 1.3.3.2.b

a. Geometrical constraints and materials

For the material requirements,

- Concrete of class lower than C16/20 is not used in the primary seismic elements.
- With the exceptions of closed stirrups and cross-ties, only ribbed bars shall be used as reinforcing steel in critical regions of primary seismic elements.
- In critical regions of primary seismic elements, reinforcing steels of class B and C in EN 1992-1-1:2004 are used

The geometrical constraints with respect to this work take into consideration just the beams and the columns.

i. Beams

- The eccentricity of the beam axis relative to that of the column into which it frames is limited, to enable efficient transfer of cyclic moments from a primary seismic beam to a column to be achieved.
- In order for the eccentricity of the beam axis relative to that of the column to be limited, the distance between the centroidal axes of the two members is also limited to less than $b_c/4$, where b_c is the largest cross-sectional dimension of the column normal to the longitudinal axis of the beam.

- To take advantage of the favourable effect of column compression on the bond of horizontal bars passing through the joint, the width bw of a primary seismic beam shall satisfy the following expression:

$$b_W \leq min\{b_c + h_W;\ 2b_c\} \qquad \text{(Eq. 1.5)}$$

where hw is the depth of the beam

ii. Column

Unless θ ≤ 0,1, the cross-sectional dimensions of primary seismic columns should not be smaller than one tenth of the larger distance between the point of contraflexure and the ends of the column, for bending within a plane parallel to the column dimension considered.
where θ is the interstorey drift sensitivity coefficient.

b. Design action effects

The designed values of shear forces of primary seismic beams and columns are determined in this sub section.

i. Beams

In primary seismic beams the design shear forces are determined in accordance with the capacity design rule, on the basis of the equilibrium of the beam under transverse load acting on it in the seismic design situation and end moments $M_{i,d}$ (with i=1,2 denoting the end sections of the beam), corresponding to plastic hinge formation for positive and negative directions of seismic loading. The plastic hinges are formed at the ends of the beams or (if they form there first) in the vertical elements connected to the joints into which the beam ends frame as shown in figure 1.13.

At end section i, two values of the acting shear force are calculated, that is the maximum $V_{Ed,max,i}$ and the minimum $V_{Ed,min,i}$ corresponding to the maximum positive and the maximum negative end moments $M_{i,d}$ that can develop at ends 1 and 2 of the beam. $M_{i,d}$ is determined as shown in equation 1.6.

$$M_{i,d} = \gamma_{Rd} M_{Rb,i} min\left(1, \frac{\sum M_{Rc}}{\sum M_{Rb}}\right) \qquad \text{(Eq. 1.6)}$$

Where γ_{Rd} is the factor accounting for possible overstrength due to steel strain hardening, which in the case of DCM beams is taken as being equal to 1,0;

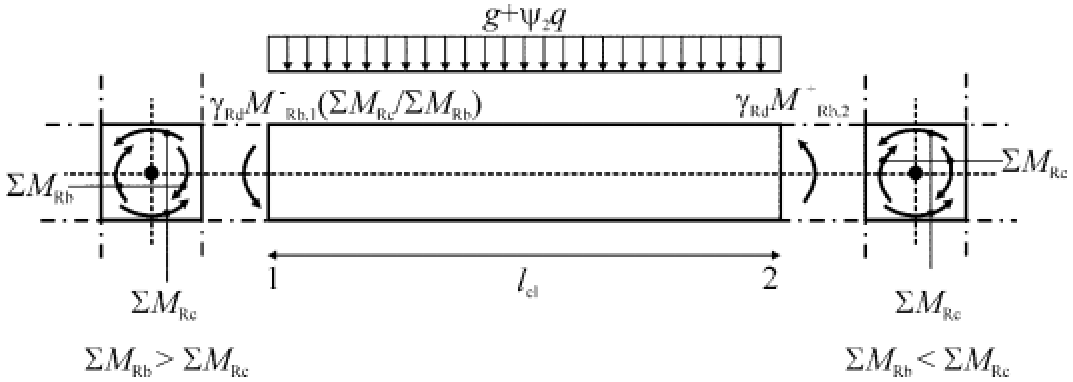


Figure 1. 13. Capacity design values of shear forces on beams

ii. Columns

In primary seismic columns the design values of shear forces are determined in accordance with the capacity design rule, on the basis of the equilibrium of the column under end moments $M_{i,d}$ (with i=1,2 denoting the end sections of the column), corresponding to plastic hinge formation for positive and negative directions of seismic loading. The plastic hinges are formed at the ends of the beams connected to the joints into which the column end frames, or (if they form there first) at the ends of the columns as shown in figure 1.14. End moments $M_{i,d}$ is determined from equation 1.7.

$$M_{i,d} = \gamma_{Rd} M_{Rc,i} min\left(1, \frac{\sum M_{Rb}}{\sum M_{Rc}}\right) \qquad \text{(Eq. 1.7)}$$

Where γ_{Rd} is the factor accounting for overstrength due to steel strain hardening and confinement of the concrete of the compression zone of the section, taken as being equal to 1,1

The values of $M_{Rc,i}$ and ΣM_{Rc} correspond to the column axial force(s) in the seismic design situation for the considered sense of the seismic action.

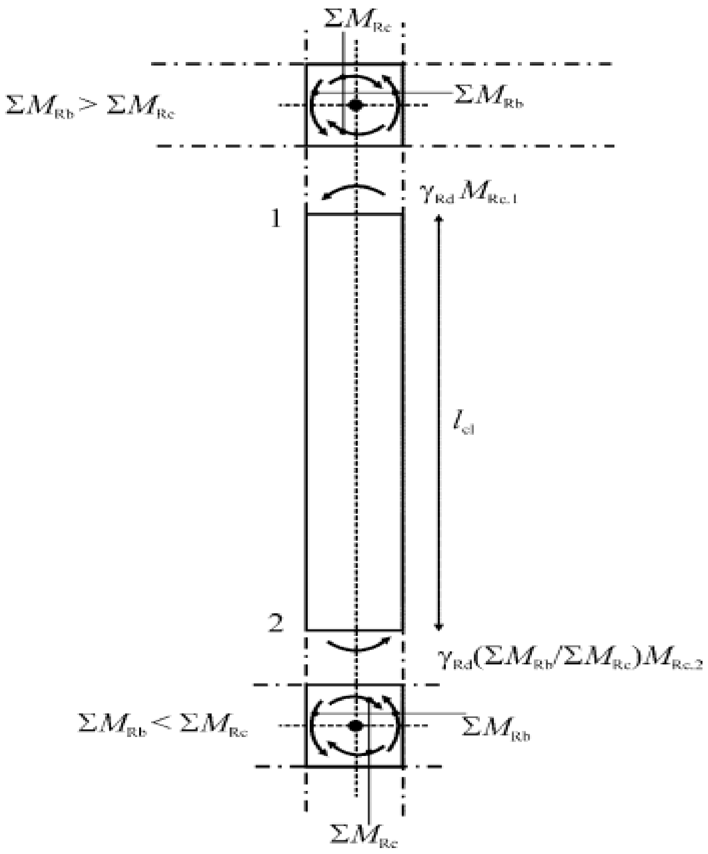


Figure 1. 14. Capacity design shear force in columns

c. ULS verification and detailing for beam-column joints

The horizontal confinement reinforcement in joints of primary seismic beams with columns should be not less than that specified in the critical regions of columns, with the exception of the case listed in the following paragraph

If beams frame into all four sides of the joint and their width is at least threequarters of the parallel cross-sectional dimension of the column, the spacing of the horizontal confinement reinforcement in the joint may be increased to twice that specified in the column, but may not exceed 150 mm.

At least one intermediate (between column corner bars) vertical bar shall be provided at each side of a joint of primary seismic beams and columns.

1.3.3.3. Ductility class high (DCH)

Buildings of DCH enjoy higher values of q-factor than those of DCM and DCL. In return, they are subject to stricter detailing rules and have higher safety margins in capacity design against shear. DCH provide larger safety margins than M against collapse under earthquakes (much) stronger than the design seismic action and may be more economic for high seismicity, especially if there is a strong local tradition and expertise in seismic design and on-site implementation of complex detailing.

Eurocode 8 does not relate the choice between DC M and H to seismicity or the importance of the structure, nor puts limits to their application. (Fardis et al, 2015)

a. Geometrical constraints and materials

The material requirements are the same as those stated in DCM except for the fact that the concrete class used in primary seismic elements should not be less than C20/25.

i. Beams

In addition to the rules stated for DCM for beams in section 1.3.3.2 a, the following rules are also considered for the geometrical constraints in beams.

- The width of primary seismic beams should not be less than 200mm
- The width to height ratio of the web of primary seismic beams should satisfy expression 5.40b of EN 1992-1-1:2004

ii. Columns

In addition to the rule stated for DCM for columns in section 1.3.3.2a, the minimum cross-sectional dimension of primary seismic columns should not be less than 250mm.

b. Design action effects

i. Beams

The rules stated in section 1.3.3.2b for beams apply for DCH except for the fact that the value of γ_{Rd} in equation 1.6 is equal to 1.2.

ii. Columns

The rules stated in section 1.3.3.2b for columns apply for DCH except for the fact that the value of γ_{Rd} in equation 1.7 is equal to 1.3.

iii. Beam-columns joints

The horizontal shear acting on the core of a joint between primary seismic beams and columns is determined taking into account the most adverse conditions under seismic actions, that is capacity design conditions for the beams framing into the joint and the lowest compatible values of shear forces in the other framing elements.

The horizontal shear force acting on the concrete core for interior and exterior beam-column joints are given in equation 1.8 and 1.9 respectively.

$$V_{jhd} = \gamma_{Rd}(A_{s1} + A_{s2})f_{yd} - V_C \qquad \text{(Eq.1.8)}$$

$$V_{jhd} = \gamma_{Rd}.A_{s1}.f_{yd} - V_C \quad \text{(Eq.1.9)}$$

where

A_{s1} is the area of the beam top reinforcement;

A_{s2} is the area of the beam bottom reinforcement;

V_C is the shear force in the column above the joint, from the analysis in the seismic design situation;

γ_{Rd} is a factor to account for overstrength due to steel strain-hardening and is not less than 1,2

c. ULS verifications and detailing

The ULS verification and detailing in this work is focused on the beam-column joints only. At the beam-column joints:

- The diagonal compression induced in the joint by the diagonal strut mechanism should not exceed the compressive strength of concrete in the presence of transverse tensile strains. That is equation 1.10 shall be satisfied for the interior beam-column joints

$$V_{jhd} \leq \eta f_{cd}\sqrt{1-\frac{v_d}{\eta}}\,b_j h_{jc} \quad \text{(Eq.1.10)}$$

Where $\eta = 0.6(1 - f_{ck}/250)$

For the exterior beam-column joints, the left-hand side of equation 1.14 should be less than 80% of the value given by the right-hand side.

- Adequate confinement (both horizontal and vertical) of the joint are provided, to limit the maximum diagonal tensile stress of concrete max σ_{ct} to f_{ctd}. In the absence of a more precise model, this requirement is satisfied according to equation 1.11 by providing horizontal hoops with a diameter of not less than 6 mm within the joint.

$$\frac{A_{sh}.f_{ywd}}{b_j.h_{jw}} \geq \frac{\left(\frac{V_{jhd}}{b_j.h_{jc}}\right)^2}{f_{ctd} + v_d f_{cd}} - f_{ctd} \quad \text{(Eq.1.11)}$$

- As an alternative to the rule specified in equation 1.15, integrity of the joint after diagonal cracking may be ensured by horizontal hoop reinforcement. To this end, total area of horizontal hoops is provided in the interior and exterior joint as specified in equation 1.12 and 1.13 respectively. The horizontal hoops calculated is uniformly distributed within the depth h_{jw} between the top and bottom bars of the beam. In exterior joints they enclose the ends of beam bars bent toward the joint.

$$A_{sh}f_{ywd} \geq \gamma_{Rd}(A_{s1} + A_{s2})f_{yd}(1 - 0.8v_d) \quad \text{(Eq.1.12)}$$

$$A_{sh} \geq \gamma_{Rd} A_{s2} f_{yd} (1 - 0.8 v_d) \quad \text{(Eq. 1.13)}$$

- Adequate vertical reinforcement of the column passing through the joint is also provided, such that equation 1.14 is satisfied.

$$A_{sv} \geq (2/3). A_{sh}(h_{jc}/h_{jw}) \quad \text{(Eq.1.14)}$$

In addition to the ULS verification and detailing stated in this section, the rules stated in section 1.3.3.2c for beam-column joints also apply for DCH.

1.3.4. Behavior factor of DCM and DCH buildings.

In Eurocode 8, the value of the behavior factor, q of DC M and DCH buildings depends on:

- Ductility Class
- The type of lateral-force-resisting-system
- The regularity or lack of the structural system in elevation.

The value of the q-factor is linked indirectly (through the ductility classification) or directly, to the local ductility and detailing requirements for members.

According to Eurocode 8, basic value, qo of DCM and DCH behavior factor per EC8 for height-wise regular frame moment-resistant buildings is shown in the table 1.5.

Table 1. 5. Basic value q_0, of behaviour factor per EC8 for height-wise regular buildings

Lateral-load-resisting structural system:		DC M	DC H
1	Inverted pendulum	1.5	3
2	Torsionally flexible	2	3
3	Uncoupled wall system, not in one of the two categories above	3	$4\alpha_u/\alpha_l$
4	Any structural system other than the above	$3\alpha_u/\alpha_l$	$4.5\alpha_u/\alpha_l$

Where α_u/α_1 = 1.3 for multi-storey multi-bay frames or frame-equivalent dual systems.

1.4. Joints in reinforced concrete building

The joint is defined as the portion of the column within the depth of the deepest beam that frames into the column. The functional requirement of a joint, which is the zone of intersection of beams and columns, is to enable the adjoining members to develop and sustain their ultimate capacity. The demand on this finite size element is always severe especially under seismic loading. The joints should have adequate strength and stiffness to resist the internal forces induced by the framing members.

1.4.1. Performance criteria

It is generally recognized that beam-column joints can be critical regions in reinforced concrete frames designed for inelastic response to severe seismic attack. As a consequence of seismic moments in columns of opposite signs immediately above and below the joint and similar beam moment reversal across the joint, the joint region is subjected to horizontal and vertical shear forces whose magnitude is typically many times higher than in the adjacent beams and columns. If not designed for, joint shear failure can result. The reversal in moment across the joint also means that the beam reinforcement is required to be in compression on one side of the joint and at tensile yield on the other side of the joint. The high bond stresses required to sustain this force gradient across the joint may cause bond failure and corresponding degradation of moment capacity accompanied by excessive drift.

The ductility and associated energy dissipating capacity of reinforced concrete frame is anticipated to originate primarily from chosen and appropriately detailed plastic hinges in beams and columns. Since the response of joints is controlled by shear and bond mechanisms, both of which exhibit poor hysteretic properties, joints should be regarded as being unsuitable as major sources of energy dissipation. Hence, the criteria for desirable performance of joints in ductile structures designed for earthquake resistance is summarized as follows:

- The joint should have sufficient strength to enable the maximum capacities to be mobilized in the adjoining flexural members.
- The degradation of joints should be so limited such that the capacity of the column is not affected in carrying its design loads.
- The joint deformation should not result in increased storey drift.
- The joint reinforcement necessary to ensure satisfactory performance should not cause undue construction difficulties.
- During moderate disturbances, joints should preferably respond within the elastic range.

1.4.2. Types of joints in frames

Joints may be classified in terms of geometric configuration (exterior and interior joints), as well as structural behaviours that is ductile and non-ductile joints (Paulay et al, 1992)

1.4.2.1. Classification according to geometric configuration

Based on geometric configuration, two types of joints can be identified that is exterior and interior joints.

a. Exterior beam-column joints

Different kinds of exterior joints exist in a building as shown in Figure 1.15. These joints are distinguished in a plane frame as well as in a space frame. In a plane frame we have corner joints at the roof (Figure 1.15(a)) and at intermediate floor (Figure 1.15(d)). In the space frame, we have the roof corner joint (Figure 1.15(b)), the roof edge joint (Figure 1.15(c)), the intermediate floor corner joint (Figure 1.15(e)), and the intermediate floor edge joint (Figure 1.15(f)). Framing of beams in columns, depending on design purposes, at angles other than 90° and 180° can happen as illustrated in Figure 1.15(g). A third beam, from the floor interior could possibly frame into the column as shown by the dashed lined beam.

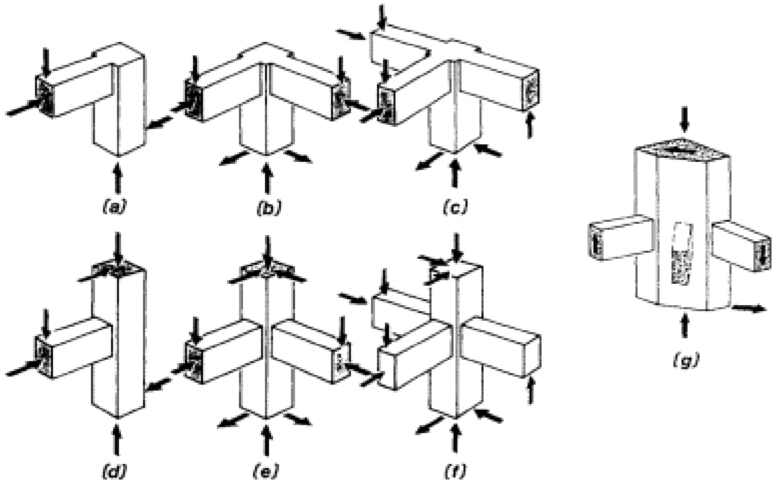


Figure 1. 15. Exterior beam-column joints. (Paulay. T and Preistley M.N.J, 1992)

b. Interior beam-column joints

Interior beams-column joints are those joints in at least two beams framing into a continuous column on the opposite sides. According to this definition, joints like space frame roof edge joints and plane frame floor edge joints can be treated as interior joints depending on loading direction. Likewise, exterior beam-column joints, different types of interior beam-column joints exist and are shown in Figure 1.16. In the plane frame, there is the intermediate floor middle joint (Figure 1.16(b)) and the middle roof joint (Figure 1.16(a)). The space frame comprises of the middle roof joint (Figure 1.16(c)), and the intermediate floor middle joint (Figure 1.16(d)). The severity of forces and demands on the performance of these joints calls for greater understanding of their seismic behaviour. These forces develop complex mechanisms involving bond and shear within the joint.

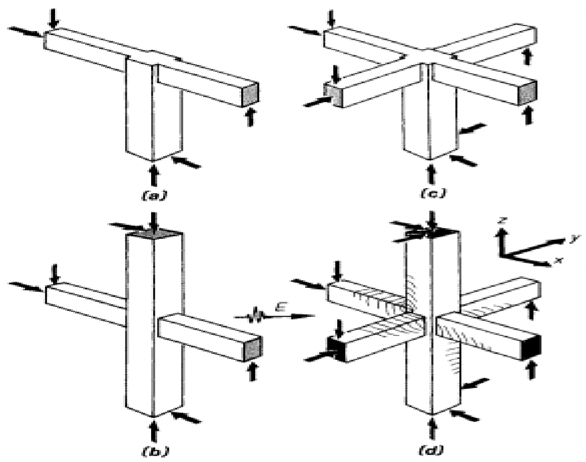


Figure 1. 16. Interior beam column joints. (Paulay. T and Priestley M.N.J, 1992)

1.4.2.2. Classification according to structural behaviour

The reinforcement detailing of beam-column joints forms a major part of the ductile detailing norms prescribed by newer seismic design codes ACI 318 (2011), NZS 3101 (2006), EC8 (2004), IS 13920 (2002). The parameters verified by these codes include shear reinforcement in joint core, larger anchorage length, long lap splice, closely spaced ties for better confinement. Whether a joint of the structure will behave in a brittle or ductile manner depends largely on its reinforcement details. Joints can be further classified according to detailing specifications. Indeed, reinforcements in joints play a major role in the resisting mechanism of the moment resisting frame of the reinforced concrete structure.

Based on the amount of reinforcements present in a beam-column joint and the behaviour under loading, joints can be classified in two categories (Rolf et al, 2003).

- Joints of non-seismically detailed structures
- Joints of seismically detailed structures

a. Non-ductile joints

Non-ductile joints are those, not expected to undergo large deformations before failure (Rolf et al, 2013). The characteristics of such joints are insufficient development lengths, short lap splices, discontinuous reinforcements, larger stirrup spacing, and no reinforcement in the joint core. Figure 1.17 shows detailing of beam-column joints of a non-seismically detailed reinforced concrete frame structure (ATC- 40:1996). In these joints, the amount of horizontal reinforcements is lower than the limit proposed by the seismic codes and in ancient builds, reinforcements aren’t provided.

$$A_{sjh} \leq A_{sjh,EC8} \qquad \text{(Eq. 2.15)}$$

A joint of non-seismically detailed structures is illustrated in Figure 1.17. Such a joint is likely to fail through shear if subjected to seismic forces.

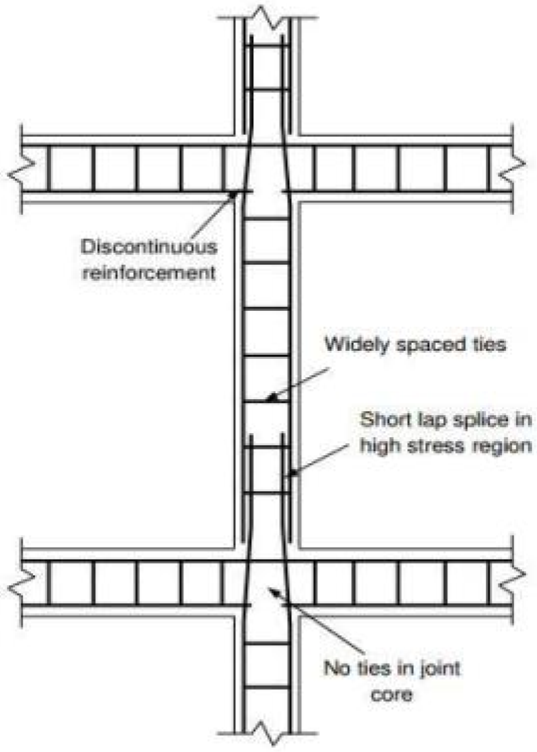


Figure 1. 17. Joints of non-seismically detailed structures (Rolf et al, 2013).

b. Ductile joints

Ductile joints have an ability to undergo large deformations without failure. Such joints absorb large amount of energy through the hysteretic behaviour under a severe earthquake (Rolf et al, 2013). The characteristics of such joints are large anchorage lengths, long lap splices, continuous reinforcements, closer stirrup spacing, and shear reinforcement in the joint core. Figure 1.18 shows typical ductile type reinforcement detailing prescribed by new codes of practice (IS 13920, 2002) for beam-column joints of a frame. Seismic codes compliant joints comprise of joints with an amount of shear reinforcements according to EC8 recommendations for DCH RC frames.

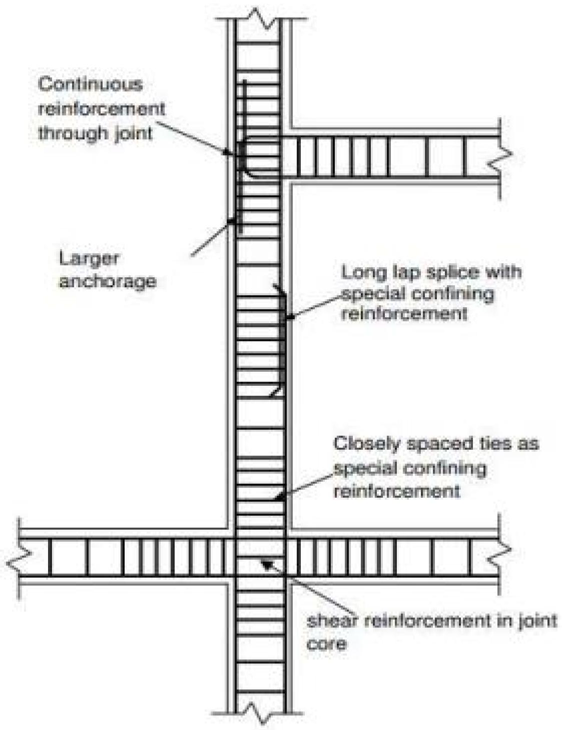


Figure 1. 18. Joints of seismically detailed structures (Rolf et al, 2013)

1.4.3. Features of joint behaviour

Under seismic and gravity loading, large shear forces may be introduced into the beam-column joints irrespective of whether plastic hinges develop at column faces or at other sections of the beam (Paulay et al,1992). These shear forces may cause a failure in the joint core due to the breakdown of shear or bond resisting mechanisms or both.

1.4.3.1. Equilibrium criteria of exterior and interior joints

As a joint is also part of a column, examination of it function as a column component is instructive. An interior column extending between points of contraflexure, at approximately half-story heights, may be isolated as a free body as shown in figure 1.19(a). Actions introduced by symmetrically reinforced beams to the column are shown in this figure to be internal horizontal tension Tb and compression C_b forces and vertical beam shear V_b forces. (Paulay et al, 1992).

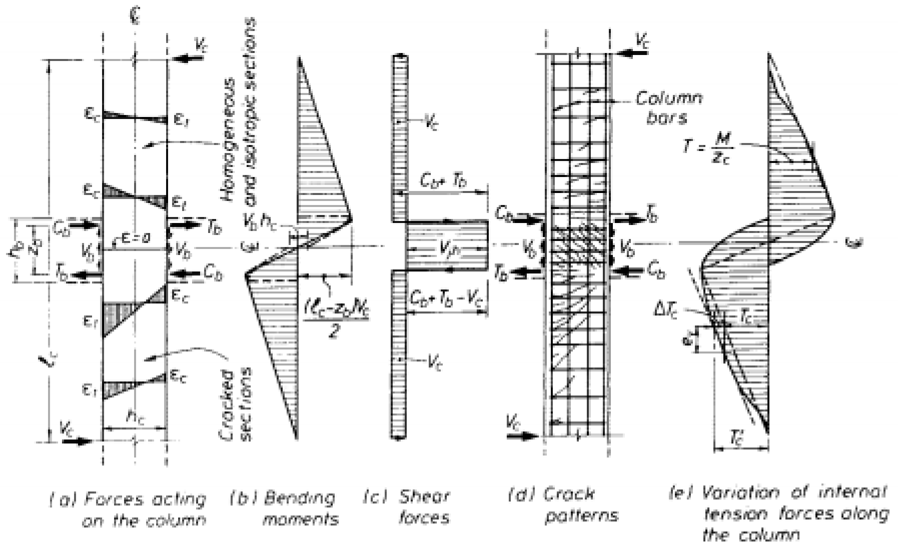


Figure 1. 19. Features of column and joint behaviour (Paulay et al, 1992)

Making the approximation that Tb = Cb, and that the beam shears on opposite sides of the joint are equal, for equilibrium condition, will require a horizontal column shear force equal to that shown by equation 1.16, where the variables are clearly identified in Figure 1.19(a).

$$V_C = \frac{2T_b Z_b + V_b h_c}{l_c} \qquad \text{(Eq.1.16)}$$

The corresponding moments and shear force diagrams for the column are shown in Figure 1.19(b) and Figure 1.19(c). The large horizontal shear force across the joint region is obtained as shown in equation 1.17. The right-hand side of the equation is obtained from the consideration of the moment gradient within the joint core.

$$V_{jh} = C_b + T_b - V_c = V_c\left(\frac{l_c}{Z_b} - 1\right) - \frac{h_c}{Z_b}V_b \qquad \text{(Eq.1.17)}$$

There is a moment decrement, $h_c V_b$, which could be, for example, assumed to occur at the centerline (dashed line across Figure 1.19(b)), which when considered gives the correct value of the horizontal shear force. The decrement is not shown in Figure 1.19(b), as it can be shown in the conventionally full-line moment diagram (Figure 1.19(b)). That is why its slopes does not give the correct value of the horizontal joint shear force.

1.4.3.2. Joint shear strength

According to the model of Paulay et al. (1978), the total force acting on joint core is resisted on one hand by a diagonal concrete strut (Figure 1.20(b)), and on the other hand by truss mechanism (idealized), consisting of stirrups, intermediate column vertical bars and inclined concrete struts between diagonal cracks. To prevent shear failure, usually along a potential corner to corner failure plane, both vertical and horizontal shear reinforcements will be required. Such reinforcements will enable a diagonal compression zone to be mobilized, which provides a feasible load path for both horizontal and vertical shearing forces (Paulay et al, 1992).

Yielding of the hoops occurs when the joint shear reinforcement is insufficient. Irrespective of the direction of the diagonal cracking, the horizontal shear reinforcements transmit essentially tension forces. Thus, irreversible inelastic steel strains may occur. Therefore, during subsequent loading, only if tensile strains imposed are larger than those developed previously, stirrups may contribute significantly to the shear resisting mechanism.

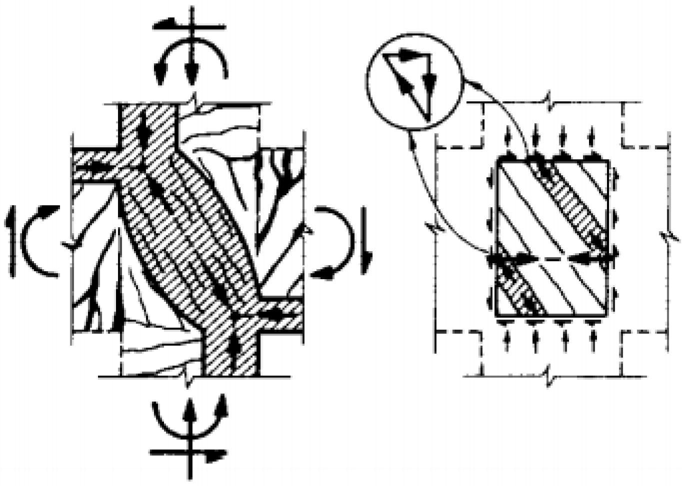


Figure 1. 20. Mechanisms of shear transfer at an interior joint (Paulay et al, 1992).

1.4.3.3. Bond strength

In a situation of an earthquake, the framing beams in a joint are subjected to moments in the same direction. The top and bottom bars are pulled in opposite directions by the moments and are countered (balanced) by the bond stresses developed between concrete and steel in the joint region. Figure 1.21 illustrates the bond stresses.

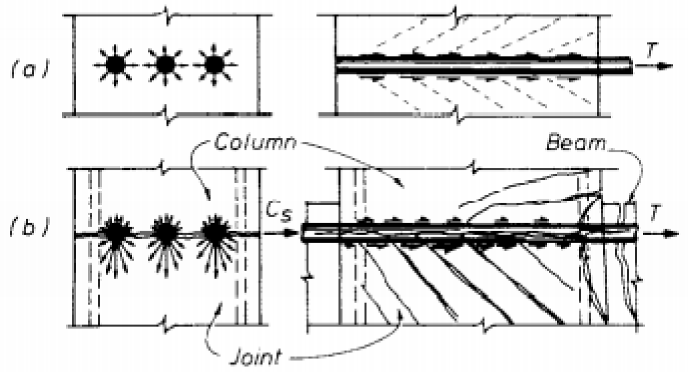


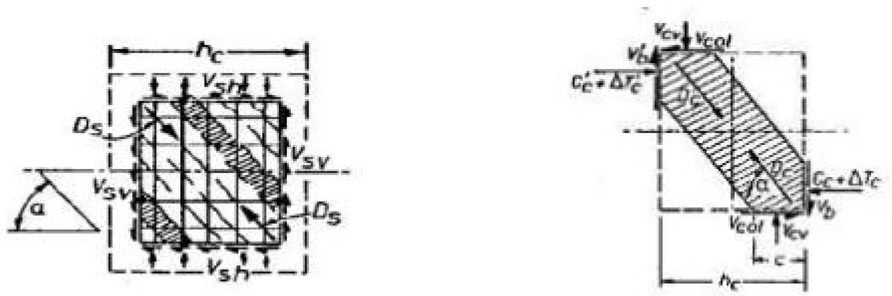
Figure 1. 21. Bond stresses around (a) bars simply anchored (b) those passing through an interior joint. (Paulay et al, 1992)

Even at moderate ductility demands, a slip of beam bars across the joint can occur. Fortunately, a breakdown of bond within the joint does not necessarily result in sudden loss of strength (Paulay et al, 1992). Still, hysteretic response of a ductile frame is significantly affected when a bond slip occurs.

As the stiffness of frames is rather sensitive to the bond performance passing through a joint, at interior columns mostly, special precautions must be taken to prevent premature bond deterioration in joints under seismic attack. At exterior joints, anchorage failure of beam bars is unacceptable at any stage because it results in complete loss in beam strength. The bond performance of bars anchored in joint greatly affect the relative contribution to the shear strength of the strut and the truss shear resisting mechanisms (Paulay et al., 1978).

1.4.4. Joint shear mechanisms

Internal forces transmitted from adjacent members to the joint as shown in Figure 1.19, result in joint shear forces in both the horizontal and vertical directions. These shear forces are a result of the moments from the structural elements framing into the joints. The induced shear forces lead to diagonal compression and tension stresses in the joint core. The latter will usually result in diagonal cracking of the concrete core. The diagonal compressive forces generated at the corners of the joint are responsible for resisting the major part of the total shear force and constitute the strut mechanism (Figure 1.20). Also, steel forces are transferred through bonds with concrete to concrete at the four boundaries of the joint core thereby producing a compression field zone (Figure 1.22(b)) in the joint core with diagonal cracks and a total diagonal compression force, Dc. The mechanism associated here is the truss mechanism. Transverse shear reinforcements are provided in this case, for effective resistance, to resist directly when the concrete core becomes severely cracked due to diagonal tensile strains (assuming no bond deterioration).



(a) Truss mechanism (b) Diagonal compression field under seismic loading

Figure 1. 22. Internal shear resisting mechanism internal joint (Paulay and Priestley, 1992)

1.4.4.1. Shear mechanisms in interior joint

Internal forces transmitted from adjacent members to joint result in joint shear horizontal and vertical forces. These forces lead to diagonal compression and tension forces which in turn lead to diagonal cracking. At this level, the resisting mechanism changes completely.

a. Actions and dispositions of internal forces at an interior joint

The study of the actions and disposition of internal forces is based on the assumption that, due to gravity loads and earthquake-induced lateral forces, moments introduced to the joints cause rotations in the same directions. The resulting moments, shear forces and axial forces (Figure 1.23(a)) are, for the sake of satisfying the equilibrium criteria and identifying load paths, are assembled around the joint core (Figure 1.23(b)).

Tensile stress resultants are denoted by T, compression stress resultants in concrete and in steel are denoted by Cc and Cs respectively. Figure 1.23(b) shows a situation where plastic hinges would have developed in the beams immediately adjacent to joint.

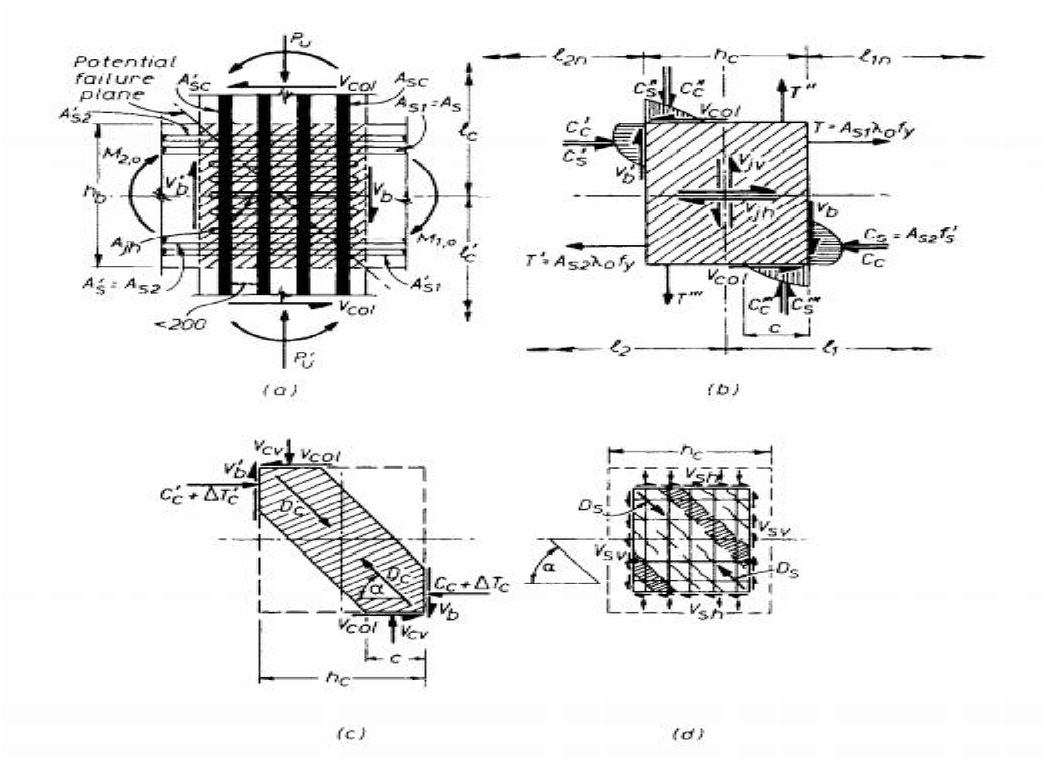


Figure 1. 23. External actions and internal stresses in an interior joint (Pauley and Priestly, 1992)

b. Development of joint shear forces

External actions on joints result in horizontal and vertical shear forces acting at the joint region. Also, these resultant shear forces are contributed for by the concrete and steel through the strut and truss shear resisting mechanism. As for the horizontal shear force, it is obtained through equation 1.18. The assemblage of an interior joint is shown in Figure 1.24.

$$V_{jh} = T + T' - V_{col} \qquad \text{(Eq.1.18)}$$

Where $T' = C_c' + C_s'$

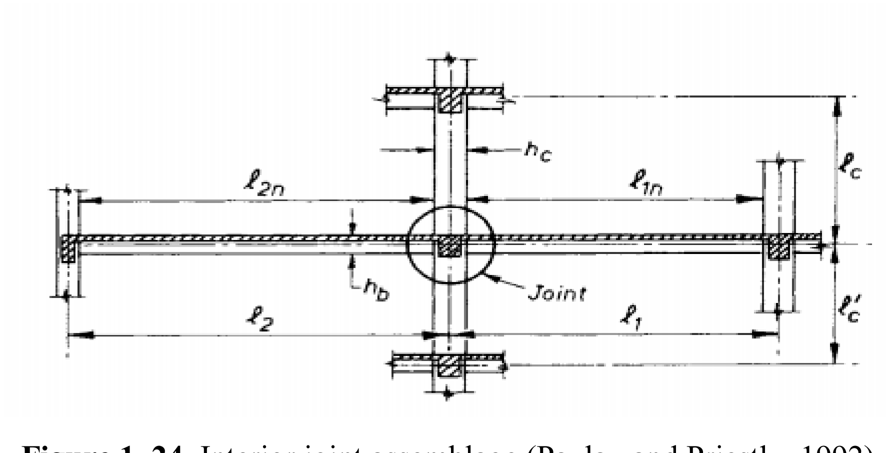


Figure 1. 24. Interior joint assemblage (Paulay and Priestly, 1992)

1.4.4.2. Shear mechanisms in exterior joints

In an exterior joint, the resisting mechanisms are the same as those in an interior joint. The strut and truss mechanisms are common to all types of joints irrespective of their location in the building. The main difference is at the number of elements framing into the joint. An example is an intermediate interior beam-column joint which six elements framing into it of which we have four beams and two columns. Whereas, an edge corner joint, still in a plane frame, has four elements consisting of two beams and two columns.

a. Actions and disposition of internal forces at an exterior joint

As an exterior joint, in a plane frame, has only one beam framing into a column, the joint shear strength will generally be lesser than that of interior joints. The assumptions made for the beam-column interior joints are valid for the exterior joint as well. Tensile stress resultants are denoted by T, compression stress resultants in concrete and in steel are denoted by Cc and Cs respectively.

b. Development of joint shear forces

The horizontal and vertical joint shear forces are given by equation 1.19 and 1.20 respectively

$$V_{jh} = T - V_{col} \quad \text{(Eq.1.19)}$$

$$V_{jv} = \left(\frac{h_b}{h_c}\right) V_{jh} \quad \text{(Eq.1.20)}$$

Where $T = f_s A_S \ or \ \lambda_0 f_y A_s$ depending whether an elastic beam section or critical section of a beam plastic hinge at the face of the column is being considered.

Similarly, to interior joints, concrete contribution to shear strength as well as shear reinforcement contribution can be computed separately. The concrete and steel contributions are estimated with the following expressions.

$$V_{ch} = C_C + \Delta T_c - V_{col} \quad \text{(Eq.1.21)}$$

$$V_{sh} = V_{jh} - V_{ch} \quad \text{(Eq.1.22)}$$

$$V_{cv} = V_{ch} tan\alpha \quad \text{(Eq.1.23)}$$

$$V_{sv} = D_s sin\alpha \quad \text{(Eq.1.24)}$$

1.4.5. Principal failure mode of beam column joints

Beam-column joints can fail under various failure modes. The principal failure modes of exterior and interior joints are the same and can be summarized as follows (Subramanian and Rao, 2003):

- Shear failure within the joint (mainly due to insufficient shear reinforcement and poor confinement of the joint).
- Anchorage failure of beam bars, if anchored within the joint.
- Bond failure of beam or column bars passing through the joint.

Yet, other mechanisms of failure are made taking into consideration the failure sequence of involved members in the joints. Hence, the possible failure modes are as follows:

- Joint failure: In this failure modes, beams and columns reinforcements are adequately designed to resist the seismic forces and thus, the unreinforced joint is the weak element in the frame. Consequently, without yielding of the beam and column bars, the joint fails in pure shear. This failure mode is the most representative of the actual shear strength of unconfined joints. Therefore, the joint-failure mode is less ductile, that is, it shows less ductility capacity.
- Column-joint failure mode: The connection is a strong beam-weak column, which is very rampant in older non-seismic resistant buildings. Failure is introduced by yielding of the column bars that penetrates the joint core and triggers shear failure. Joint shear capacity is awaited to be less than that of the joint failure due to softening of joint concrete strut due to column reinforcement yield penetration.
- Beam-joint failure: The yielding of the top or bottom beam reinforcements initiate the failure sequence. Immediately after the beam yielding, cracks appear on the joint followed by the joint shear failure. In this type of failure, the joint shear strength is directly related to beam flexural capacity. Unlike joint-failure mode, the beam-joint failure is more ductile as it involves yielding of the beam bars. The column experiences no yielding and the connection acts as a strong column-weak beam one.
- Beam -column joint failure mode: This is the combination of beam-joint and column-joint failure modes. Yielding in columns and beams occur simultaneously immediately followed by joint shear failure. Likewise, the column-joint and beam-joint failure modes, the joint shear strength is less than that in the joint-failure mode.
- Beam failure and column failure: In these failure modes, the mechanism consist of flexural yielding of beam reinforcements or column reinforcements undergoing large inelastic deformation. The deformation continues until ultimate rotational capacity of beam or column without shear failure.

1.4.6. Design specifications for capacity design and detailing of joints.

The essential requirements for the satisfactory performance of a joint in an RC structure during earthquakes, in line with the capacity design principles, can be summarized as follows (Paulay and Priestley (1992):

- The strength of the joint should be equal or greater than that of members framing into it. More specifically, the joint shear capacity of a beam-column joint should be such that it assures joint shear failure proceeds flexural yielding of beams and columns framing into it
- Detailing in the joint should assure that adjoining members may develop their full capacity.
- Lap splices of the joint should be located as far from the joint as practical. Longitudinal bars should not be terminated within a joint without suitable anchorage. Detailing should be such to ensure that longitudinal bars continuing through the joint do not buckle.
- The joint should be detailed with the consideration given to the ease of reinforcement assembly and concrete placement.

1.5. Finite element method (FEM)

Finite element method also known as FEM is a numerical method used to solve variety engineering problems by reducing the degrees of freedom from infinite to finite with the help of discretization that is meshing of nodes and elements (Sanjay et al, 2008). FEM involves analysis of the entire structure, instead of separately considering individual elements with simplified or assumed end conditions (Jagota et al, 2013). It thus helps in a more accurate estimate of the stresses in the members, facilitating optimum design.

All real-life objects are continuous. Means there is no physical gab between any two consecutive particles. As per material science, any object is made up of small particles, particles of molecules, molecules of atoms and so on and they are bonded together by the force of attraction. Solving a real-life problem with continuous material approach is difficult and the basis of all numerical methods is to simplify the problem by discretizing it. In simple words nodes work like atoms and with gap in between, filled by an entity called element. Calculations are made at nodes and the results are interpolated for elements. Hence FEM follows discrete approach.

Finite Element Analysis (FEA) based on FEM is a simulation, not reality, applied to the mathematical model. Even very accurate FEA may not be good enough, if the mathematical model is inappropriate or inadequate. A mathematical model is an idealization in which geometry, material properties, loads and/or boundary conditions are simplified based on the analyst's understanding of what features are important or unimportant in obtaining the results required. The error in solution can result from three different sources.

- Modelling error associated with the approximations made to the real problem. These approximations do not take into account the real behaviour
- Discretization error associated with type, size and shape of finite elements used to represent the mathematical model. It can be reduced by modifying mesh
- Numerical error which is based on the algorithm used and the finite precision of numbers used to represent data in the computer; most software uses double precision for reducing numerical error.

1.5.1. Principle of FEM

The simplified model replaces the actual component in FEM, represented by a finite number of elements connected at common points known as nodes, with an assumed behaviour or response of each element to the set of applied loads, and evaluating the unknown field variable (displacement, temperature) at these finite number of points. In FEM, the entire structure is analysed without using assumptions about the degree of fixity at the joints of members and hence better estimation of stresses and strains in the member is possible.

1.5.2. Classification of FEM

The basic problem in any engineering design is to evaluate displacements, stresses and strains in any given structure under different loads and boundary conditions. Several approaches of Finite Element Analysis have been developed to meet the needs of specific applications. They include the following methods:

1.5.2.1. Displacement method

The structure in this method is subjected to applied loads and/or specified displacements. The primary unknowns are displacements, obtained by inversion of the stiffness matrix, and the derived unknowns are stresses and strains. Stiffness matrix for any element can be obtained by variational principle, based on minimum potential energy of any stable structure. The displacement method is the most commonly used method and is suitable for solving most of the engineering problems.

1.5.2.2. Forced method

The structure is subjected to applied loads and/or specified displacements. The primary unknowns are member forces, obtained by inversion of the flexibility matrix, and the derived unknowns are stresses and strains. Calculation of flexibility matrix is possible only for discrete structural elements (such as trusses, beams and piping) and hence, this method is limited in the early analyses of discrete structures and in piping analysis.

1.5.2.3. Mixed method

The structure is subjected to applied loads and/or specified displacements. The method deals with large stiffness coefficients as well as very small flexibility coefficients in the same matrix. Analysis by this method leads to numerical errors and is not possible except in some very special cases

1.5.2.4. Hybrid method

Here the structure is subjected to applied loads and stress boundary conditions. This deals with special cases, such as airplane door frame which should be designed for stress-free boundary, so that the door can be opened during flight, in cases of emergencies.

1.5.3. Types of analysis

FEM with the help of CAE (Computer Aided Engineering) program perform linear static analysis, non-linear static analysis, dynamic analysis, thermal analysis, fatigue analysis, optimization, Computational Fluid Dynamics (CFD) analysis, crash analysis, Noise Vibration and Harshness (NVH) analysis

1.5.4. Meshing

Basic theme of FEA is to make calculation at only limited (finite) number of points and then interpolate the results for the domain. Any continuous object has infinite degrees of freedom and it’s just not possible to solve the problem in this format. As mentioned earlier in the definition of FEM, it reduces the degrees of freedom from infinite to finite with the help of discretization that is meshing (Sanjay et al, 2008). Meshing can be classified as 1 dimensional, 2 dimensional and 3-dimensional element meshing.

1.5.4.1. 1-D element meshing

It is used when one of the dimensions is very large in comparison to the rest of the two. Element shape is the line and the element types are: rod, bar, beam, pipe, axisymmetric shell.

1.5.4.2. 2-D element meshing

It is used when two of the dimensions are very large in comparison to the third. Element shape are: quad, tria and the element types are: thin shell, plate, membrane, plane stress, plane strain, axisymmetric solid.

1.5.4.3. 3-D element meshing

It is used when all dimensions are comparable. Element shape are: tetra, hex, pyramid, penta and the element type is a solid.

1.5.5. FEM based procedure

In performing any FEA using a software, the ae three steps to follow.

1.5.5.1. Pre-processing

In the pre-processing stage, modelling of the element, meshing and application of boundary conditions are carried out. After completion of pre-processing that is CAD, Meshing and boundary conditions, the software internally forms mathematical equations of the form $[F] = [K][\delta]$.

1.5.5.2. Processing or Solution

In the processing stage, the software carries out matrix formations, inversion, multiplication and solution for unknown. It also finds strain and stress for static analysis.

1.5.5.3. Post-processing

In the post processing stage, results are being viewed, verifications are made and conclusions are arrived at. Modifications are considered about the possible steps that can be taken to improve the design.

Conclusion

In this chapter, we were interested in outlining the main constituent of reinforced concrete, the different design methods according to Eurocode. We also discussed on ductility relationships and classification according to different codes. For beam-column joints, it was possible to present the main features of joint, the mechanical properties and how the properties could be evaluated. Finally, we ended with the procedure on how to carryout finite element analysis using a software. The continuation of this work presents the different method used in order to achieve the objective of this work

CHAPTER 2: METHODOLOGY

Introduction

This chapter presents the methodology used in this work to evaluate the ductility of joint in reinforced concrete using finite element analysis. To attain the said objective, the chapter is divided into two main parts. In this work, the first part consists of site recognition, data collection, the norms used and the design procedure for most solicitated beam and column. Meanwhile the second part consist of procedure to model and calculate the ductility of a reinforced concrete beam-column connection using non-linear finite element analysis.

2.1. Site recognition

The site recognition will be carried out from a documentary research whose essential goal is to know the location of the site, the climate, the hydrology and socio-economic parameters in the region.

2.2. Site visit

The purpose of this activity is the building description results from the observation and the presentation of the use category, the dimension, the floor plans and elevation configuration.

2.3. Data collection

Several data are collected among which is the architectural data which gives the disposition of the different floor levels, beams, columns and stair. Meanwhile the bearing capacity of the soil is assumed.

2.4. Codes and standards

The norms that will be used for the design of elements are the

- Eurocode 0, basis of structural design
- Eurocode 1, actions on structure,
- Eurocode 2, design of concrete structures,
- Eurocode 8, design for earthquake resistance.

These European standards define the loads and the combination of loads for the design.

2.4.1. Loads

A structure can be subjected to a variety of load types and at the same time. The concern of this thesis is beam- column joints in RC structures, so the type loads applied to the chosen structure will be; permanent loads, and variable loads (imposed loads).

2.4.1.1. Permanent loads

These are actions acting during the whole nominal life of the structure with negligible time variation of their intensity (that can be considered as constant in time):

- Self-weight of structural elements (G1); self-weight of the soil, if present; forces due to the soil (excluding the effects of the service loads applied to the soil); forces due to water pressure (when they are constant in time)
- Self-weight of non-structural elements (G2); imposed displacements and deformations determined by the designer and realized in-situ
- Prestressing
- Shrinkage and creep (fluage)
- Differential displacements

Permanent action or load consist essentially of the weight of the element, whether structural or not. Provisions for the evaluation of the self-weight of these elements are given in Eurocode1.

2.4.1.2. Variable loads

Variable actions are those which, as the name goes, vary with respect to time. They consist of actions on the structure (or on the structural element) with instantaneous values which can be significantly different in time: That is, their magnitude is time dependent.

- With long duration: acting with a significant intensity, also if non-continuously, for a not negligible time compared to the nominal life of the structure;
- With brief duration: acting with brief duration compared to the nominal life of the structure; This variation is nonnegligible and monotonic. Variable loads fall under two main kinds; Imposed loads and seismic- induced loads.

2.4.2. Combination of actions

A combination of actions defines a set of values used for the verification of the structural reliability for a limit state under the simultaneous influence of different actions. In the case of a building, they are defined by:

- The fundamental combination, used for the Ultimate Limit State (ULS) associated with collapse or other similar forms of structural failure is:

$$\sum_{j\geq 1} \gamma_{G,j} G_{k,j} + \gamma_{Q;1} Q_{k,1} + \sum_{i>1} \gamma_{Q;1} \psi_{0.i} Q_{k,i} \qquad \text{(Expression. 2.1)}$$

Where the coefficients $\gamma_{G,\,j}$ and $\gamma_{Q,\,i}$ are partials factors which minimize the action which tends to reduce the solicitations and maximize the one which tends to increase it. The recommended values preconized by the Eurocode 0 for the structural and geotechnical (STR and GEO) verifications are:

$\gamma_{G,\,jsup} = 1.35$ and $\gamma_{G,\,jinf} = 1$

$\gamma_{Q,1,\,\mathrm{sup}} = 1.50$ and $\gamma_{Q,1,\,\mathrm{sup}} = 0$

$\gamma_{Q,\,i,\,\mathrm{sup}} = 1.50$ and $\gamma_{Q,\,\mathrm{i},\,\mathrm{inf}} = 0$

The characteristic combination (rare), used for non-reversible serviceability limit states (SLS) to be used in the verifications with the allowable stress method is the quasi permanent load combination as shown in expression 2.2

$$\sum_{j\geq 1} G_{k,j} + Q_{k,1} + \sum_{i>1} \psi_{0.i} Q_{k,i} \qquad \text{(Expression. 2.2)}$$

Where:

Ψ: is the combination factors that is function of the use category of the building. The recommended values by the Eurocode 0 are presented in the table A3 of the annex A.

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

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

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

2.5. Design steps

The static design is done based on the static analysis. Static analysis studies the behaviour of the structure under static loads application. The analysis starts with the modelling of the structural members. In that line, the concrete cover, the design and verification of one horizontal (beam) and one vertical (column) structural element, both considered as representative of the other elements of their type. A footing also analysed and designed in order to study the case of joints at the footing

2.5.1. Durability and concrete cover

To ensure the required design working life of the structure, it is necessary to protect each structural element against the environmental action. For concrete structures, Eurocode 2 ensured this protection by the definition of a concrete cover taking into account the structural class of the structure and the exposure class. This concrete cover is defined as the distance between the surface of the reinforcement closest to the nearest concrete surface and the nearest concrete surface as shown in the figure 2.1.

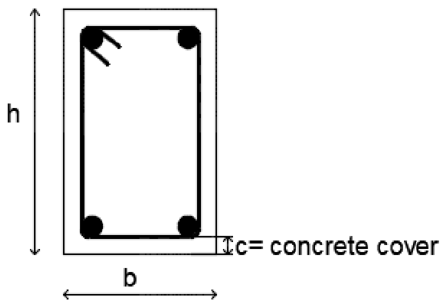


Figure 2. 1. Illustration of concrete cover

The nominal value of the concrete cover is defined as a minimum cover C_{min} plus an allowance in the design for deviation and it expressed by:

$$C_{nom} = C_{min} + \Delta C_{dev} \qquad \text{(Eq. 2.1)}$$

Where: $\Delta Cdev$ is the allowance in design for deviation with a recommended value of 10 mm.

The minimum cover C_{min} is defined in equation 2.2 as:

$$C_{min} = \max\left(C_{min,b};\ C_{min,dur} + \Delta C_{dur,\gamma} - \Delta C_{dur,st} - \Delta C_{dur\ add};\ 10mm\right) \qquad \text{(Eq. 2.2)}$$

Where:

$C_{min,b}$: The minimum cover due to bond requirement, equal to the diameter of the bars or the equivalent diameter in the case of bundled bars

$\Delta C_{dur,\gamma}$: The additive safety element with a recommended value of 0 mm

$\Delta C_{dur,st}$: Reduction of minimum cover for use of stainless steel

$\Delta C_{dur\ add}$: The add reduction of minimum cover for use of additional protection

$C_{min,\,dur}$: The minimum cover due to environmental conditions obtain from the table A4 of the annex A in function of the exposure and the structural class of the building.

2.5.2. Beam element design

The beam design is composed of an Ultimate Limit State (ULS) design and a Serviceability Limit State verification (SLS).

2.5.2.1. Ultimate Limit State Design

The ULS design of this element will be done for the bending moment and the shear force solicitations as there is not axial force inside the elements.

a. Bending moment design

For the envelope curve of bending moment solicitations obtained from the solicitation curves, each span is divided into three sections that is from 0 to L/4, L/4 to 3L/4 AND 3L/4 to L. The maximum bending moment M_{ED} at each section is used for the design.

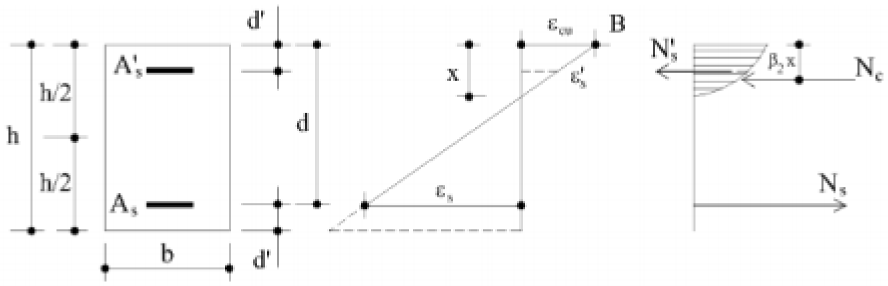


Figure 2. 2. Rectangular section at ultimate limit state (D'Antino et al, 2016)

From figure 2.2, the effective depth d is determined by equation 2.3

$$d = h - C_{nom} - \emptyset_s - \emptyset r/_2 \qquad \text{(Eq. 2.3)}$$

Where:

h: Is the height of the rectangular section

$\emptyset$s: Is the diameter of the stirrup

$\emptyset_{r/2}$: Is the radius of the longitudinal reinforcement

The calculation of the limit position of the neutral axis *Xlim* is determined by equation 2.4

$$X_{lim} = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{yd}}.d \qquad \text{(Eq. 2.4)}$$

Where:

ε_{cu}: Ultimate strain in concrete

ε_{yd}: Design yield strain in steel which is determined from equation 2.5

$$\varepsilon_{yd} = \frac{f_{yd}}{E_s} \qquad \text{(Eq. 2.5)}$$

Where:

E_s: Is the modulus of elasticity of longitudinal reinforcement

f_{yd}: Is the design yield strength of longitudinal reinforcement which is determined from equation 2.6

$$f_{yd} = \frac{f_{yk}}{\gamma_s} \qquad \text{(Eq. 2.6)}$$

Where:

γ_s: Material partial safety factor for steel

f_{yk}: Characteristic yield strength of longitudinal reinforcement

The limit resistant moment is determined by equation 2.7.

$$M_{Rd,lim} = \beta_1 . f_{cd} . X_{lim} . b f_{cd} (d - \beta_2 X_{lim}) \qquad \text{(Eq. 2.7)}$$

Where:

β_1: Is the filling ratio

β_2: Is the position of the centre of mass of the parabola

b: Is the width of the rectangular section

f_{cd}: Is the design concrete compressive strength which is determined from equation 2.8

$$f_{cd} = \frac{\alpha_{cu} f_{ck}}{\gamma_c} \qquad \text{(Eq. 2.8)}$$

Where:

γ_c: Material partial safety factor for concrete

α_{cu}: Material coefficient taking account of long-term effects on the compressive strength

f_{ck}: Characteristic compressive cylinder strength of concrete at 28 days

If $M_{Rd,lim} > M_{ED}$, the compressive reinforcement steel area A'_S is not required. The new position of the neutral axis is given by equation 2.9

$$X = \frac{d}{2\beta_2} - \sqrt{\left(\frac{d}{2\beta_2}\right)^2 - \frac{M_{ED}}{\beta_1.\beta_2.b.f_{cd}}} \quad \text{(Eq. 2.9)}$$

The tensile reinforcement steel area is given by equation 2.10

$$A_s = \frac{M_{ED}}{f_{yd}(d - \beta_2 X)} \quad \text{(Eq. 2.10)}$$

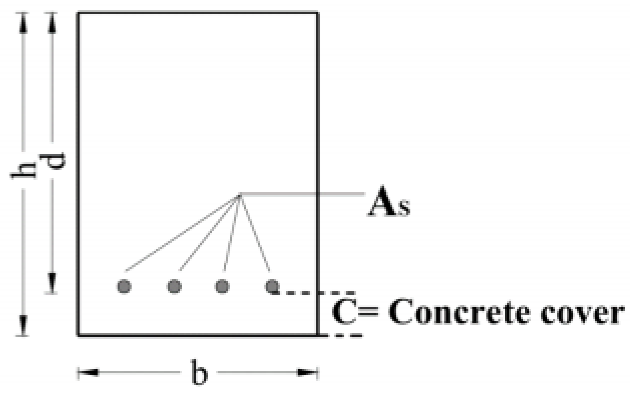


Figure 2. 3. Beam section with tensile reinforcement steel area only

If $M_{Rd,lim} < M_{ED}$, the compressive reinforcement steel area A'_S is required and it is given by equation 2.11

$$A'_S = \frac{\Delta M_{Esd}}{f_{yd}(d - d')} \quad \text{(Eq. 2.11)}$$

Where:

ΔM_{Esd}: Is the difference between the solicited moment and the resistive moment and is given by equation 2.12

$$\Delta M_{Esd} = M_{ED} - M_{Rd,lim} \quad \text{(Eq. 2.12)}$$

And the tensile reinforcement area is now given by equation 2.13

$$A_S = \frac{M_{Rd,lim}}{f_{yd}(d - \beta_2 X_{lim})} + A'_S \quad \text{(Eq. 2.13)}$$

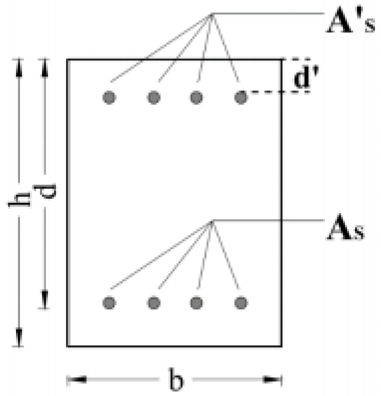


Figure 2. 4. Beam section with tensile and compressive reinforcement steel areas

Generally, in the case where $M_{Rd,lim} < M_{ED}$ that is compressive reinforcement steel area A'_S is not required, 50% of the tensile reinforcement steel area is usually taken for the compressive reinforcement steel area.

The reinforcement steel area obtained for both the tensile and compressive zone has to verify the detailing of beams prescribed by the Eurocode 2 which defines the minimum and the maximum reinforcement areas in the equation 2.14 and 2.15 respectively as:

$$A_{s,min} = \max(0.26\frac{f_{ctm}}{f_{yk}}b_t d; 0.0013 b_t d) \quad \text{(Eq. 2.14)}$$

$$A_{s,max} = 0.04A_c \quad \text{(Eq. 2.15)}$$

Where:

b_t: Is the mean width of the tension zone

d: Is the is the effective depth of the section

f_{ctm}: Is the tensile strength of the concrete

A_C: Is the area of the concrete section

The spacing between the longitudinal reinforcements is given in equation 2.16

$$S_b = \frac{b - 2C_{nom} - 2\emptyset_S - n\emptyset_r}{n-1} \quad \text{(Eq. 2.16)}$$

Where:

n: Is the number of longitudinal reinforcements

The spacing of the reinforcement steel obtained for both the tensile and compressive zone has to verify the detailing of beams prescribed by the Eurocode 2 which defines the minimum reinforcement steel spacing in equation 2.17 as:

$$S_{b,min} = \max\left(k_1 d_b, d_g + k_2, 20mm\right) \qquad \text{(Eq. 2.17)}$$

Where:

d_b: Diameter of the longitudinal reinforcement

d_g: Maximum diameter of aggregate

k_1: Constant which is equal to 1

k_2: Constant which is equal to 5mm

b. Shear verification

In order to take over the shear force inside the beam, transversal steel reinforcement has to be inserted inside the section as shown in the figure 2.5.

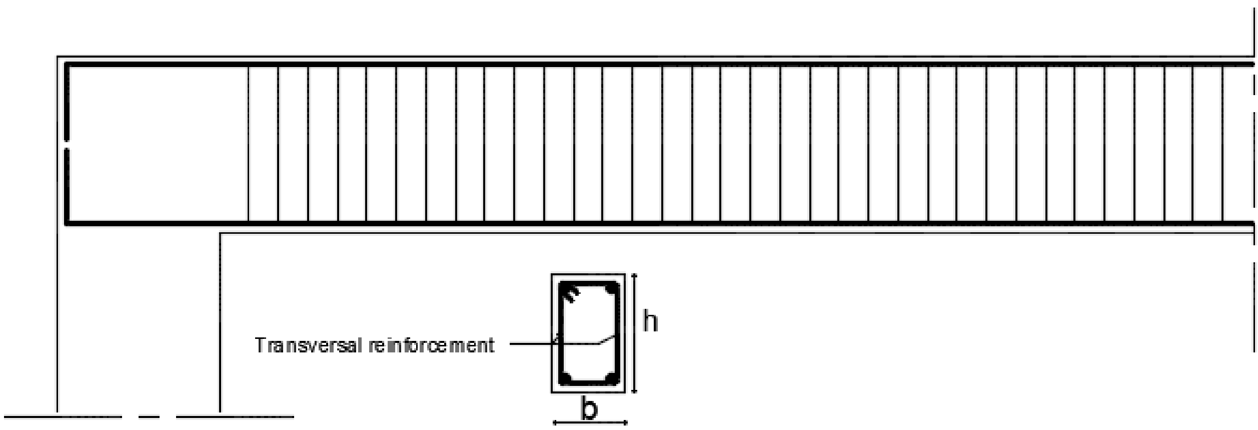


Figure 2.5. Longitudinal and transversal beam section with transversal reinforcement.

From the envelope curve of the shear solicitation, the necessity of the shear reinforcement is verified by comparing the acting shear V_{Ed} to the design shear resistance of the member without shear reinforcement $V_{Rd,c}$ which is defined by:

$$V_{RDC} = max\left\{\left[C_{rd,c}k(100\rho_l f_{ck})^{\frac{1}{3}} + k_1\sigma_{cp}\right]b_w d; (V_{min} + k_1\sigma_{cp})b_w d\right\} \qquad \text{(Eq. 2.18)}$$

Where:

f_{ck}: is the characteristic strength of the reinforcement

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

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

N_{Ed}: is the axial force in the cross section due to loading or prestressing (in N)

A_c: is the area of the concrete cross section

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

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

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

For members where the design shear reinforcement is required, the shear resistance is the minimum between V_{rds} and V_{rdmax} defined by the equations 2.19 and 2.20 respectively.

$$V_{rd,max} = \alpha_{cw} b_w z v_1 f_{cd} / (cot\theta + tan\theta) \quad \text{(Eq. 2.19)}$$

$$V_{Rd,s} = \frac{A_{sw}}{S} z f_{ywd} cot\theta \quad \text{(Eq. 2.20)}$$

Where:

f_{ywd}: is the design yielding strength of the shear reinforcement

v_1: is a reduction factor for concrete cracked in shear ($v_1 = 0.6$ for $f_{ck} \leq 60\ N/mm^2$)

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

$\alpha_{cw} = 1$ for non-prestressed structures.

θ: is the inclination of the cracks or the concrete struts.

A_{sw}: is the cross sectional area of the shear reinforcement with a maximum value given by the relation in equation 2.21 as:

$$\frac{A_{sw,max f_{ywd}}}{b_w S} \leq \frac{1}{2} \alpha_{cw} b_w v_1 f_{cd} \quad \text{(Eq. 2.21)}$$

The design shear reinforcement obtained has to verify the detailing of members. In the case of the beam, it defines the maximum longitudinal spacing of the shear assembly, the maximum transversal spacing of the legs in a series of shear link and the minimum shear reinforcement ratio as illustrated in the figure 2.6.

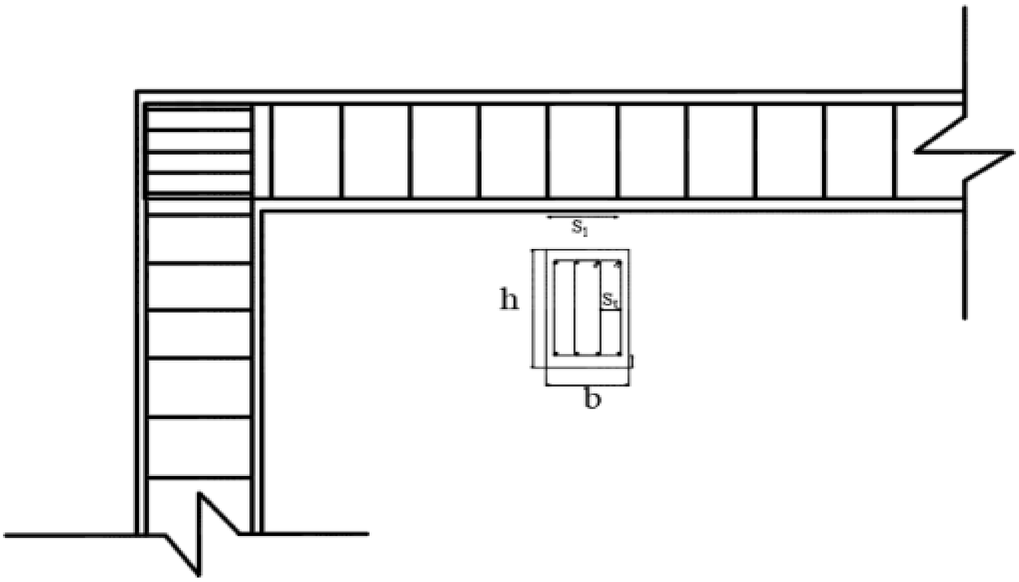


Figure 2.6. Illustration of the maximum longitudinal spacing and transversal spacing.

These limitations are given respectively in the equations 2.22, 2.23 and 2.24.

$$S_{l,max} = 0.75d(1 + cot\alpha) \quad \text{(Eq. 2.22)}$$

$$S_{l,max} = 0.75d \leq 600\ mm \quad \text{(Eq. 2.23)}$$

$$\rho_w, min = (0.08\sqrt{f_{ck}})/f_{yk} \quad \text{(Eq. 2.24)}$$

With the minimum shear reinforcement ratio computed as shown in equation 2.25 as:

$$\rho_w = A_{sw}/(s.b_w.sin\alpha) \quad \text{(Eq. 2.25)}$$

2.5.2.2. Serviceability Limit State Verification

The common serviceability limit states are the stress limitation, the crack and the deflection control. Only the stress limitation is presented on this work. The verification of the allowable stress on the beam is done using the characteristic rare combination because it permits to avoid inelastic deformation of the reinforcement and longitudinal cracks in concrete. The stress value is a function of the modular ratio in short terms and long terms expressed in equation 2.26 and 2.27 respectively:

$$n_o = \frac{E_s}{E_c} \qquad \text{(Eq. 2.26)}$$

$$n_\infty = n_o(1 + \varphi_L \times \rho_\infty) \qquad \text{(Eq. 2.27)}$$

Where φ_L = 0.55 for shrinkage of concrete and the parameter ρ_∞ = 2/2.5

The neutral axis position is computed for an uncracked concrete using equation 2.28.

$$x = \frac{-n(A_S + A'_S) + \sqrt{[n(A'_S + A_S)]^2 + 2bn(A_S d + A'_S d')}}{b} \qquad \text{(Eq. 2.28)}$$

Where A'_S and A_S, are the upper and lower steel reinforcement inside the section respectively. b, d' and d are the geometrical characteristics of the section presented in the figure 2.7.

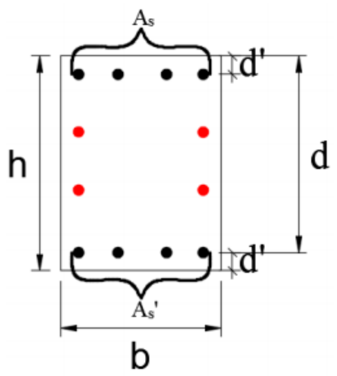


Figure 2.7. Transversal section of the beam with the different characteristics.

The moment of inertia of the uncracked section is given by equation 2.29 as:

$$J_{cr} = \frac{bx^3}{3} + nAs(d - x)^2 + nA's(x - d')^2 \qquad \text{(Eq. 2.29)}$$

The stress in the steel reinforcement and in the concrete are then obtained using the equation 2.30 and 2.31 respectively.

$$\sigma_s = \frac{M_{Ed}x}{J_{cr}} \qquad \text{(Eq. 2.30)}$$

$$\sigma_c = \frac{M_{Ed}(d - x)}{J_{cr}} \times n_\infty \qquad \text{(Eq. 2.31)}$$

The Eurocode 2 limitation of these stresses as presented in the equations 2.32 and 2.33.

$$\sigma_c \leq k_1 \times f_{ck} \qquad \text{(Eq. 2.32)}$$

$$\sigma_s \leq k_3 \times f_{yk} \quad \text{(Eq. 2.33)}$$

With $k_1 = 0.6$ and $k_3 = 0.8$

2.5.3. Column design

For the column design, a 3D modelling of the building in the software SAP2000 V22 will be done. Also, different loads arrangements will be considered to obtain the envelope curve for each solicitation. The pre-dimensioning is done and the design at ULS for the axial force, the bending moment and the shear force and the verification is done for the slenderness.

2.5.3.1. Column pre-dimensioning

The preliminary design of the column is done in two steps. The first step is based the axial loads resistance to determine the minimum area section and the second step on the modal analysis of the 3D model of the structure to verify the global dynamic behaviour.

a. Axial load resistance of the section

In a seismic area, the preliminary design of the column considers that 60% of the concrete resistance is used to take over the axial force. Then we can estimate the minimum area section of the column using equation 2.34.

$$N_{Rd} = 0.6 \times f_{cd} \times A_c \geq N_{sd} \quad \text{(Eq. 2.34)}$$

Where:

A_c: is the concrete section area;

N_{sd}: is the axial load computed using the recovery area of the column

The axial load is computed using equation 2.35:

$$N_{sd} = q \times S_r \times n \quad \text{(Eq. 2.35)}$$

Where:

q: is the uniform distributed loads on each floor computed at ULS;

S_r: is the recovery area of the column;

n: is the number of stories above the considered column

b. Modal analysis of the building

The modal analysis of the structure permits an estimation of the section of the vertical element through the verification of the vibration modes of the structure and the period of the first vibration. The structure is to be modelled with fixed support at the column base in contact with the foundations, so that a 30% participation of the total mass of the building can be assumed which will permits us to have the first two modes as translations and the third as torsion.

2.5.3.2. Bending moment-axial force verification

The envelope of the bending moment and the axial force solicitations obtained, the design is done through the M-N interaction diagram. For each level, we have to ensure that the maximum M-N solicitation belong to the M-N interaction diagram of the section considered. The interaction diagram is a diagram that shows all the limit situation that can determine the failure of the section. The points which are lying within the diagram represent the limit configuration, beyond them, failure occurs. This diagram is computed by determining some significant points. The procedure is presented below considering a rectangular section presented in the figure 2.8.

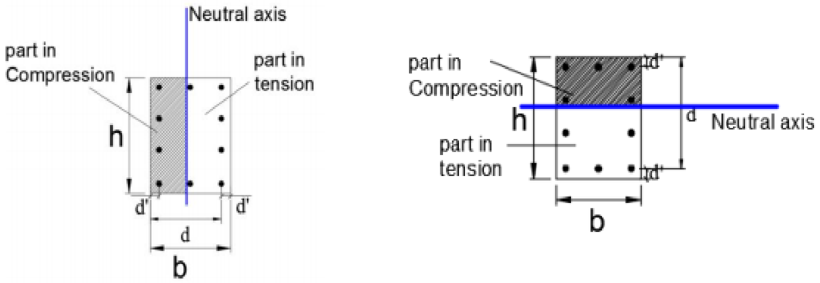


Figure 2.8. Rectangular section to illustrate the computation of the M-N diagram for different direction of the neutral axis (Djeukoua,2019).

a. First point

The section is completely subjected to tension; hence, the concrete is not reacting. We impose $\varepsilon_s = \varepsilon_{su}$, $\varepsilon_s{}' = \varepsilon_{syd}$ then the stress inside the element correspond to the design yielding strength of the steel reinforcement and the limit axial force and bending moment are obtained from the equations 2.36 and 2.37 respectively.

$$N_{Rd} = f_{yd}As + \sigma'_S A's \qquad \text{(Eq. 2.36)}$$

$$M_{Rd} = f_{yd}As\left(\frac{h}{2} - d'\right) - \sigma'_S A's\left(\frac{h}{2} - d'\right) \qquad \text{(Eq. 2.37)}$$

b. Second point

The section is completely subjected to tension. We impose: $\varepsilon_s = \varepsilon_{su}$, $\varepsilon_c = 0$. We should verify if the upper steel is yielded or not by determining the strain ε_s'. The limit axial force and bending moment is obtained from the equations 2.38 and 2.39 respectively.

c. Third point

We impose that the failure is due to concrete and the lower reinforcement is yielded. We assume $\varepsilon_s \geq \varepsilon_{syd}$, $\varepsilon_c = \varepsilon_{cu2}$ and we determine the neutral axis position. Then we should verify if the upper steel is yielded or not by determining the strain ε_s'. In order to determine the corresponding stress. The limit axial force and bending moment are obtained from the equations 2.38 and 2.39 respectively.

$$N_{Rd} = -\beta_1.b.x.f_{cd} + f_{yd}As - \sigma'_s A's \quad \text{(Eq. 2.38)}$$

$$M_{Rd} = \sigma'_s A's\left(\frac{h}{2} - d'\right) + f_{yd}As\left(\frac{h}{2} - d'\right) + \beta_1.b.x.f_{cd}(\frac{h}{2} - \beta_2 x) \quad \text{(Eq. 2.39)}$$

d. Fourth point

We impose that the failure is due to concrete and the lower reinforcement reaches exactly the yielding point, $\varepsilon_s = \varepsilon_{syd}$. As for the previous point, we determine the neutral axis position and the strain ε'_s. The limit value of the axial force and the bending moment is determined using the equations 2.38 and 2.39 respectively.

e. Fifth point

We impose that the failure is due to concrete and the lower reinforcement reaches exactly $\varepsilon_s = 0$ then the neutral axis position is equal to the effective depth of the section. The limit axial force and bending moment are obtained from the equations 2.40 and 2.41

$$N_{Rd} = -\beta_1.b.x.f_{cd} - \sigma'_s A's \quad \text{(Eq. 2.40)}$$

respectively.

$$M_{Rd} = \sigma'_s A's\left(\frac{h}{2} - d'\right) + \beta_1.b.d.f_{cd}(\frac{h}{2} - \beta_2 x) \quad \text{(Eq. 2.41)}$$

f. Sixth point

We impose that the section is uniformly compressed. We assume $\varepsilon_s = \varepsilon_c \geq \varepsilon_{c2}$. The limit axial force and bending moment is obtained from the equations 2.42 and 2.43 respectively.

$$N_{Rd} = -b.h.f_{cd} - \sigma'_s A's - f_{yd}As \qquad \text{(Eq. 2.42)}$$

$$M_{Rd} = \sigma'_s A's\left(\frac{h}{2} - d'\right) - f_{yd}As\left(\frac{h}{2} - d'\right) \qquad \text{(Eq. 2.43)}$$

An example of M-N diagram is presented in the figure 2.9. The blue point represents a couple of solicitation M_{Ed} and N_{Ed} which lies internally to the diagram hence the section is considered safe for those actions.

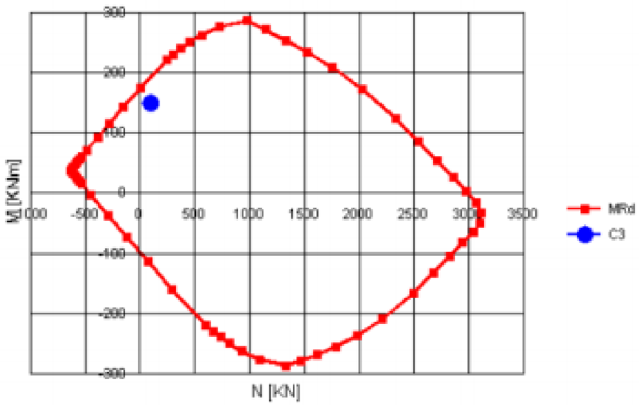


Figure 2.9. Example of M-N Diagram (D'Antino et al,2016).

The steel reinforcement of the column is considered taking into account the limitations of the Eurocode 2 defined by equation 2.44 and 2.45 for the minimum and maximum steel section respectively.

$$A_{s,min} = \max(\frac{0.10N_{Ed}}{f_{yd}}; 0.002Ac) \qquad \text{(Eq. 2.44)}$$

$$A_{s,max} = 0.04Ac \qquad \text{(Eq. 2.45)}$$

Where:

N_{Ed}: is the design axial compression force

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

For joint reinforcements:

$$A_s = 0.09sb_{ch}\frac{f_c'}{f_{yh}} \qquad \text{(Eq. 2.46)}$$

And not less than:

$$A_{sh} = 0.3sb_{ch}\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{f_{yh}} \quad \text{(Eq. 2.47)}$$

2.5.3.3. Shear verification

The verification procedure is the same for the beam. The detailing of members prescribed by the Eurocode 2 imposed a minimum diameter of 6 mm or one quarter the maximum diameter of the longitudinal bars. The maximum spacing of the transverse reinforcement is given by the equation 2.48 as:

$$S_{cl,max} = \min(20\phi_{l,min}; b; 400mm) \quad \text{(Eq. 2.48)}$$

Where:

$\phi_{l,min}$: is the minimum diameter of the longitudinal bars

b: is the lesser dimension of the column

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

2.5.3.4. Slenderness verification

The slenderness verification permits to know if we have to consider the second order effect or not. It consists in verifying if the slenderness of the element is below a limit value, defined by the Eurocode 2 as shown in equation 2.49.

$$\lambda_{lim} = 20.A.B.C/\sqrt{n} \quad \text{(Eq. 2.49)}$$

Where:

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

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

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

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

The slenderness of an element is evaluated using the equation 2.50.

$$\lambda = l_o/i \quad \text{(Eq. 2.50)}$$

Where:

Lo: Is the effective length of the element(lo=0.5l)

i: Is the gyration radius of the uncracked concrete section and is given by equation 2.51.

$$i = \sqrt{\left(\frac{I}{A}\right)} \qquad \text{(Eq. 2.51)}$$

Where I is the moment of inertia and A is the area of the section.

2.6. Interface of Abaqus/CAE

ABAQUS is a finite element calculation software package developed by ABAQUS, Inc (Dassault Systems). It consists of three products: ABAQUS/Standard, ABAQUS/Explicit and ABAQUS/CAE. It is the latter product that is used in our work. ABAQUS/CAE in particular, are written entirely in C++, Fortran for the calculation parts and Python for the scripts and parameterizations. The graphical interface is managed by FOX Toolkit. It provides an integrated visualization and modelling interface. Abaqus/CAE is a complete ABAQUS environment that provides a simple, consistent interface for creating, submitting, monitoring, and evaluating results from Abaqus/Standard and Abaqus/Explicit simulations. ABAQUS/CAE is divided into modules, where each module defines a logical 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 the ABAQUS/Standard or ABAQUS/Explicit analysis product. The analysis product performs the analysis, sends information to Abaqus/CAE to allow you to monitor the progress of the job, and generates an output database. Finally, you use the Visualization module of ABAQUS/CAE to read the output database and view the results of your analysis. The interface of Abaqus/CAE is shown in the figure 2.10.

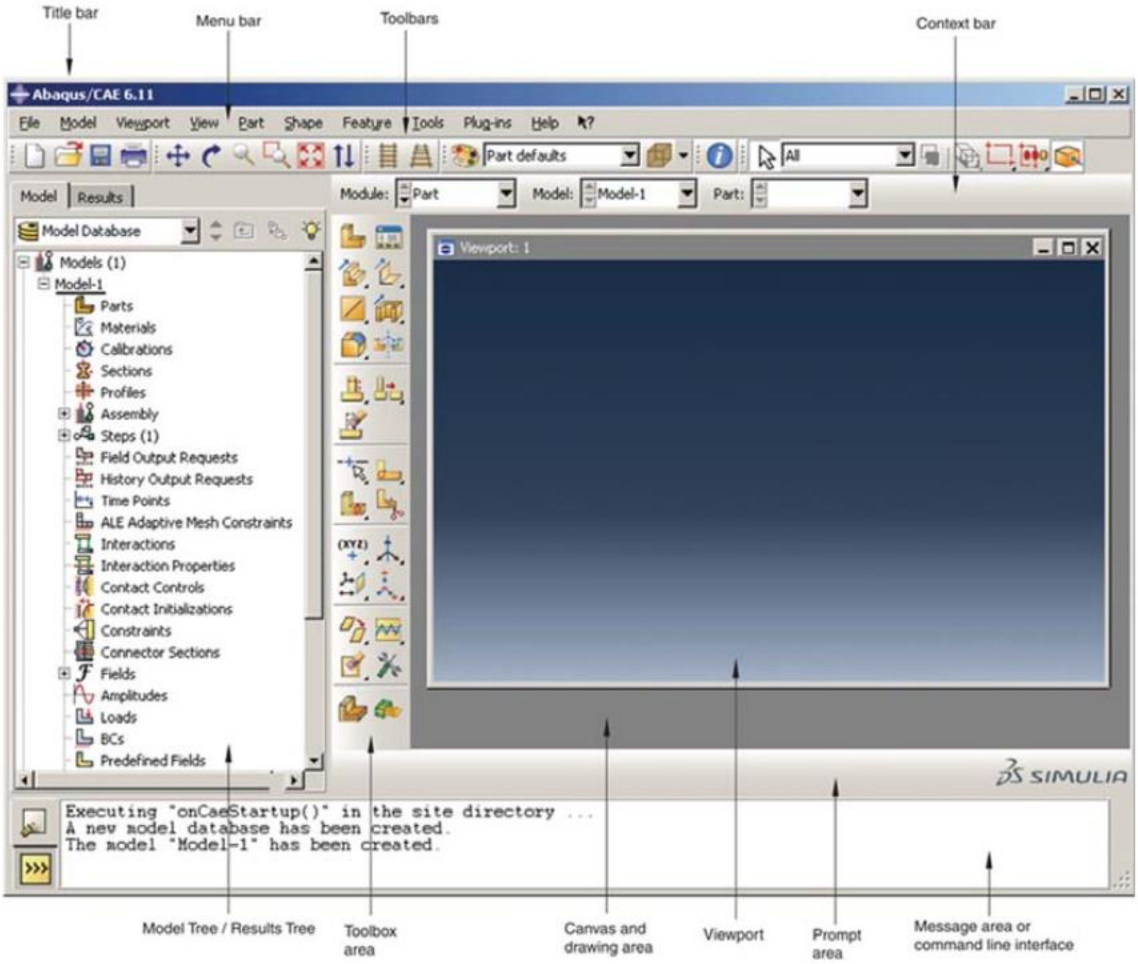


Figure 2. 10. Component of the main window (Abaqus manual, 2014)

2.7. Modelling procedure of joint using finite element analysis software

Joint analysis involves the three phases which are the pre-processing phase, the processing phase and the post-processing

2.7.1. Pre-processing stage

This is the stage during which every input detail information including the section geometry, material properties, loads and boundary conditions and the type of mesh are defined. This involves five main steps.

- First step: The first step involves creating the parts (geometry). The concrete, the longitudinal reinforcements and stirrups are defined after creating the geometry by grouping them into different sets. This helps in the subsequent steps in sorting out the different parts of the model to assign properties.

- Second step: The next step is to define the material properties of the different parts. That is for the concrete, the density, elastic, concrete damaged plasticity, concrete compression damage and concrete tension damage properties are defined meanwhile for the rebars, the density, elastic and plastic properties are also defined.
- Third step: This step involves assembling the parts and creating the step in which static general is chosen for the analysis.
- Fourth step: In this step, the boundary conditions are defined at the column edges, the secondary beam edges and the shorter span of the principal beam edge preventing rotation and displacement in the initial step meanwhile a displacement is given to the other edge of the principal beam in step 1.
- Fifth step: in this step the model is meshed and a job is created. Each part is meshed as a dependent instance and the is submitted.

2.7.2. Processing stage

In this stage, the software program solves the unknowns assigned in the pre-processing phase. The reaction forces at the base of the column and the displacements along the principal beam are what are required in the time history analysis.

2.7.3. Post processing

The final stage is the post processing stage. In this stage, engineering judgements are required. Based on the results that the processing stage provide, the analysis and reliability of the results are determined. The reaction forces at the base of the column are gotten and plotted against the displacement of the principal beam.

Conclusion

The aim of this chapter was to describe the methodology used to do the structural calculation in order to have the correct beam, column and reinforcement sections for the most solicitated beam and column element. The methodology also comprised of the process of modelling the beam -column joint on Abaqus right up to the stage of obtaining results for the calculation of ductility. The chapter began with site recognition, data collection, the norms used and also the procedure of design used. It was then followed by a brief description of the finite element analysis software and finally the modelling methodology of the reinforced concrete joint using the finite element analysis software.

CHAPTER 3: ANALYTICAL DESIGN, NUMERICAL MODELLING AND RESULTS

Introduction

In this chapter, the methodology presented in the previous chapter is carried out on the case study according to specifications and rules of Eurocode Standard EN 1992-1-1:2004 to estimate the cross section, longitudinal and transversal reinforcement and materials specifications of vertical and horizontal reinforced concrete elements which are needed to model the joint and run the analysis on Abaqus. The chapter just like the previous one is divided into two parts. The first part consists of presentation of the case study in which we have the description of the case study, material properties, structural modelling, actions on the building with load combination and static design meanwhile the second part consists of modelling the joint and running the analysis on Abaqus in order to generate results, interpret them and at the end calculate the ductility.

3.1. General presentation of site

The investigated building is found in the centre of Yaoundé, Cameroon. Yaoundé is the administrative capital of Cameroon. It has a tropical climate with two main seasons: the dry season and the rainy season. The average annual temperature is 23.7°C and the average annual precipitation is 1643 mm. The city has area of 308 km² for a population of about 3.5 million inhabitants.

3.2. Presentation of the case study

3.2.1. Description of case study

The case study, is a three floor-story reinforced concrete building frame for offices and classrooms use at the National Advanced School of Public Works Yaoundé as shown in Figure 3.1. The formwork view is shown in figure 3.2 and the sections are shown in figure 3.3 and 3.4. It is a rectangular floor, with its length being 24.5m and the width 11.45m. The slab is assumed to be a reinforced concrete slab with hollow blocks of thickness 20cm. The building is 10.1m tall from the ground level, with the height of the ground floor being equal to 3.7m, the first floor being equal to 3.4m and the second 3.0m. The building is regular in plan and in elevation without setback. The floors are of the same distribution, but with floor levels of unequal heights. Columns on the same floor do not have the same sections but each section is being maintained at each floor level. The ground level columns are at a height greater than that of columns at subsequent levels. Beams on the same floor do not have the same design. The secondary beams are of different sections but are almost of the same span length of 4.6m except for one which has a span length of

1.2m. Meanwhile the principal beams are neither of the same span lengths nor of the same sections but are group into two with each group having the same span length and the same section. The structure has identical floor plan as from level one to level three as shown in the section view of Figure 3.2.

Figure 3. 1. NASPW Block H building front view

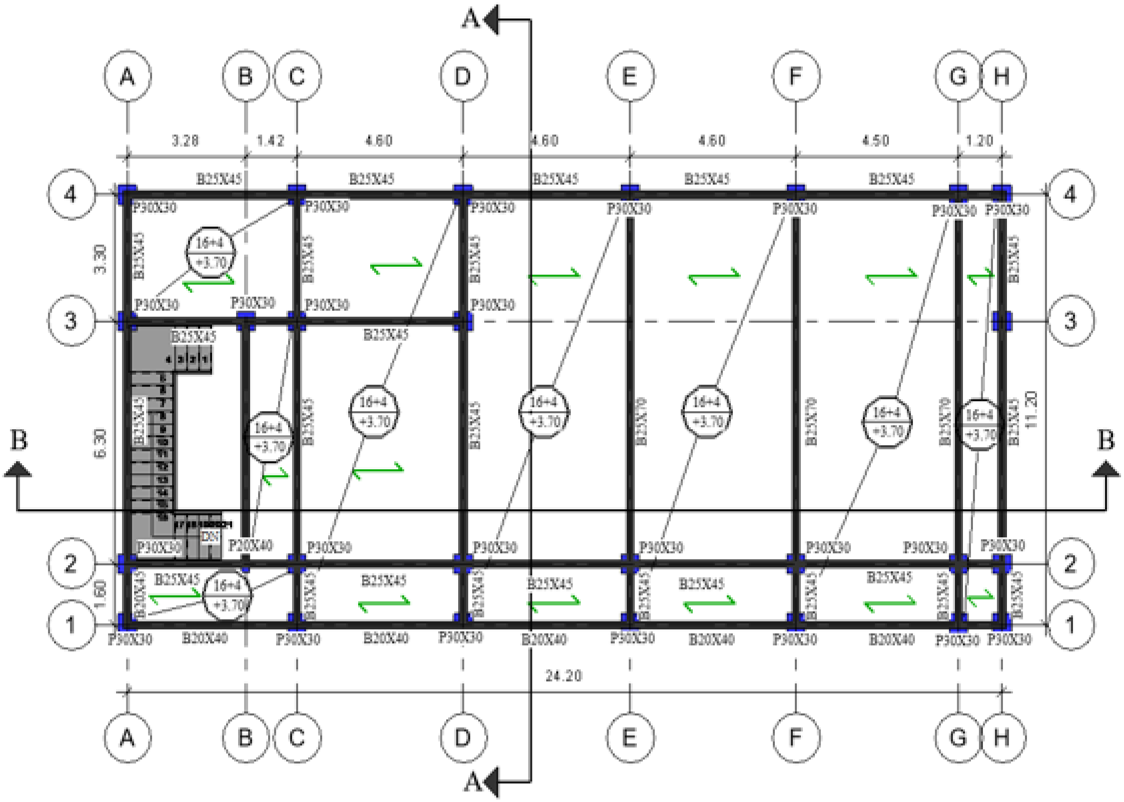


Figure 3. 2. Formwork plan of the building

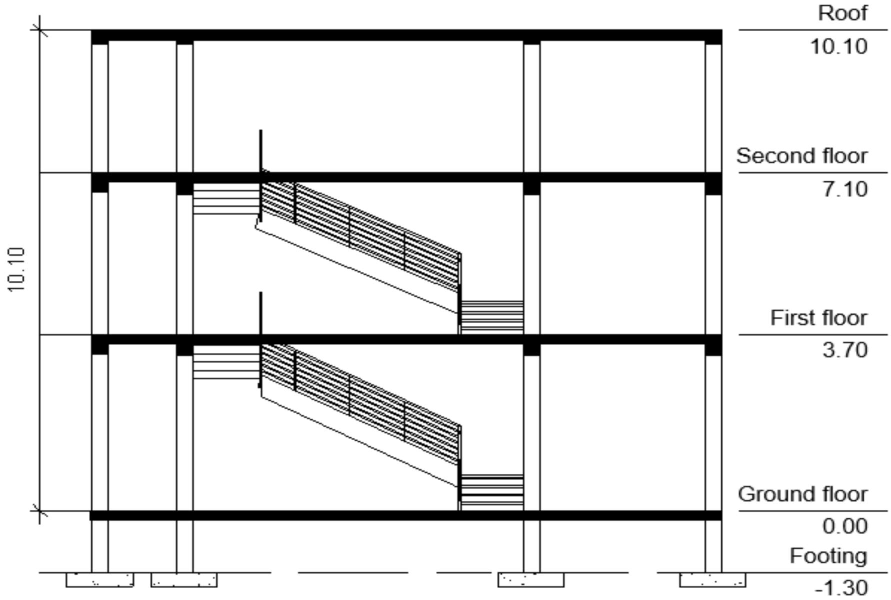


Figure 3. 3 Section A-A of the building

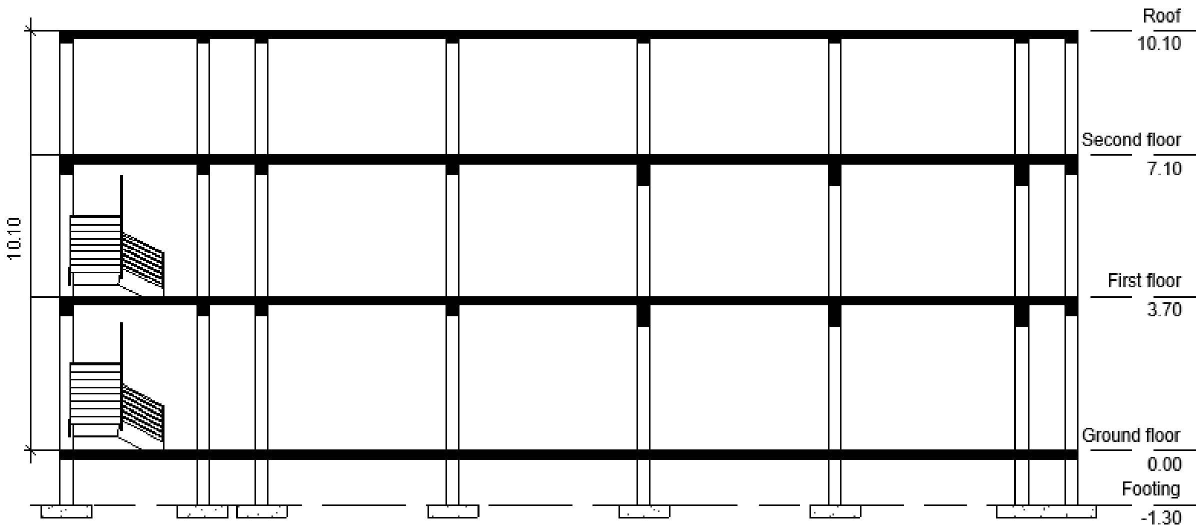


Figure 3. 4. Section B-B of the building

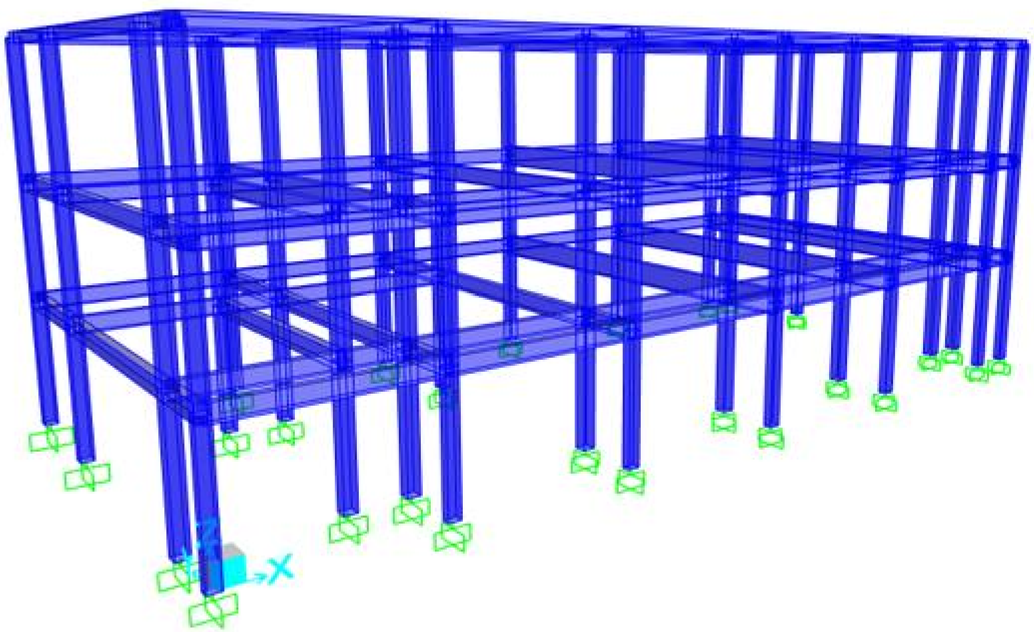


Figure 3. 5. 3D model of the building

3.2.2. Material properties

The concrete class chosen is C25/30 and the longitudinal steel reinforcement is FEA400. We consider a characteristic yield strength of 235 MPa for the transversal reinforcement. Table 3.1 below shows the main characteristics of concrete and Table 3.2 that of steel used as reinforcement for linear analysis and design of the structure.

Table 3. 1. Concrete characteristics

Property	Value	Unit	Definition
Class	C25/30	-	Concrete class
R_{ck}	30	MPa	Characteristic cubic compressive strength at 28days
f_{ck}	25	MPa	Characteristic cylindrical compressive strength of concrete at 28days
$f_{cm} = f_{ck} + 8$	33	MPa	Mean value of concrete cylindrical compressive strength
γ_c	1.5	-	Partial safety factor
$f_{ctm} = 0.3\ x\ (f_{ck})^{\frac{2}{3}}$	2.56	MPa	Mean value of axial tensile strength of concrete
$f_{ctd} = 0.7\ x\ \left(\frac{f_{ctm}}{\gamma_c}\right)$	1.2	MPa	Design resistance in traction
$E_{cm} = 22000\ x\ \left(\frac{f_{cm}}{10}\right)^{0.3}$	31476	MPa	Secant modulus of elasticity
v	0.2	-	Poisson's ratio
G	13115	MPa	Shear modulus
γ	25	KN/m^3	Specific weight of concrete

Table 3. 2. Longitudinal reinforcement characteristics

Property	Value	Unit	Definition
Class	FEA400	-	Steel class
f_{yk}	400	MPa	Characteristic yield strength
γ_s	1.15	-	Partial safety factor for steel
γ	78.5	MPa	Specific weight of steel
v	0.3	-	Poisson's ratio

3.2.3. Structural modeling

SAP 2000 V22, a structural analysis software used for the modelling and design of the case study. The building as mentioned is a three-story building for office and classroom use. The slab loads are applied directly to the beams as distributed loads. In the same way is applied the loads of the walls (exterior walls) as they are considered being carried by the slab. Beams and columns are modelled as frame elements having their connections (beam- column joints) ensured by the insertion of joints between two or more elements. To ensure rigidity of every floor above ground level, a diaphragm constrain is assigned to each node of the structure from the first story to the roof.

3.2.4. Actions on the building

The building is subjected to vertical (gravity) loads. The loads are either permanent or variable and are combined in various combinations in order to study the various effects and to determine the unfavorable (worst loading case) situation.

3.2.4.1. Permanent action

There are two categories of permanent loads acting on the structure which are the permanent structural loads and the permanent non-structural load. Both are presented in Table 3.3 and Table 3.4.

Table 3. 3. Structural load on beams

Nature	Description	Value	Unit
G_{k1}	Hollow block slab	2.85	kN/m^2

Table 3. 4. Non-structural load on beams

Nature	Description	Value	Unit
G_{k2}	Tiles	0.22	kN/m^2
G_{k2}	Coating under slab (1.25cm thick)	0.22	kN/m^2
G_{k2}	Mortar above slab (5cm thick)	1.25	kN/m^2
G_{k2}	Electrical and plumbing elements	0.5	kN/m^2
G_{k2}	Wall partition	1.0	kN/m^2

$Total\ G_{k2}$	3.19	kN/m^2

3.2.4.2. Variable actions and load combinations

The building, because of its function is classified as category B building for which the imposed load is in the range of 2.0 to 3.0kN/m^2. In this work, we consider an imposed load of 2.0kN/m^2.

3.2.5. Load combinations

The load combination in the equation 3.1 provides for the verification of the structure at the Ultimate Limit State (ULS).

$$q = 1.35G_k + 1.5Q_k \qquad \text{(Eq. 3.1)}$$

$$G_k = G_{k1} + G_{k2} \qquad \text{(Eq. 3.2)}$$

For non-reversible Serviceability Limit State (SLS), the verification is done using equation 3.3

$$q = G_k + Q_k \qquad \text{(Eq. 3.3)}$$

3.3. Static design

Static design is done for vertical static actions on the building. This implies considering only permanent and imposed loads. The procedure goes by selecting a horizontal and a vertical element of the structure for their respective designs.

3.3.1. Durability and element concrete cover

Considering a concrete structural class S4 and an exposure class XC1 together with the provisions of Eurocode 2 outlined in section 2.2.1, the concrete cover obtained by:

$$C_{min} = \max(16; 15; 10) = 16\text{mm} \qquad \text{(Eq. 3.4)}$$

From Eq. 2.3, $C_{nom} = 16 + 10 = 26mm$

We will consider a minimum concrete cover of 30mm in the design situations.

3.3.2. Beam design

3.3.2.1. Preliminary design

The principal and secondary beams constitute the horizontal structural elements.
Principal beams are those which support the slab and transfer the loads to the columns.
The choice of the beams under study is the shaded beam on grid E-E as shown in figure 3.6.

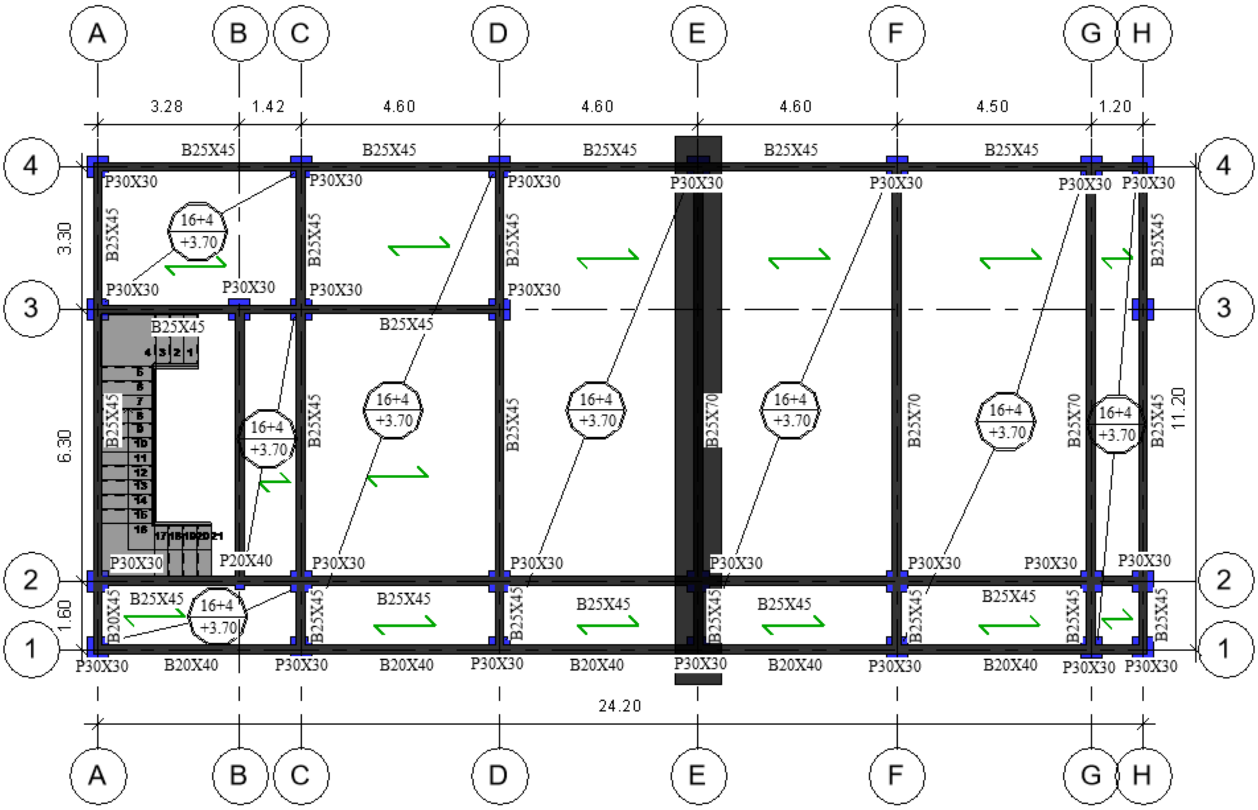


Figure 3. 6. Selected beam for design

The beam is designed following the two mechanical schemes as shown in figure 3.7 a and b

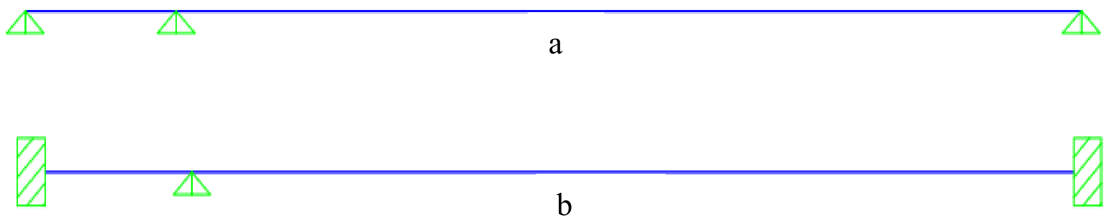


Figure 3. 7. Mechanical scheme for beam design

For the design of the horizontal elements shown in figure 3.5, four load combinations are considered from which design solicitation parameters are determined. The load arrangements are shown in figure 3.8.

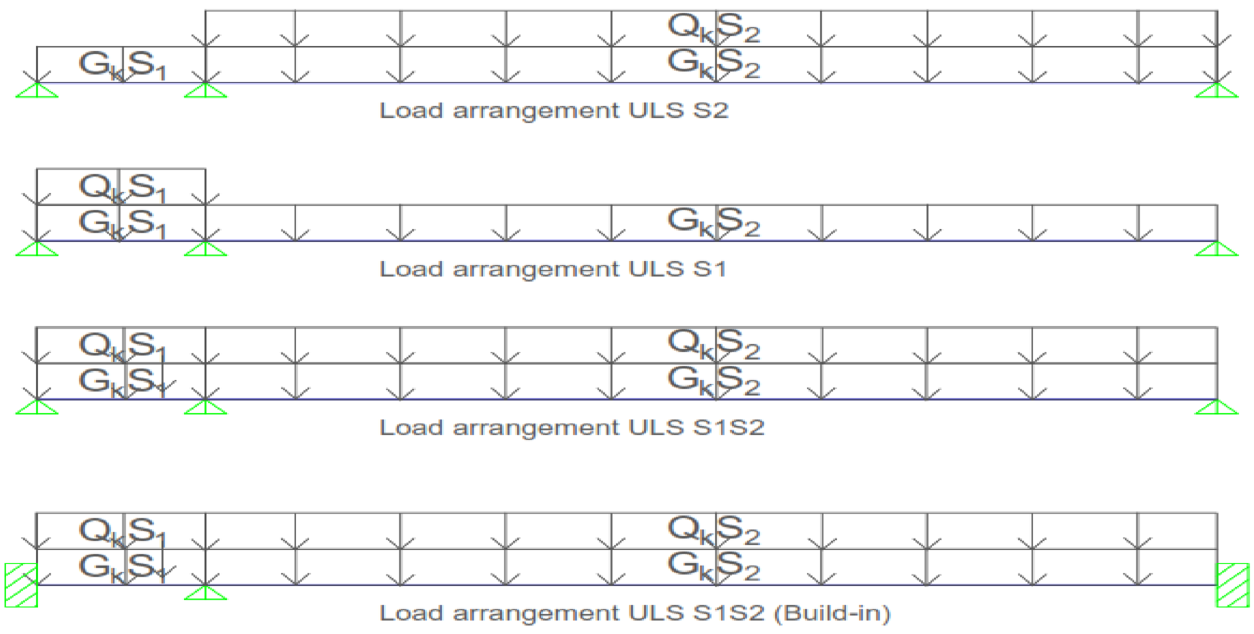


Figure 3. 8. Load combination on the beam

The preliminary design is done by considering that a span is simply supported at both ends, which implies h ≥ L/14 and 0.3h ≤ b ≤ 0.5h.

Where:

L: The longest span of the beam

b: Width of the beam

h: Height of the beam

Considering the longest span of the beam which equal 9.6, it implies

h = 0.70m and b = 0.25m.

The dimensions b and h are the geometric characteristics of the beam section. Henceforth, we proceed the modelling and the design of the horizontal structural element with the use of SAP 2000 V22 and Excel. End supports of the beam are modelled with simple support for maximum positive bending moment at mid-spans and fixed supports for maximum negative bending moment as shown in figure 3.7.

The analysis from SAP 2000, V22 generates results which present the solicitation parameters at ULS and SLS for respective verification.

3.3.2.2. Ultimate Limit State

The curves in figure 3.9 and figure 3.10 show the solicitations for the bending moment and the shear forces respectively for the beam obtained from the results of the analysis performed in SAP 2000 V22 software.

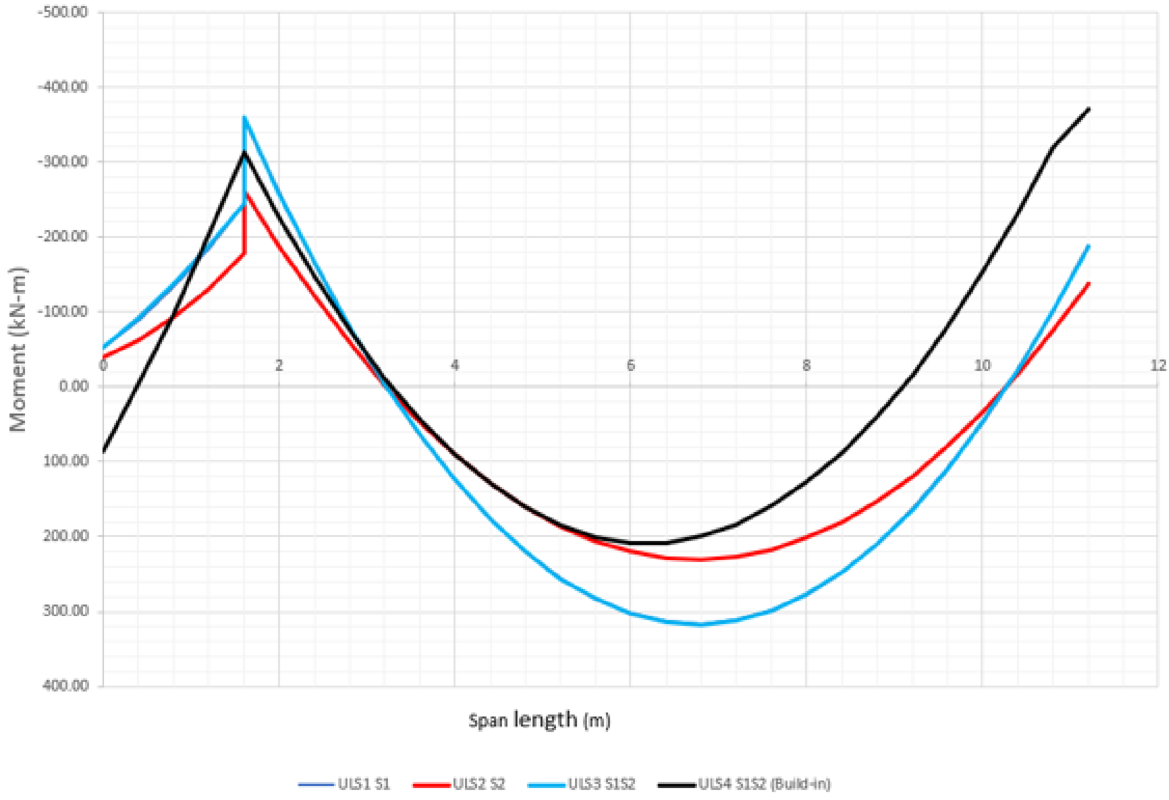


Figure 3. 9. Bending moment solicitation curves of the beam

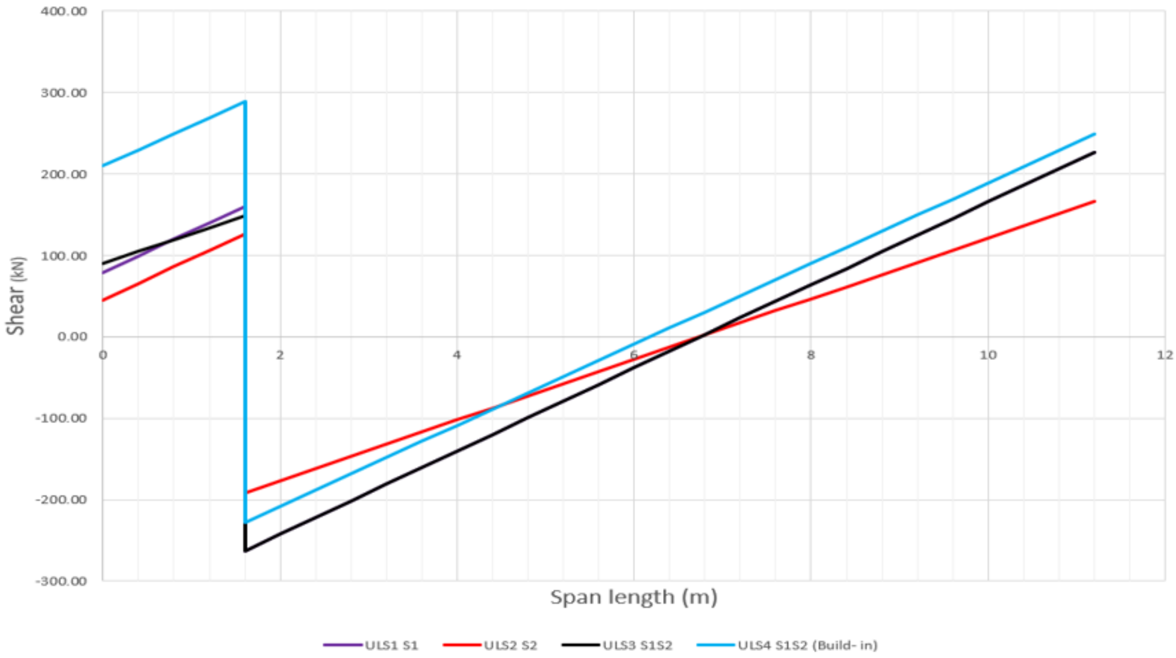


Figure 3. 10. Shear solicitation curve of the beam

Figure 3. 11. Envelope curve of bending moment at ULS

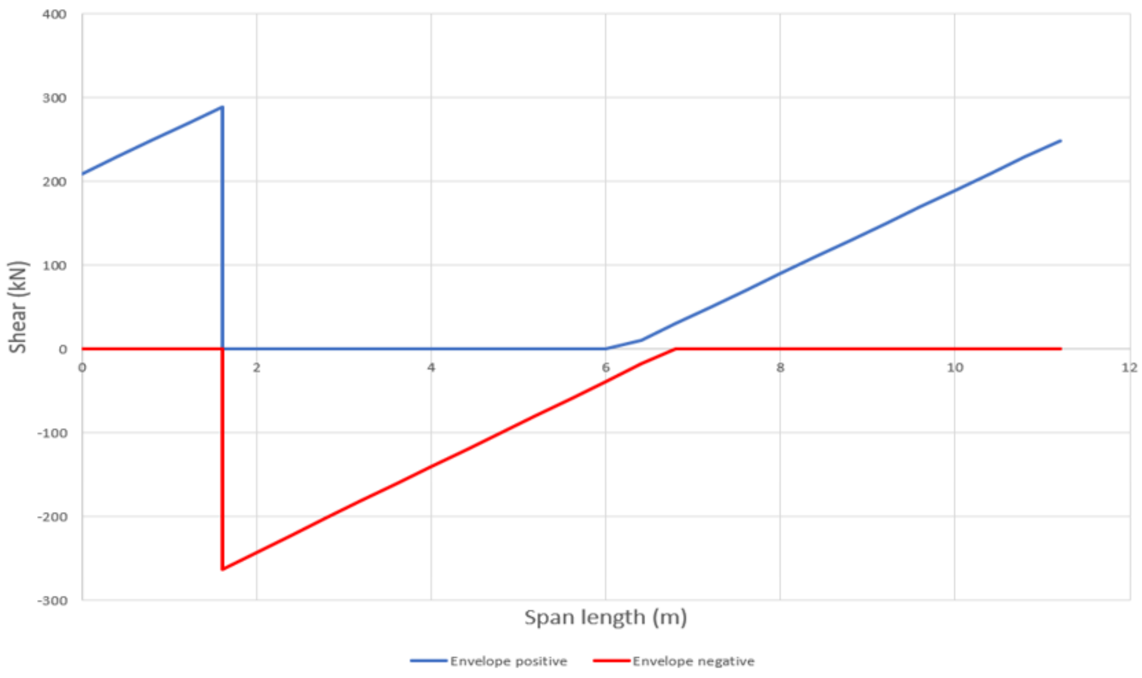


Figure 3. 12. Envelope curve of shear at ULS

The steel reinforcements are computed using equation 2.11. The verifications are performed for the detailing of the horizontal structural element using equations 2.14 and 2.15. Finally, the steel section is verified for a beam section of 250 x 700mm. Figure 3.13 shows the reinforcements obtained from the computations.

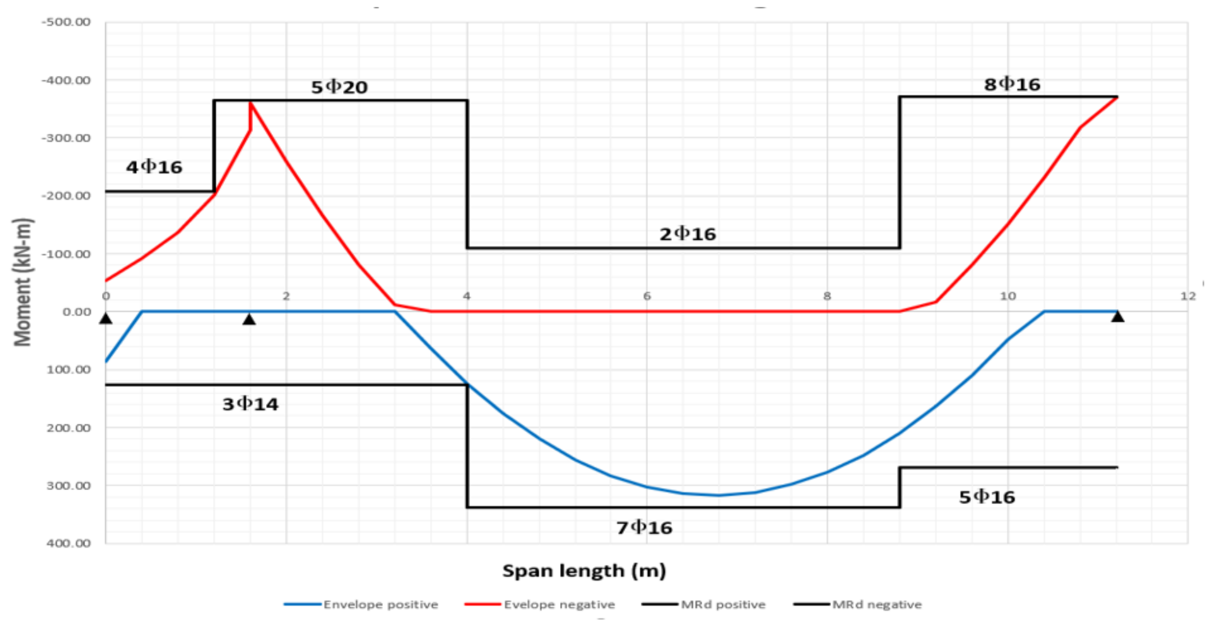


Figure 3. 13. Recapitulative curve of bending moment verification of the beam

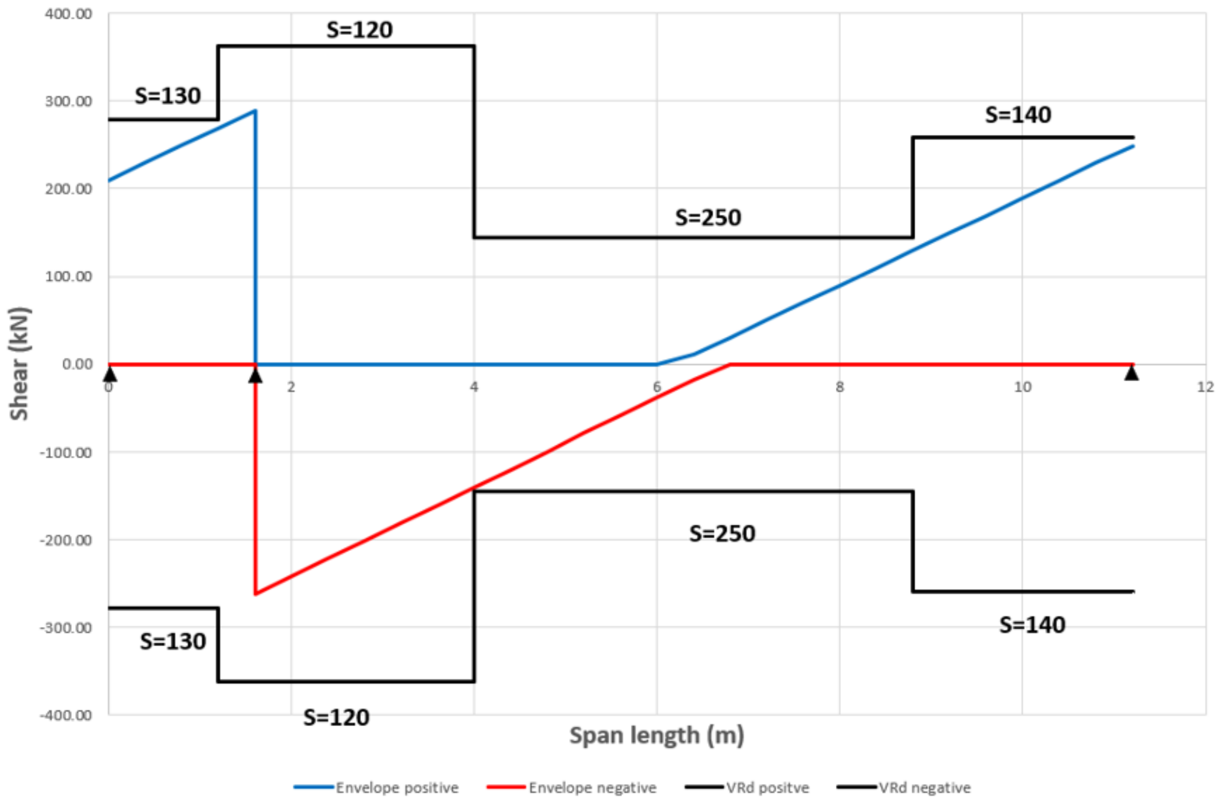


Figure 3. 14. Recapitulative curve for shear verification of the beam

3.3.2.3. Serviceability limit state

The four different load arrangements of the beam using the rare combination, the solicitation curves at SLS are obtained as shown in figure 3.15

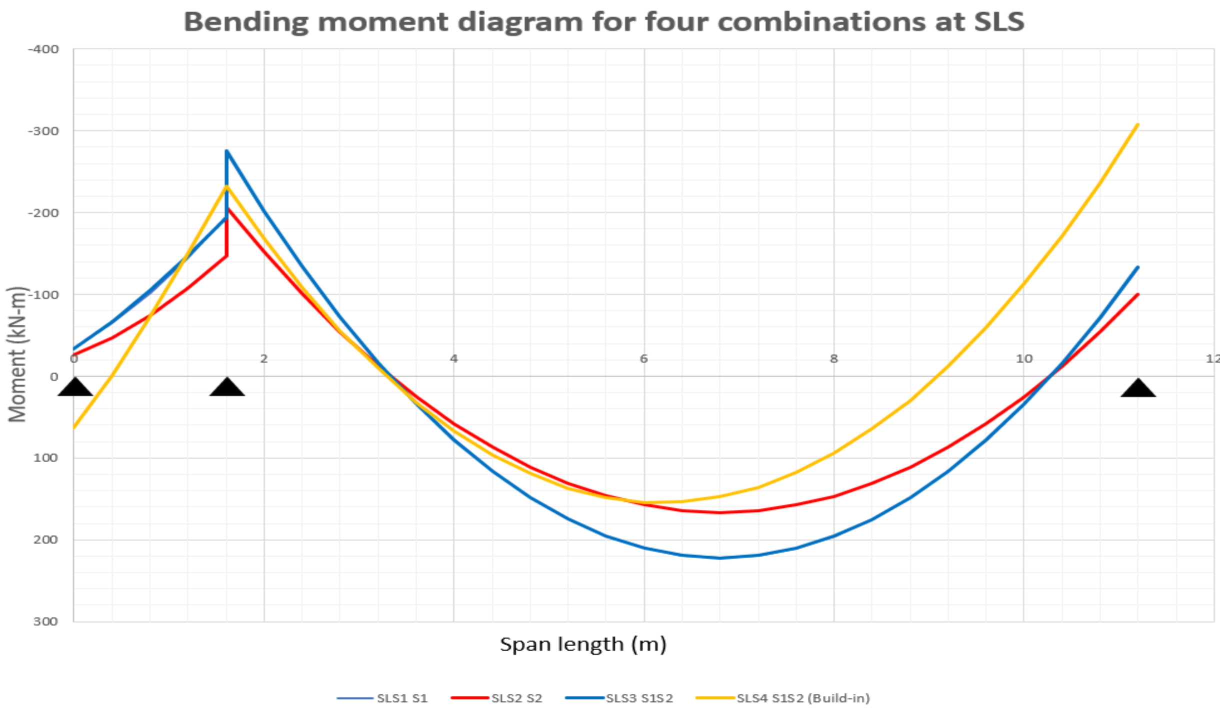


Figure 3. 15. Bending moment curves of the beam at SLS

The solicitations permit to have an envelope curve as shown in figure 3.16

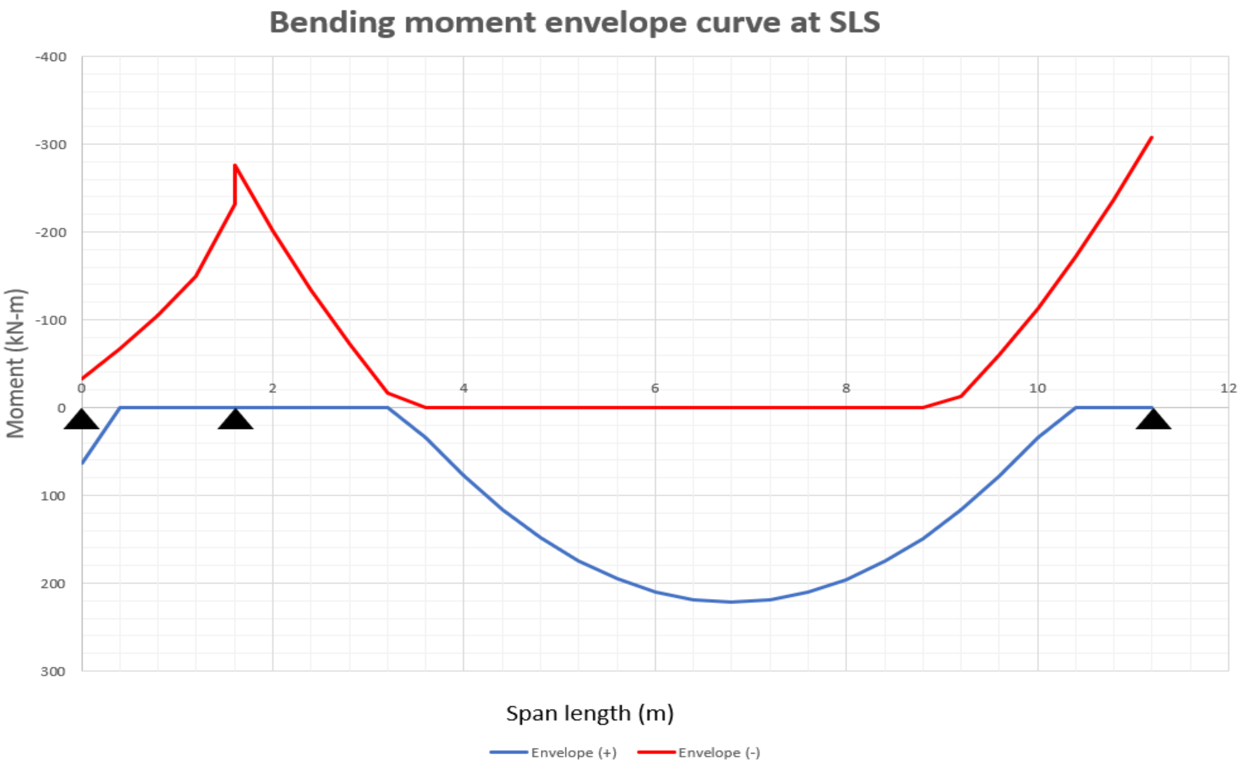


Figure 3. 16. Envelope solicitation curve of bending moment at SLS

With this envelope curve for bending moment at serviceability limit state the stress in the concrete and in the reinforcement are obtained using the equations 2.30 and 2.31. The limit value on the stress is evaluated from the equations 2.32 and 2.33 using the recommended values of the Eurocode 2, which are taking as $k1 = 0.6$ and $k3 = 0.8$. Figure 3.17 shows a comparison of the stress inside the concrete and the steel reinforcement to the admissible stress

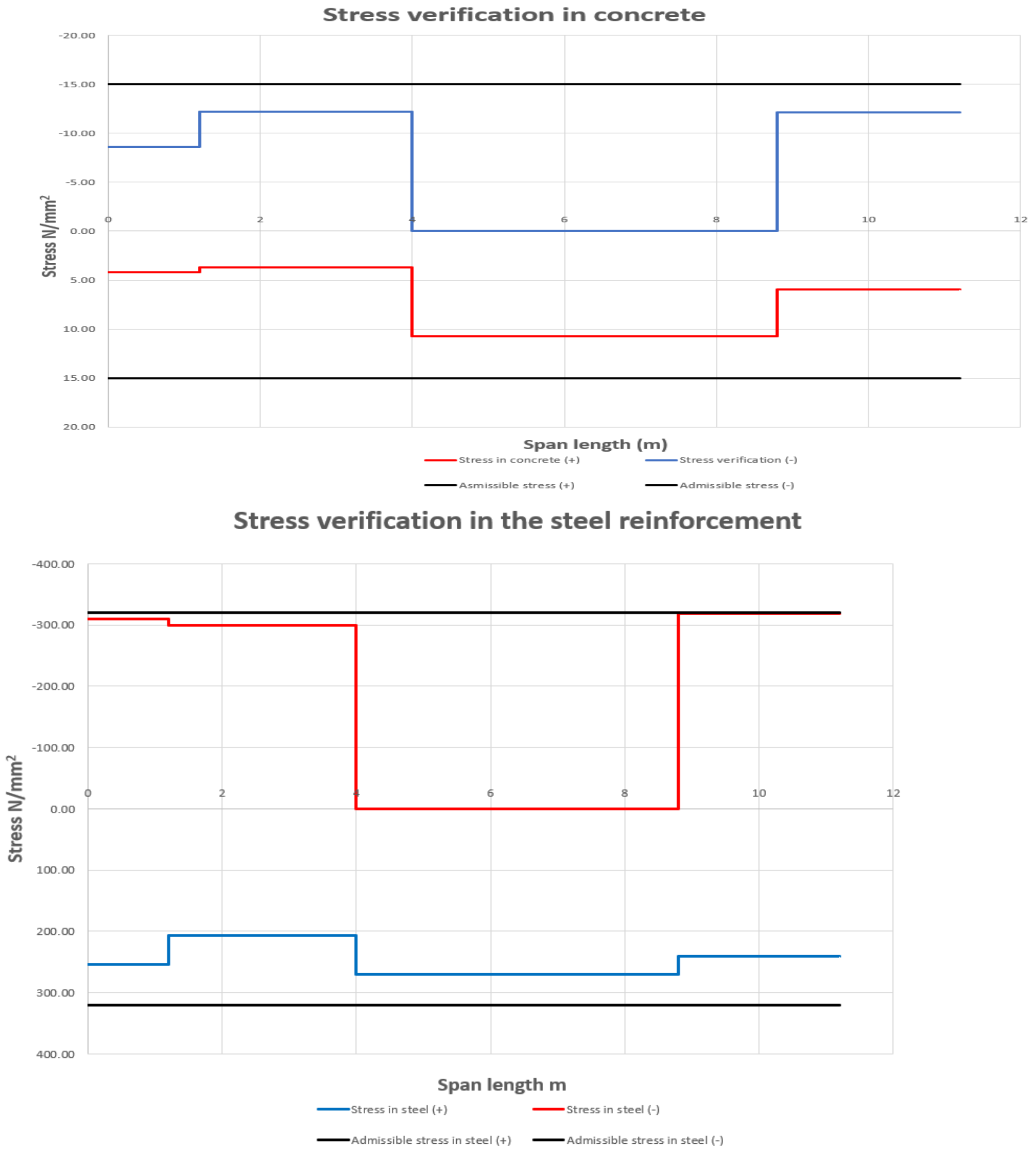


Figure 3. 17. Recapitulative curve of the stress verification in beam

3.3.3. Column design

3.3.3.1. Preliminary design

The column chosen for the design is enclosed in a green circle on the formwork plan as presented in figure 3.18. The vertical elements as well as the horizontal elements are modelled as frame elements. The combination of principal beam load arrangement leads to four load arrangement for the columns.

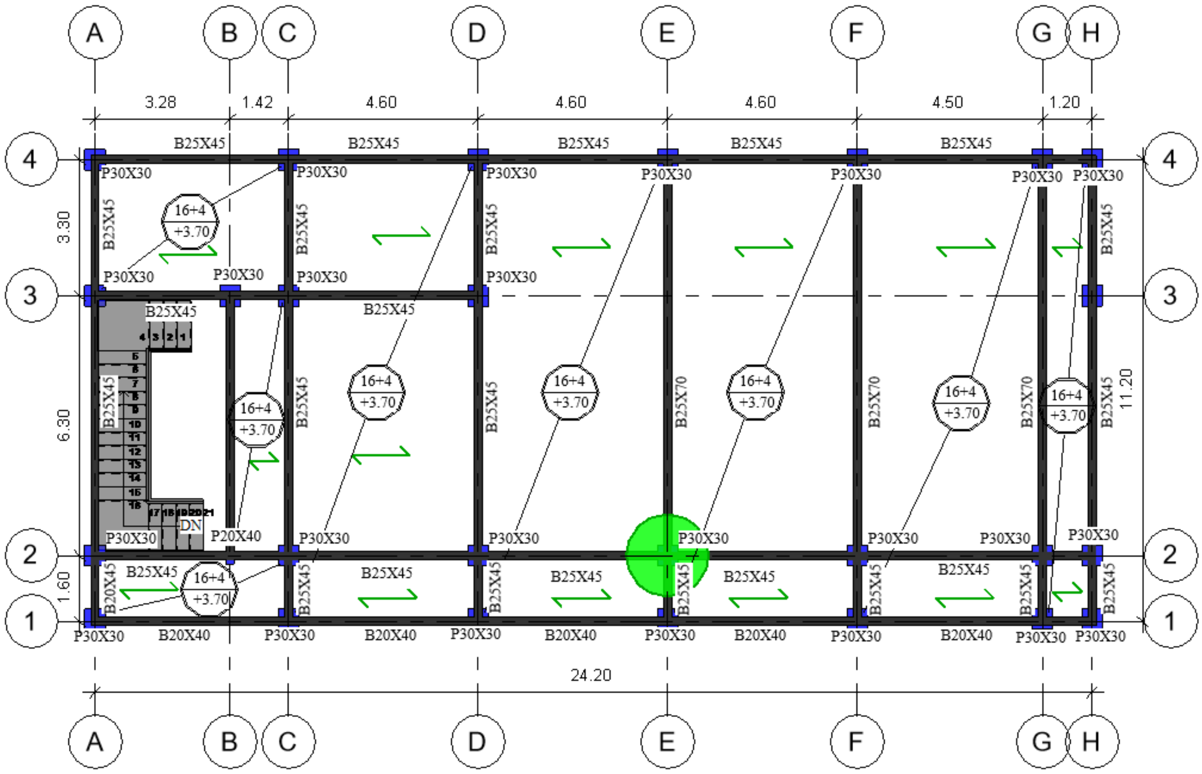


Figure 3. 18. Choice of the column for design

The columns are designed by modelling the structure in 3D with fixed supports at the base. The procedure is similar to that of horizontal structural elements.

Two steps are involved in the preliminary design of the column, the first of which is based on the axial load resistance to determine the minimum area section and the second is based on the analysis of the 3D model of the structure to verify the global dynamic behaviour.

3.3.3.2. Axial loads resistance of the section

60% of the concrete resistance is used to take over the axial force in the preliminary design for columns in seismic areas. We can thus compute the minimum area section of the equation:

$$N_{Rd} = 0.6 f_{cd} \, X \, A_c \geq N_{Sd} \qquad \text{(Eq. 3.5)}$$

Where:

Ac: Area of concrete section

Nsd; Axial load computed using the recovery area of the column

Equation 3.6 is used for the determination of the axial load

$$N_{Sd} = q\ X\ S_r\ X\ n \qquad \text{(Eq. 3.6)}$$

Where:

q: Uninform distributed load on each floor computed at ULS

Sr: Recovery area of the column

n: Number of floors of the building

We obtain $A_c \geq \frac{qXS_rXn}{0.6f_{cd}} = 59409.8mm^2$

Assuming a square section, we have a ≥ 243mm. The column section is then considered to be of section 300mmX300mm.

3.3.3.3. Modal analysis of the structure

The vibration modes of the analysis and the period of vibration of the first mode enable the estimation of the section of the column.

The structure under study in this work is two storey building with a total height of 10.1m above ground level. Its fundamental period is given as thus:

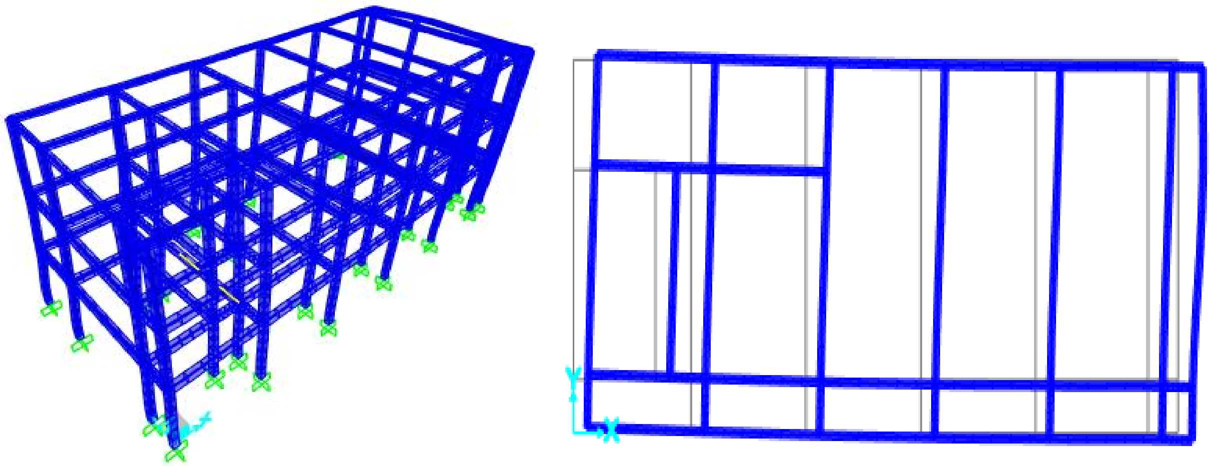
$$T_1 = C_t H^{\frac{3}{4}} = 0.075\ X\ (10.1)^{\frac{3}{4}} = 0.4s$$

Where:

Ct: is equal to 0.075 for concrete frames

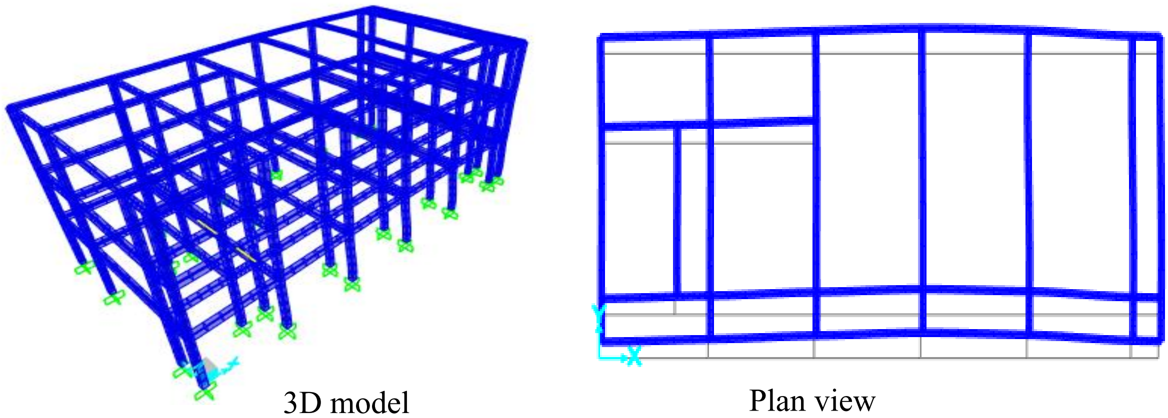
H: height of the building above ground level

The structure's period of vibration for the first mode is T=0.33s. The 3D modelling of the building in SAP2000 with fixed base and a percentage of participation of the imposed loads of 30 permit to have the first mode as a translation (figure 3.19), the second mode as a translation (figure 3.20) and the third mode as torsion (figure 3.21).



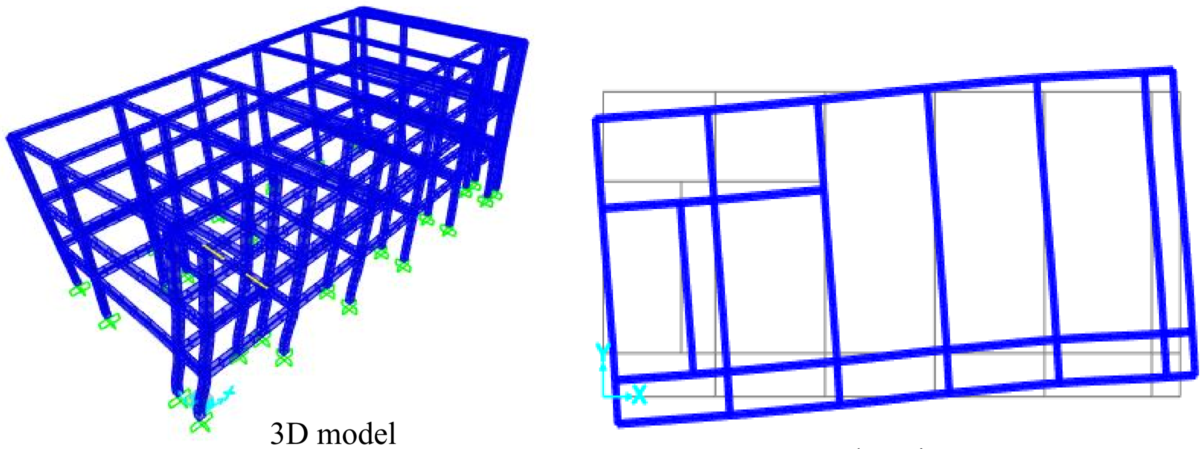
3D model Plan view

Figure 3. 19. First vibration mode of the structure: translation along the x direction



3D model Plan view

Figure 3. 20. Second vibration mode of the structure: translation along the y axis



3D model Plan view

Figure 3. 21. Third vibration mode of the structure: torsion

3.3.3.4. Bending moment and axial force column verification

The four load arrangements considered for the principal and secondary beams of the building generate the following solicitation curves for the bending moment in the x and y direction and axial loads presented in figure 3.22, figure 3.23 and figure 3.24 respectively.

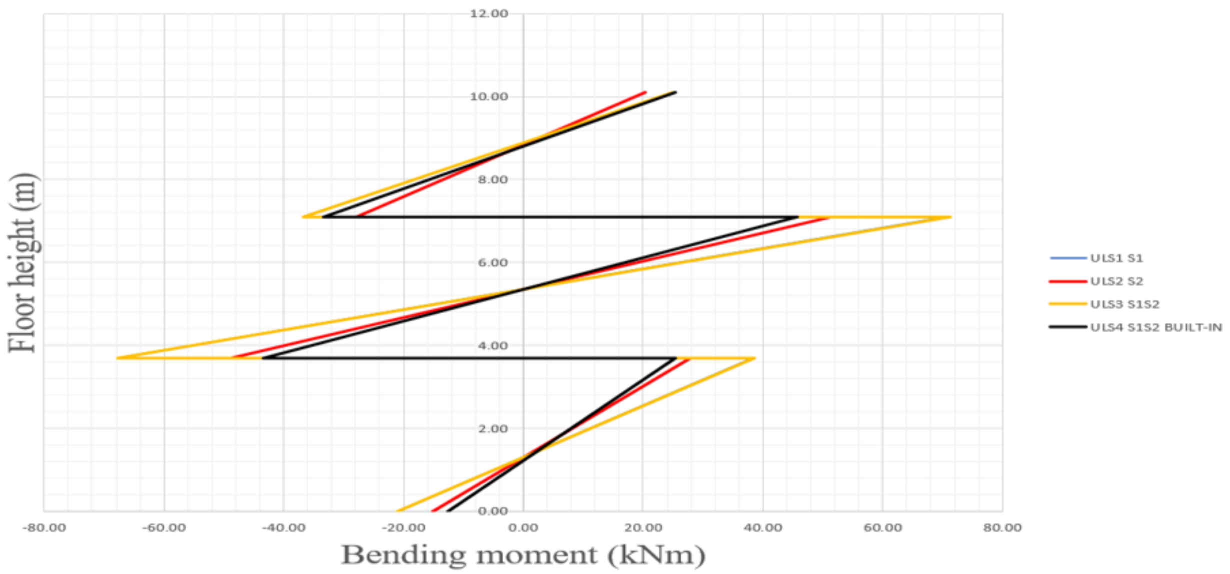


Figure 3. 22. Bending moment solicitation curves in the x-direction at ULS

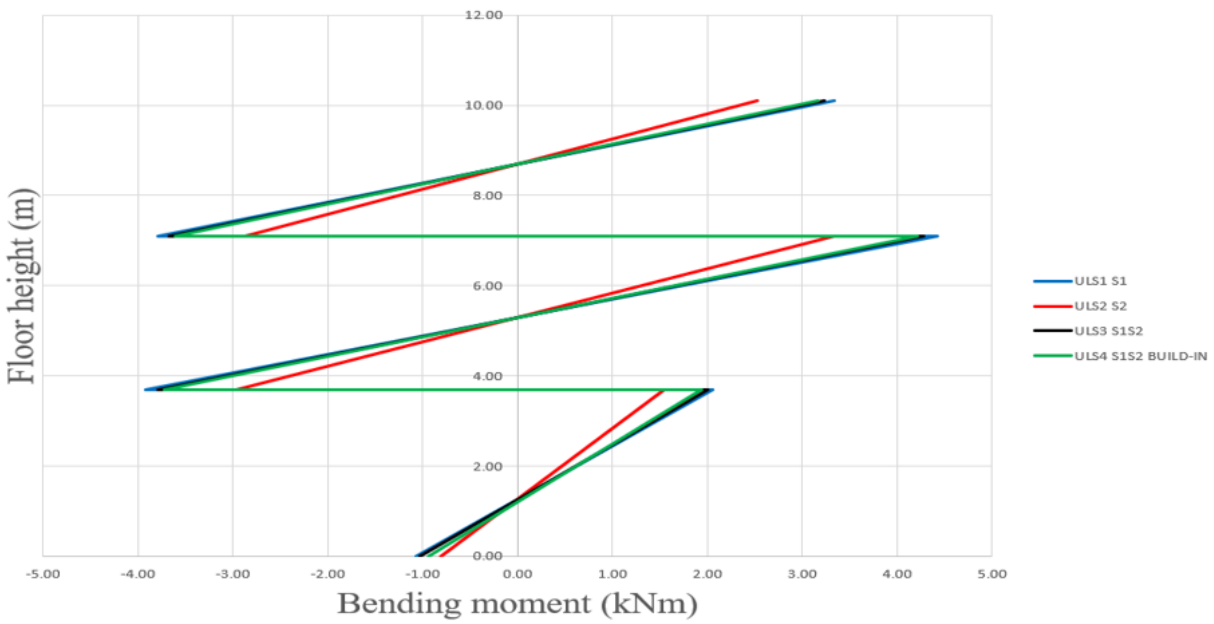


Figure 3. 23. Bending moment solicitation curves in the y-direction at ULS

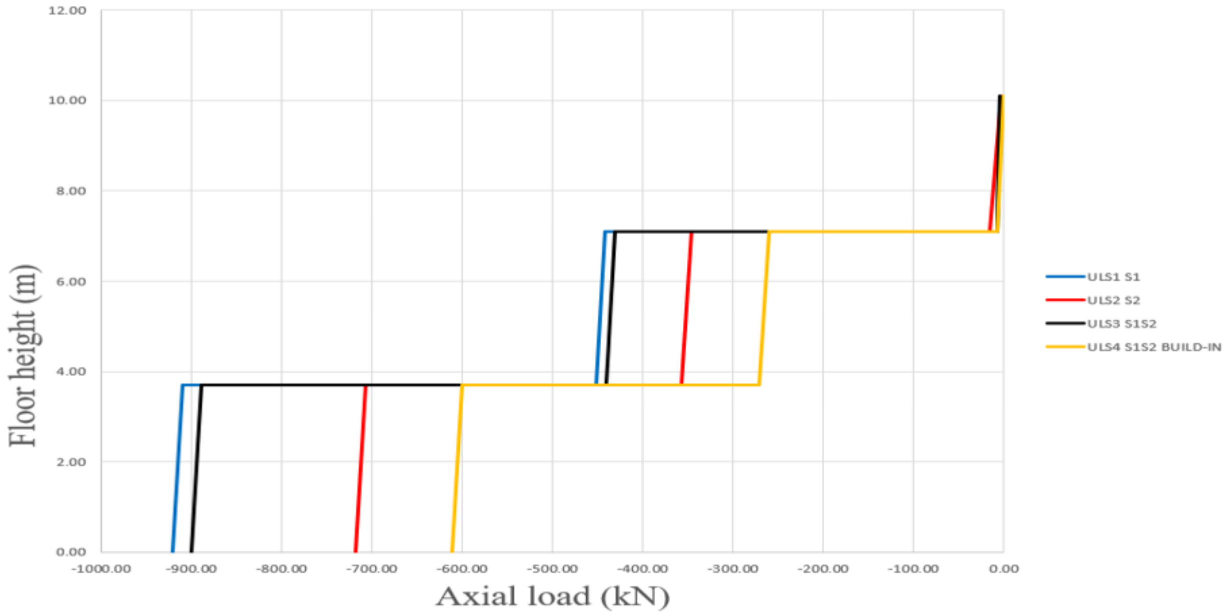


Figure 3. 24. Axial load solicitation curves on the column

From the solicitations, the envelope curves of the bending moment in both the x and y direction and the axial load solicitation are obtained and presented in figure 3.25, figure 3.26 and figure 3.27

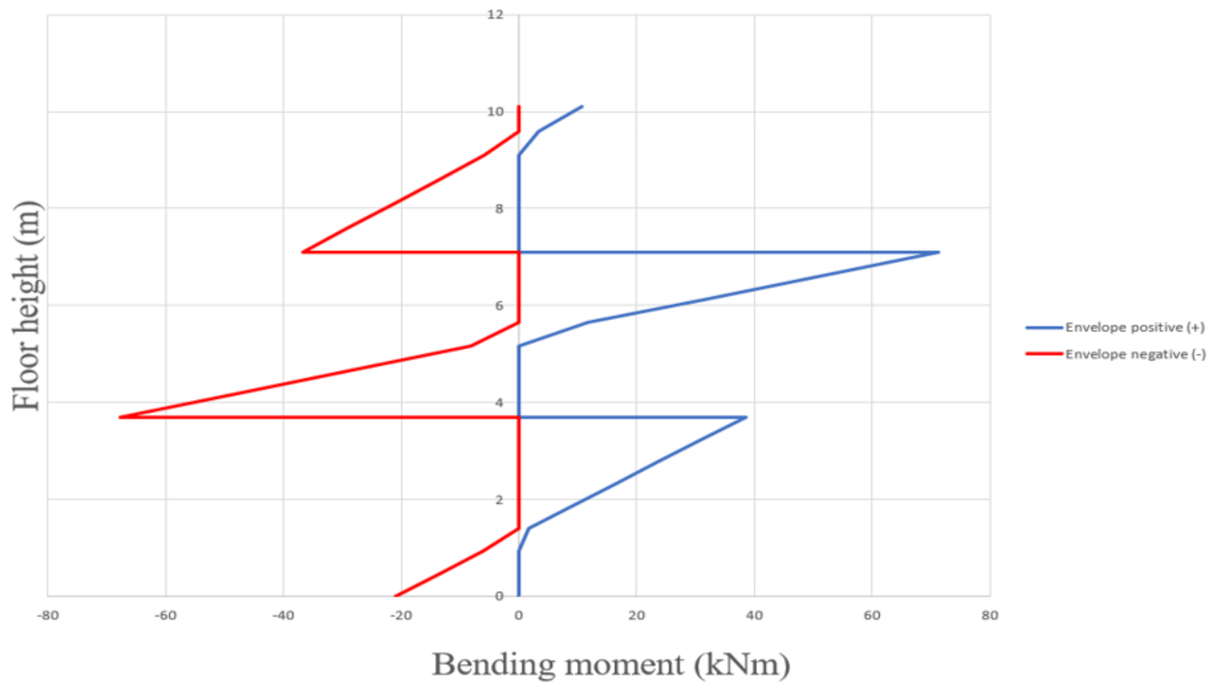


Figure 3. 25. Bending moment envelope curve in the x-direction at ULS

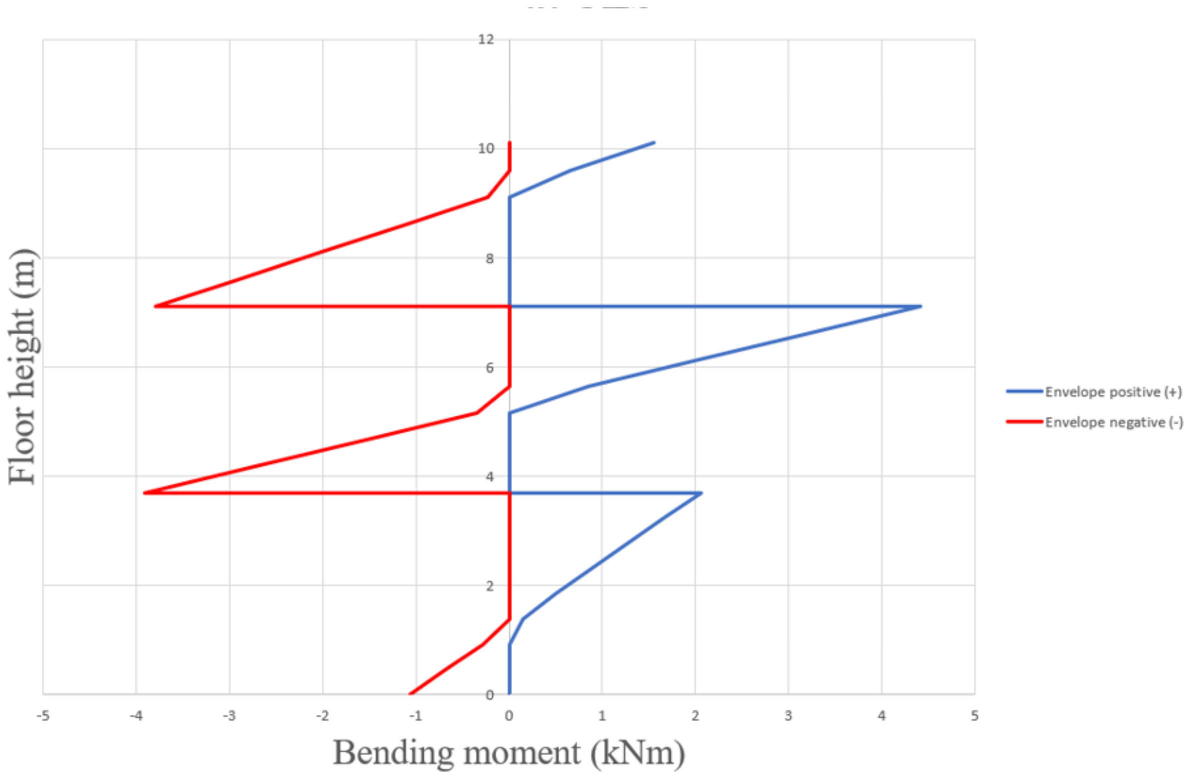


Figure 3. 26. Bending moment envelope curve in the y-direction at ULS

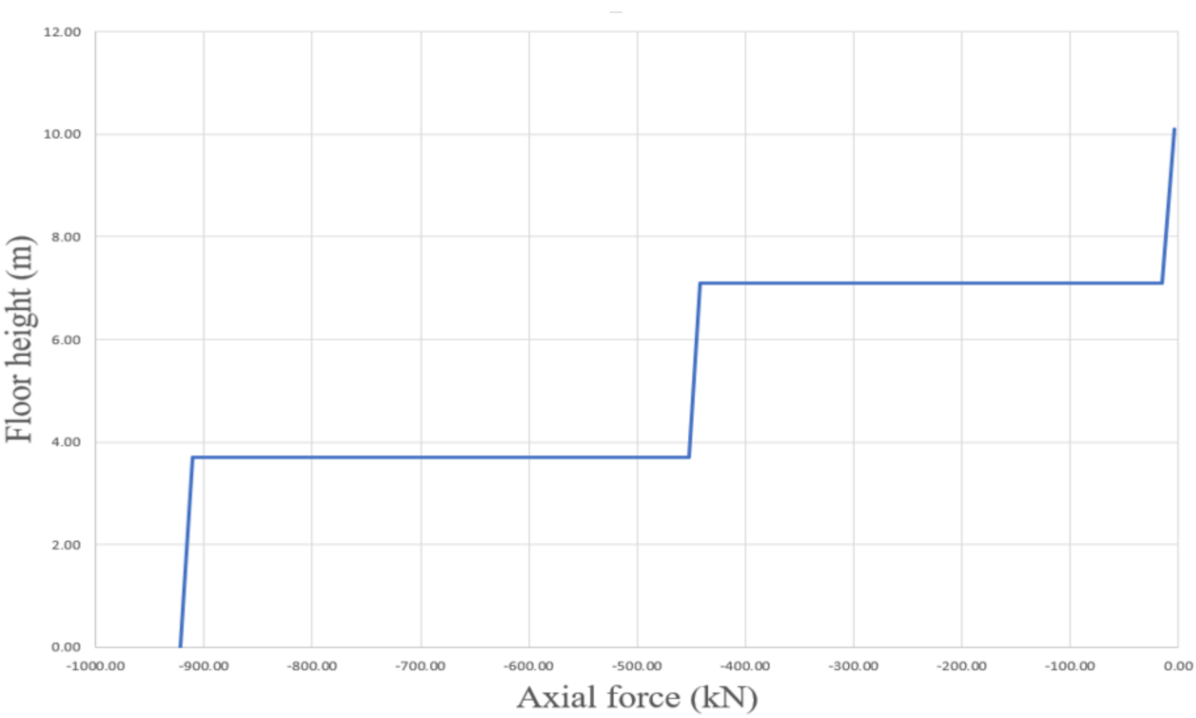


Figure 3. 27. Axial load envelope curve on column at ULS

The verification of the axial loads and the bending moment is done through the interaction diagram as presented in section 2.5.3.2. According to the aforementioned provisions described in section 2.5.3.2, equations 2.46 and 2.47 we obtain reinforcement steel area in the column as

$$265mm^2 \leq A_s \geq 3600mm^2$$

The column section of 300mm width by 300mm height has a minimum number of 6 bars of 14mm (6Φ14). As it is design in the x and y direction, the section has a total steel reinforcement of 8Φ14, giving a total reinforcement steel area of 1231.5mm^2 which is within the range of A_S.

The M-N interaction diagram for the x and y direction of the column is presented in figure 3.28 and figure 3.29 respectively.

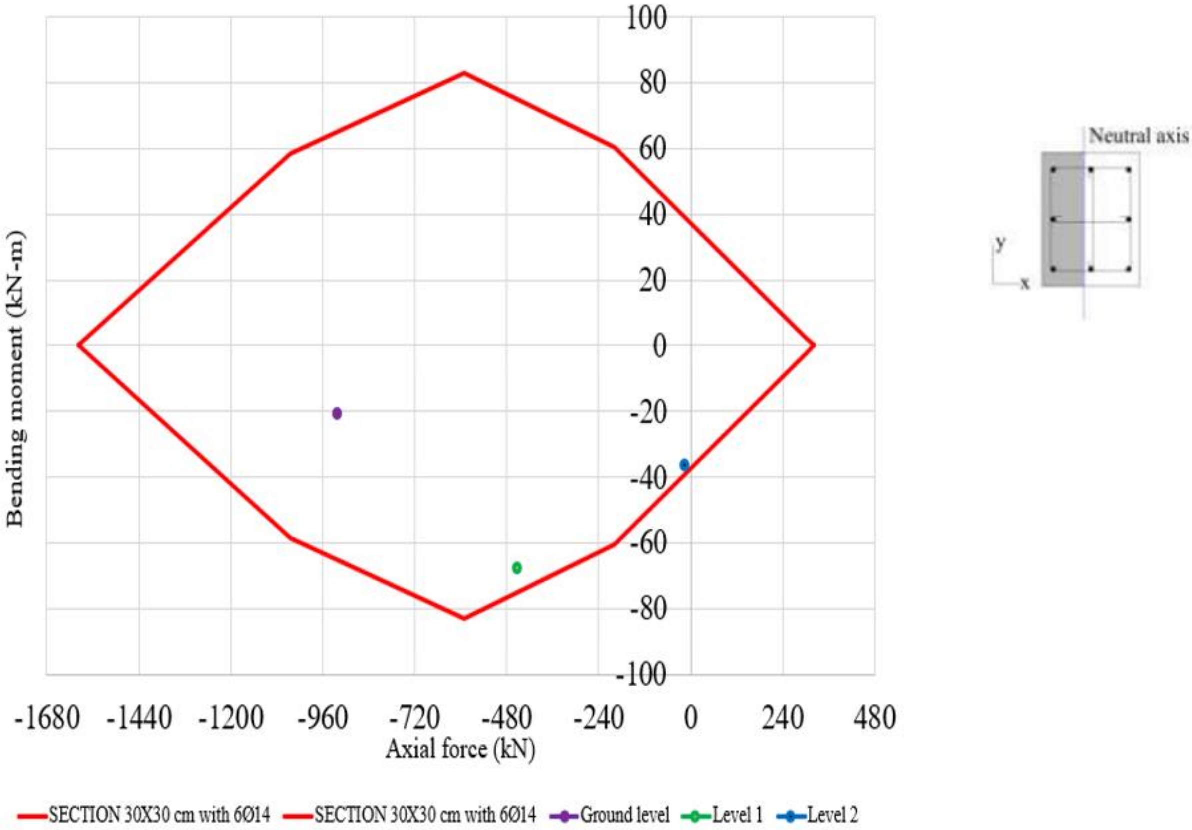


Figure 3. 28. M-N interaction diagram of column E2 in the x-direction

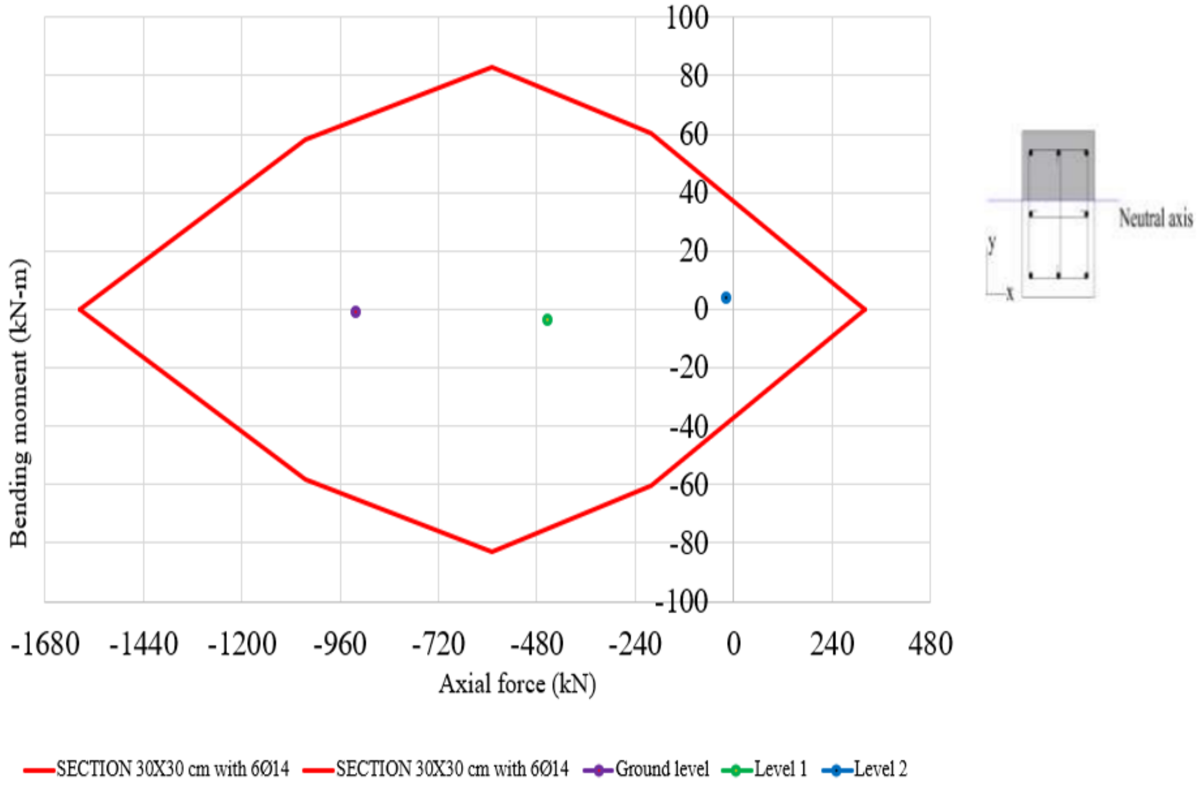
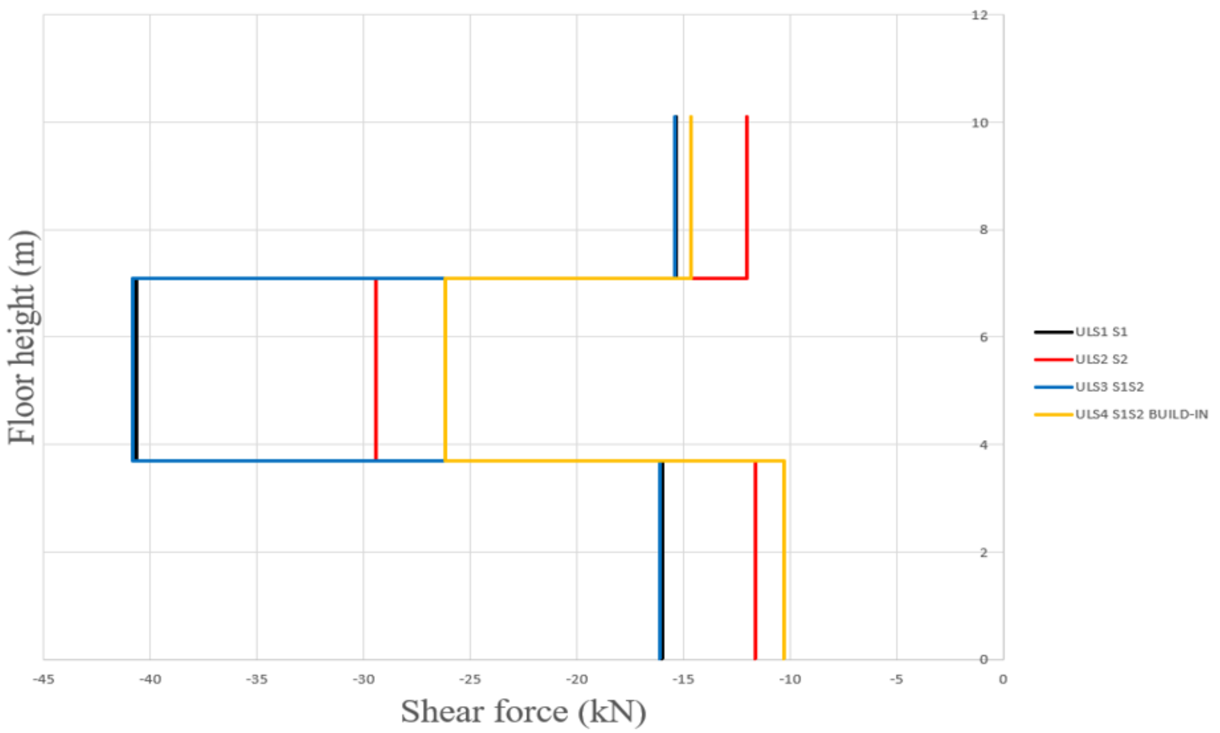


Figure 3. 29. M-N interaction diagram of column E2 in the y-direction

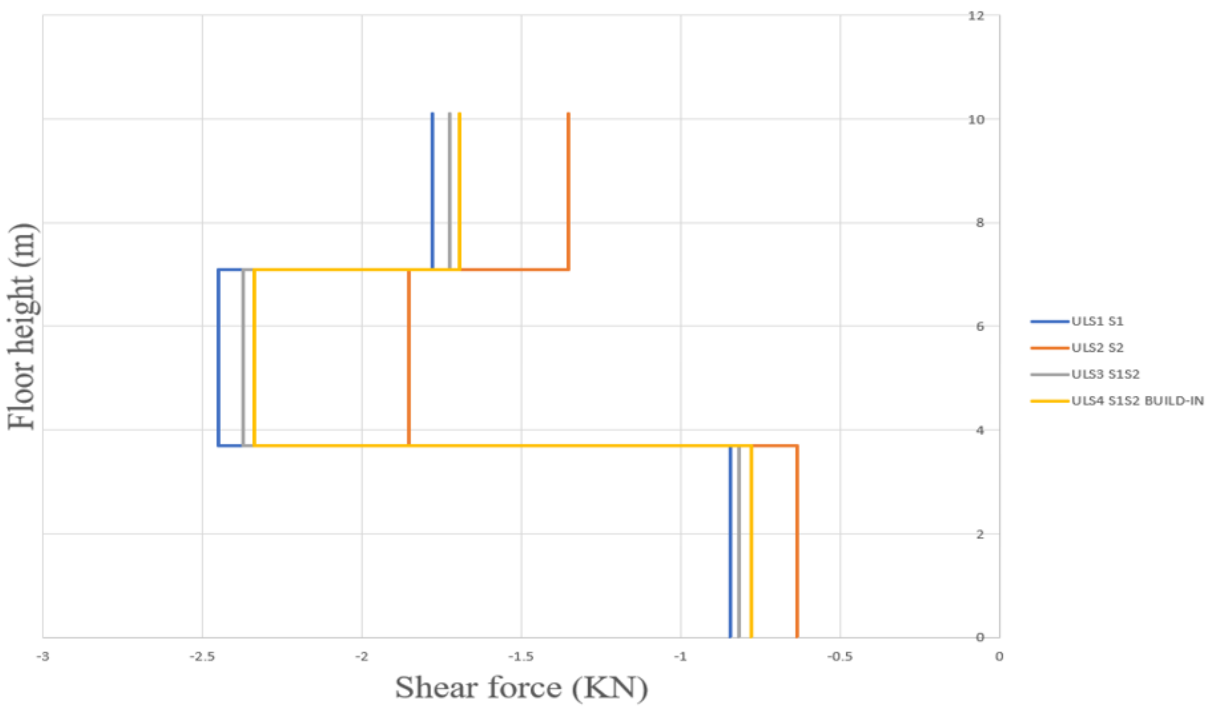
The bending moment-axial force (M-N) points for the columns on grid point E2 that is from ground level to level 2 are within the M-N interaction diagram. Thus, the section is correct.

3.3.3.5. Shear verification

The different load arrangements also make it possible to obtain solicitation curves for shear in both the x and y direction as shown in figure 30 a and b respectively.

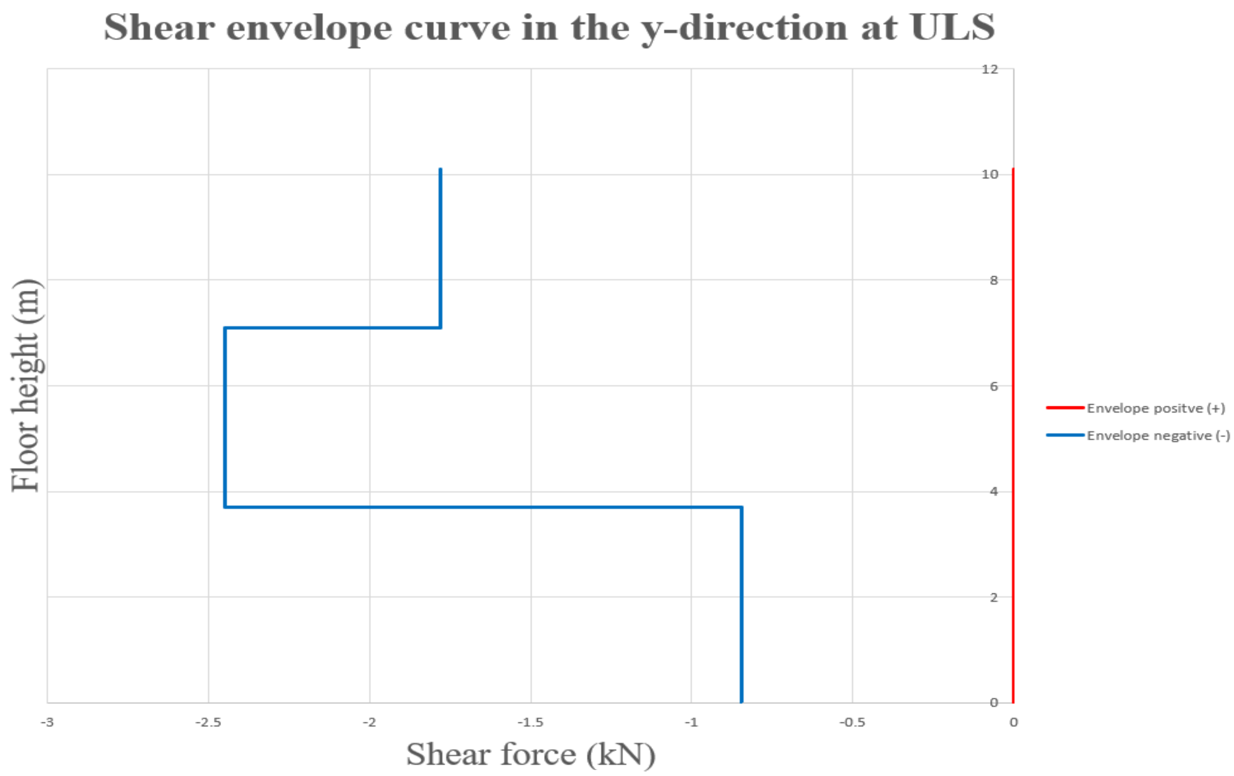


(a) Shear force solicitation curves in the x-direction at ULS

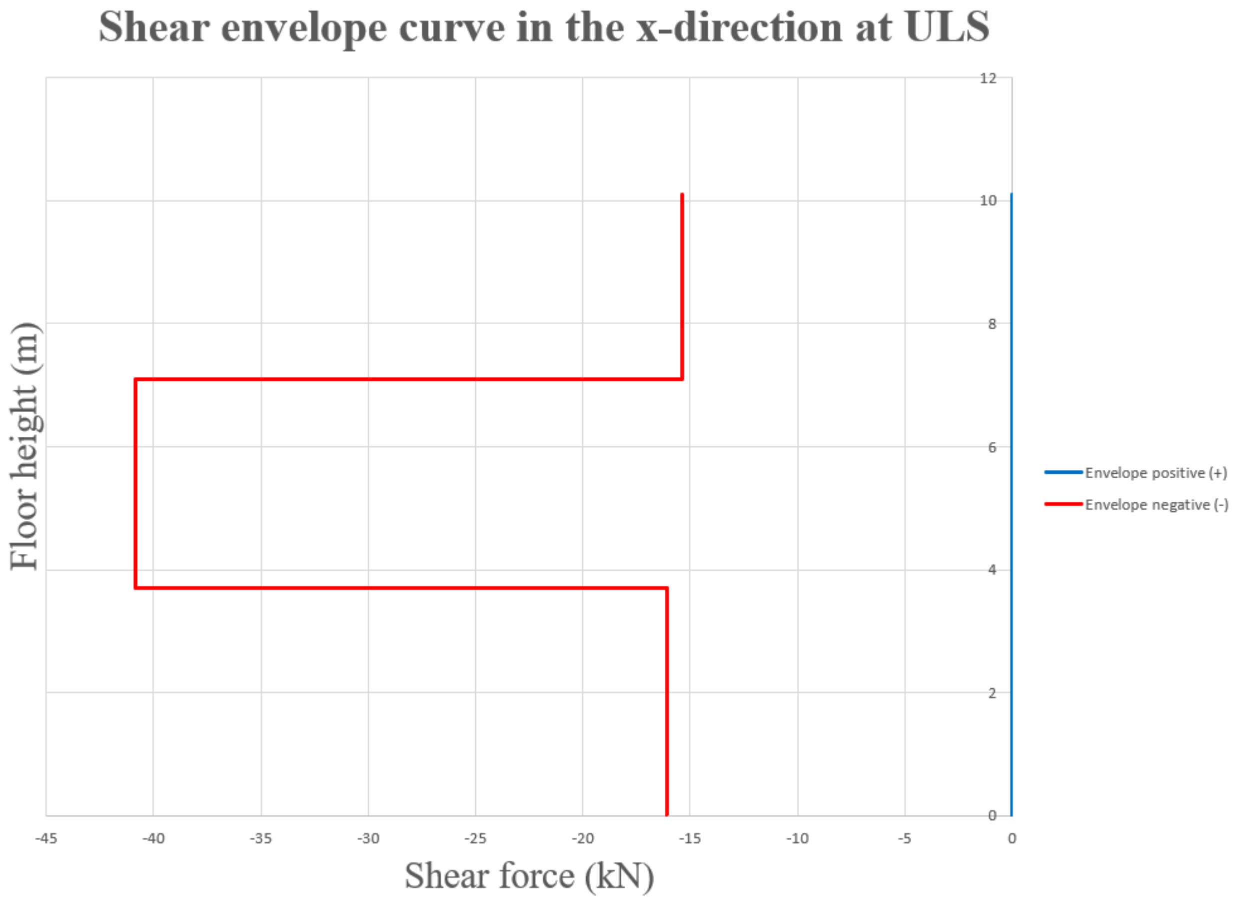


(b) Shear force solicitation curve in the y-direction at ULS

Figure 3. 30. Shear force solicitation curves on the column



(a)



(b)

Figure 3. 31. Shear force envelope curve: (a) in the x-direction, (b) in the y-direction.

Applying the procedure presented in the section 2.5.3.3, we observe that the shear resistance of the section without shear reinforcement is greater than the maximum shear solicitation on the column so the detailing of members has to be applied to have the spacing. In our case, we consider a diameter of 8 mm and the maximum spacing of the transverse reinforcement is given by:

$$S_{cl,max} = \min(320{,}300{,}400) = 300mm$$

So, applying the prescriptions of the section 2.5.3.3, we obtain a spacing of the shear reinforcement of: 18 cm within 0.3 m above and below the beams and 30 cm for the rest of the column.

3.3.3.6. Slenderness verification

Following the procedure presented in section 2.5.3.4, the different parameters are evaluated and presented in table 3.7 for the ground floor column which is the most solicitated.

Table 3. 5. Parameters for slenderness verification.

A	ω	B	C	n	λ	λ_{lim}
0.7	0.34	1.29	2.22	0.72	17.3	47.15

From table 3.5, we have $\lambda < \lambda_{lim}$, so the slenderness of the column is verified.

With the design of the beam and the column being done, the correct sections of the elements and their reinforcements are used in the modelling of the joint on Abaqus.

3.4. Finite element analysis of joint using Abaqus

3.4.1. Description of joint

The joint under study is at the first floor of frame 2-2 as shown in figure 3.32 and figure 3.33.

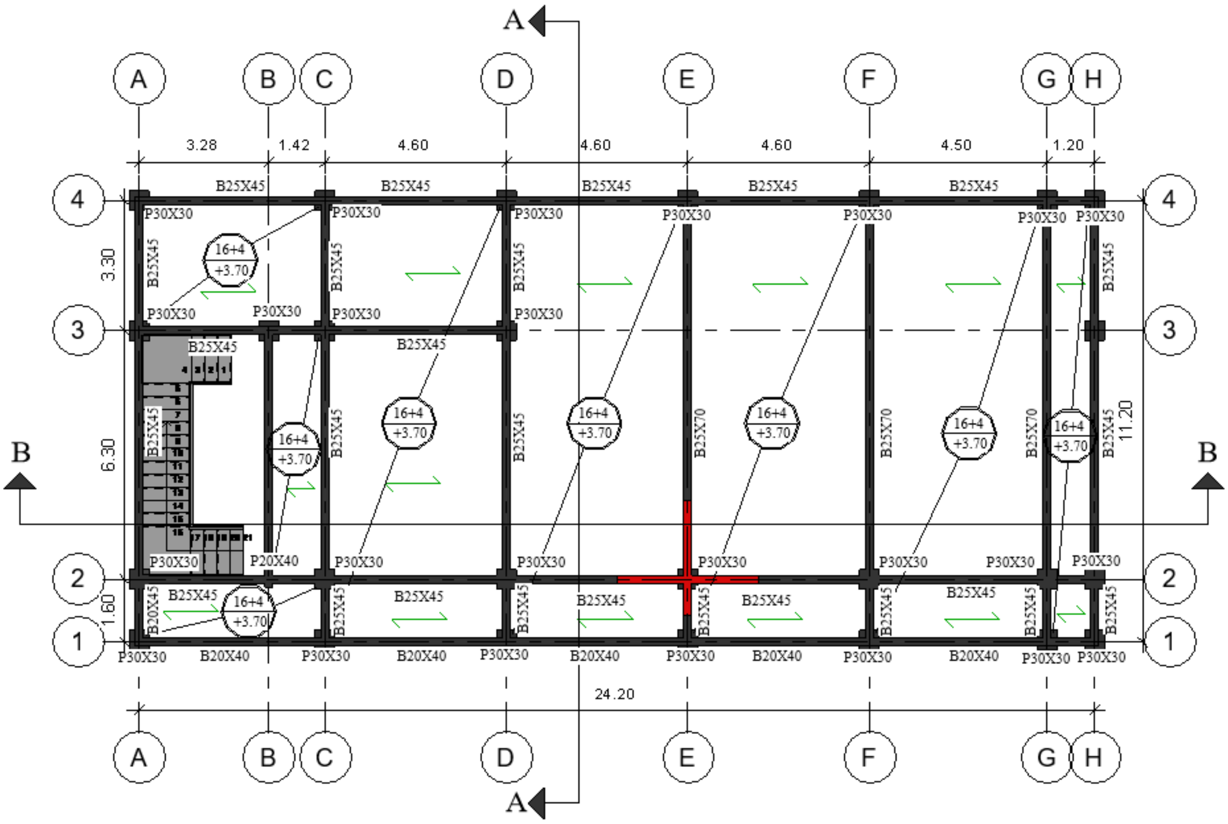


Figure 3. 32. First floor formwork indicating the joint under study in red

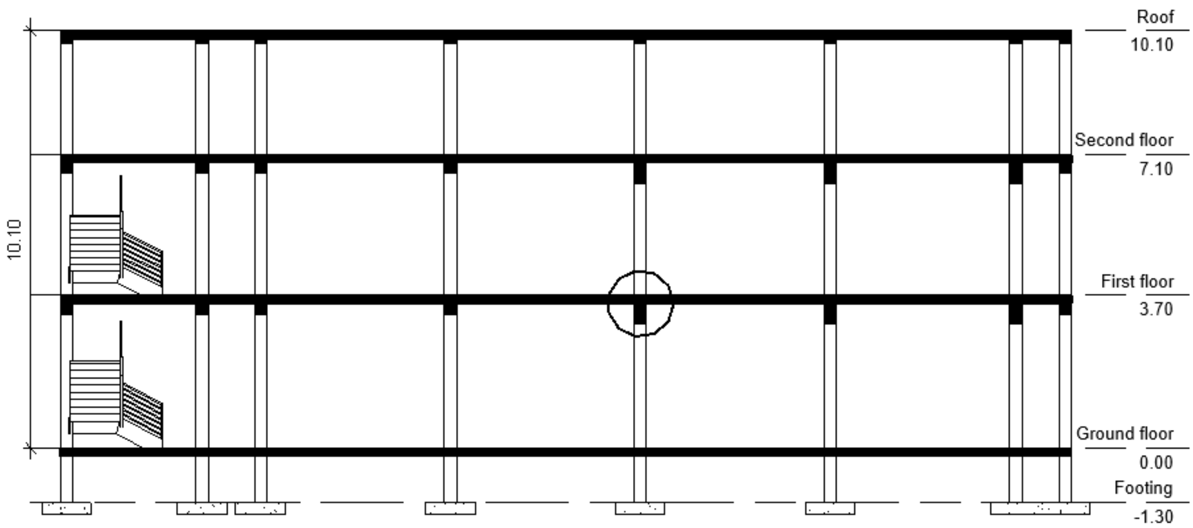


Figure 3. 33. Frame 2-2 indicating the joint with a circle.

3.4.2. Material properties used for the joint

Table 3. 6. Concrete properties.

Density	**Elastic properties**			
	Young's modulus		Poisson's ratio	
2.5E-09	31476		0.2	
Concrete damage plasticity				
Dilation angle	Eccentricity	fb0/fc0	K	Viscosity parameter
1.12	0.667	1.12	0.667	0
Concrete compressive damage				
Number	Damage parameter		Inelastic strain	
1	0		0	
2	0.9		0.011	
Concrete tension damage				
Number	Damage parameter		Cracking strain	
1	0		0	
2	0.99		0.00117	

Table 3. 7. Steel properties

Density	**Elastic properties**	
	Young's modulus	Poisson's ratio
7.85E-09	200000	0.3
Plastic properties		
Number	Yield stress	Plastic strain
1	250	0
2	345	0.02

3.4.3. Elements

As discussed in section 2.7.1, an element is created through the five different steps of the pre-processing stage. A uniform mesh size of 80mm is chosen for the concrete element over the whole geometry as shown in figure 3.34 and the same size is adopted for the steel bars and stirrups.

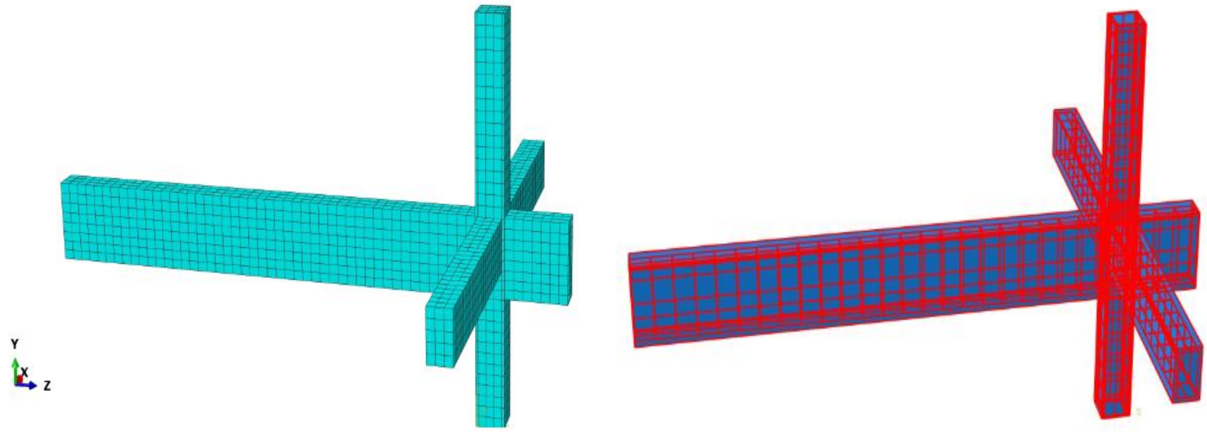


Figure 3. 34. Model specimen of concrete element meshed and reinforcement details.

3.5. Finite element analysis results

The finite element analysis results of the RC beam-column joint subjected to static loading is presented in terms of force-displacement curve. The validity of the finite element model is assessed by with the following results:

- Tensile stresses in concrete
- Compression stresses in concrete
- Cracking in the beam
- Tension stresses in the reinforcements
- Stress diagram in the reinforcement against strain

3.5.1. Tensile stresses in concrete

The tensile stresses in concrete is shown in figure 3.35. From the figure, the part in blue are those in compression unlike those in red which are in tension and have attained the maximum tensile stress in concrete. Those in green are also in traction but they do not exhibit elastic behaviour.

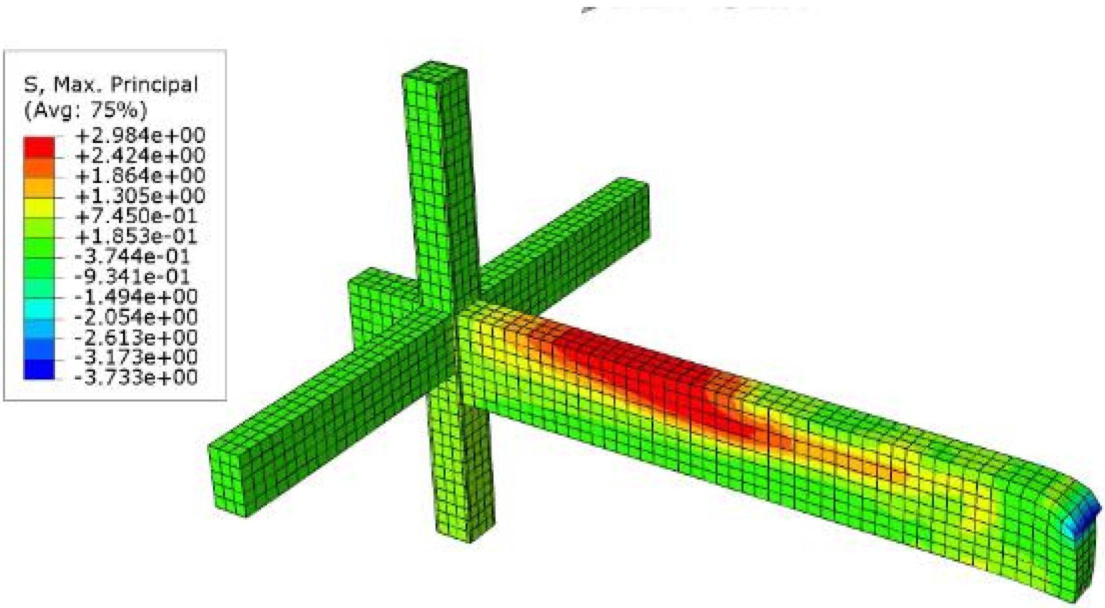


Figure 3. 35. Tensile stresses in concrete

3.5.2. Compressive stresses in concrete

As seen in figure 3.36, the red parts are in tension and have exceeded the maximum tensile strength. Unlike in tensile stresses, the green elements are in compression but are still in the elastic phase. As also observed from the figure, the compressive zones are lesser than concrete tensile zones.

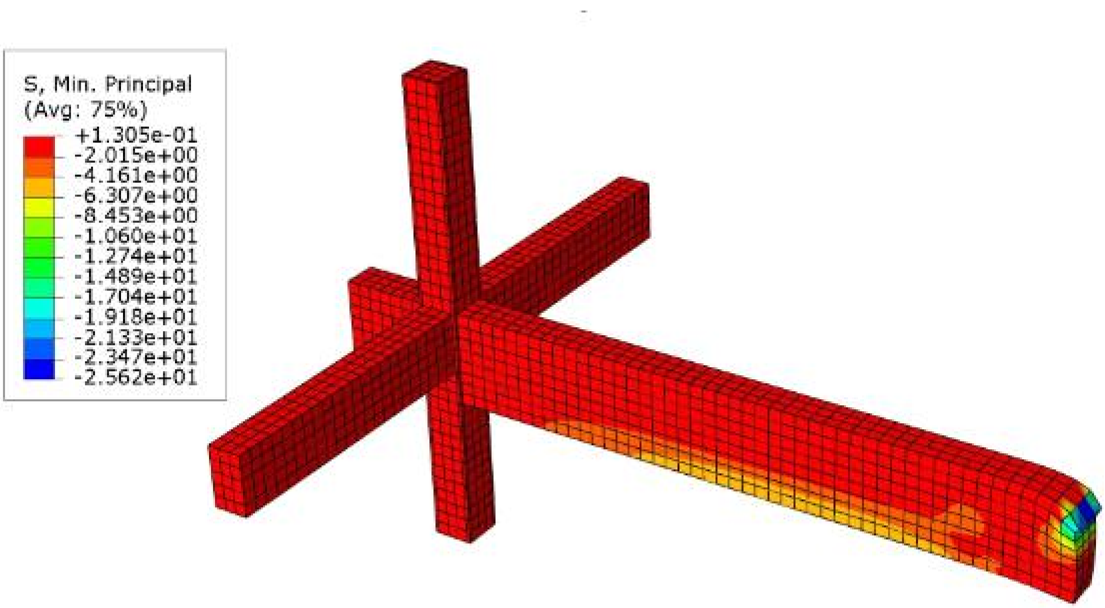


Figure 3. 36. Compressive stresses in concrete

3.5.3. Crack pattern

The crack pattern as seen in figure 3.37 shows that the crack development pattern starts from the centre of the principal beam and moves toward the loading direction. As discussed in section 1.4.4.1, joint shear forces in both horizontal and vertical directions are present at the joint as a result of moments from structural elements framing into the joint in their role of transmission of forces as shown in Figure 1.23. The joint is therefore subjected to non-negligible compressive and tensile stresses as shown in figure 3.37 and figure 3.38.

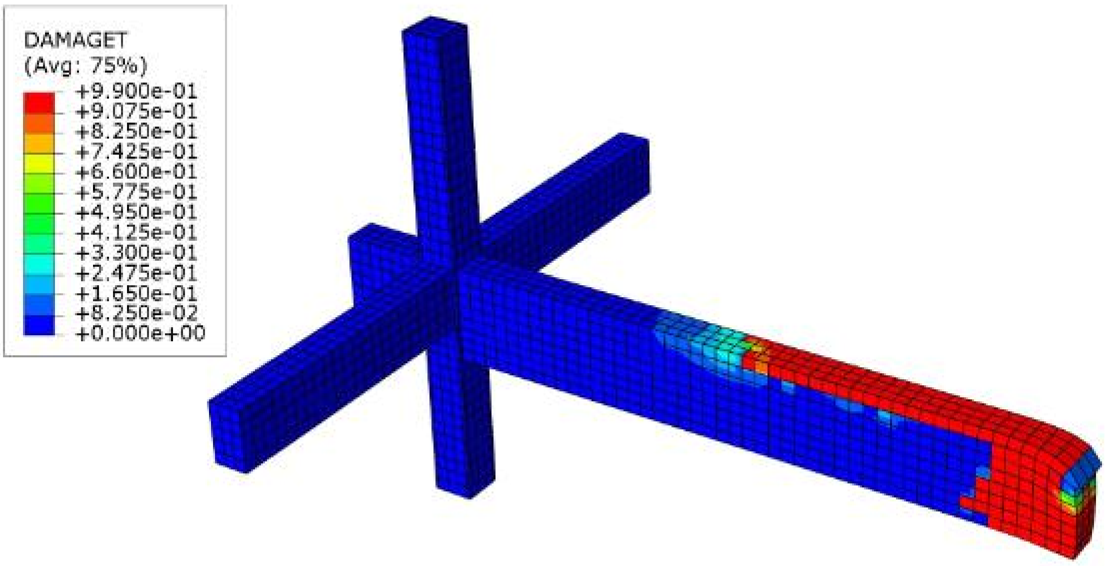


Figure 3. 37. Tensile stresses crack pattern

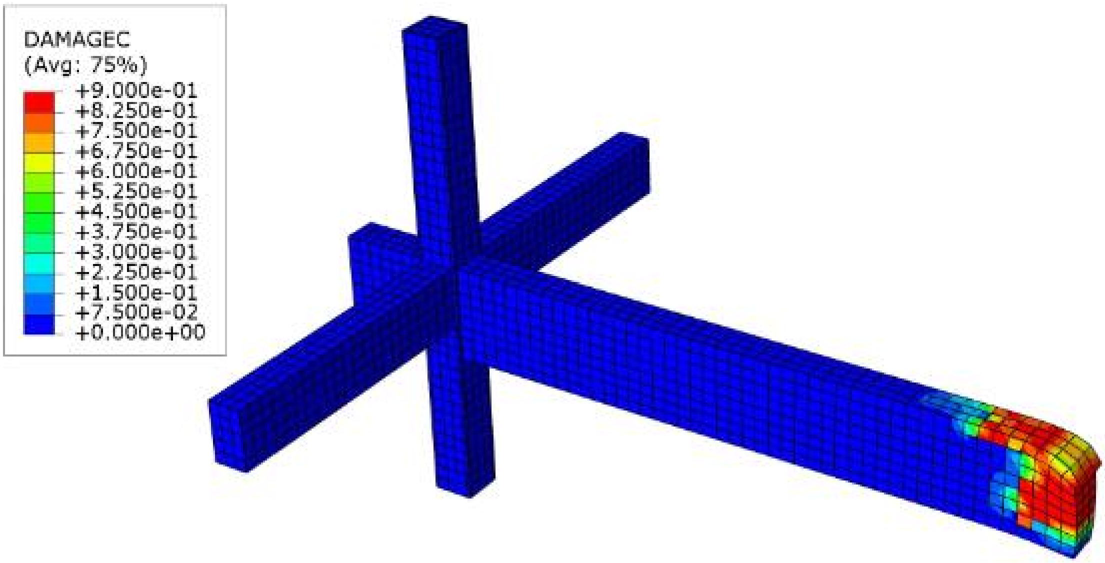


Figure 3. 38. Compressive stresses crack pattern

3.5.4. Tension in the reinforcements

As seen from figure 3.39, the tensile stress recorded in the steel reinforcements is up to 345MPa. It can also be seen from the figure that the longitudinal reinforcements are being stressed the central part of the principal beam that is the green colour to the edge of the beam where the load is being applied. In this part if the joint, the reinforcements are in tension and they are responsible for the ductile behaviour of the joint. As predicted by Pauley and priestly (1992), the steel reinforcements in tension are fully responsible for load transfer.

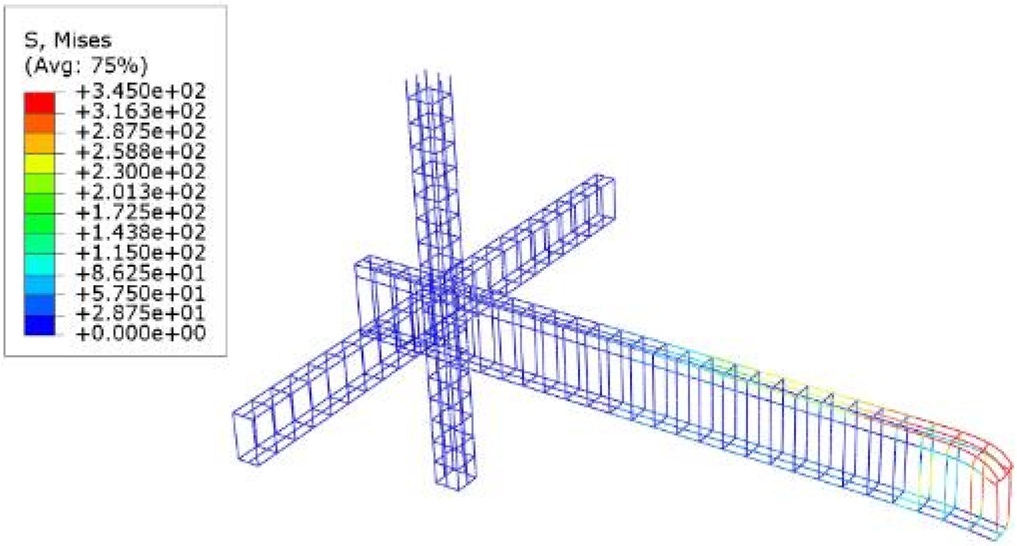


Figure 3. 39. Tension in the reinforcement

3.5.5. Force-displacement diagram and ductility

We used the force-displacement diagram to calculate the ductility. In section 1.3 of this work, ductility was defined as the ability of a structure or a selected structural component to deform beyond the elastic limits without excessive strength or stiffness degradation. As seen in that same section, figure 1.8 and figure 1.11 permit us to calculate the displacement ductility by first of all linearizing the curve in figure 3.40 which shows the force displacement diagram into an approximated perfectly elasto-plastic curve in order to obtain a value for yield displacement and ultimate displacement. From figure 3.40, it can be seen that the joint has an elastic behavior from a displacement of 0mm to 20mm, elasto-plastic behavior from 20cm to 60cm and a plastic behavior from 60mm to 88mm. also, from figure 3.41, the value of the displacement is gotten to be 4.1.

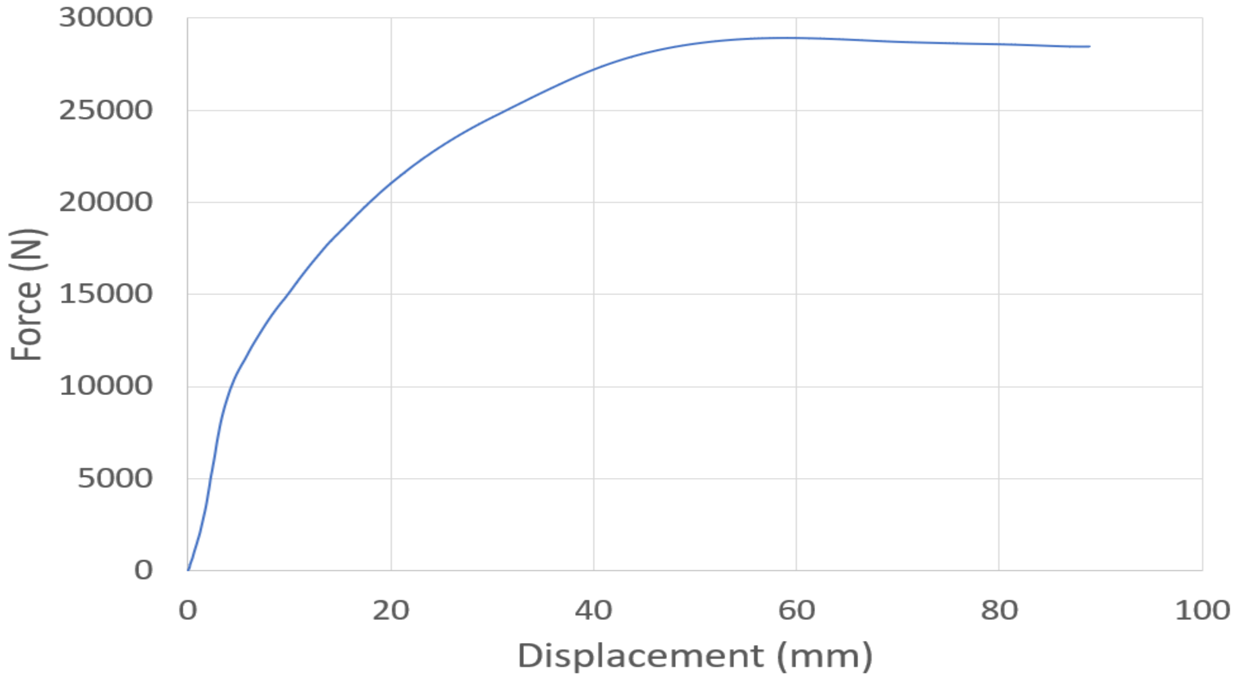


Figure 3. 40. Force-displacement diagram of the joint

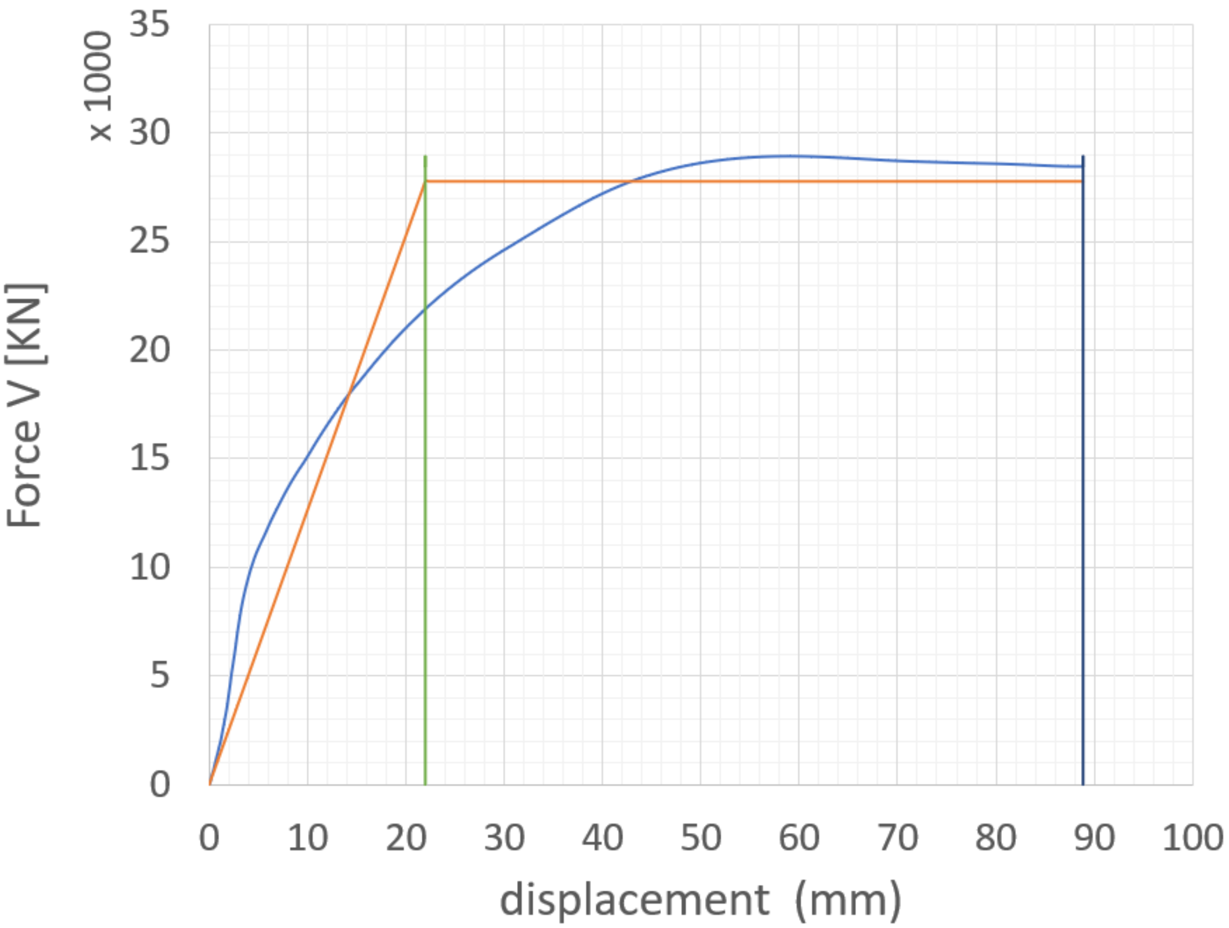


Figure 3. 41. Bilinear force-displacement diagram

Conclusion

The aim of this chapter was to present the case study, to perform the analysis and the design of the horizontal and vertical structural elements in order to analyse the joint using finite element method and calculate its ductility. At the end a section of 70cm height and 25cm width was obtained for the beams with the stress verification being verified and a section of 30cm by 30cm for the most solicitated column with slenderness being verified. The joint between these two elements was then modelled on Abaqus, the results of the compressive, tensile and crack pattern stresses in the concrete and the tension in the reinforcements were gotten coupled with the reaction forces at the bottom of the column that were plotted against the displacement along the principal beam for the ductility to be calculated.

GENERAL CONCLUSION

The purpose of this work was to determine the ductility of a reinforced concrete beam-column joint using finite element method analysis. This study has been done firstly through a literature review on finite element method, ductility, joints and reinforced concrete. Secondly, the methodology outlined the procedure on how to design reinforced concrete beams and columns. It also outlined the procedure on how to model a joint and run the analysis on Abaqus. Following this methodology, a three-floor storey building was modelled using SAP2000 (version 22) and the solicitations gotten. The solicitations were then used to design the most solicited beam and column in order to obtain their correct sections and reinforcement details. The joint between the most solicitated beam and column was the modelled on Abaqus taking into consideration all the necessary parameters for the analysis. The boundary conditions and the mesh size were also inserted and a graph of the reaction forces at the bottom of the column against the displacement along the principal beam was plotted using the data obtained from the results. The graph was then approximated to a bilinear elasto-plastic curve based on equal energy principle according to Annex B of Eurocode 8 in order to determine the yield displacement and the ultimate displacement for the ductility to be calculated. A value of 4.1 was gotten for the ductility which indicated from table 1.5 of section 1.3.4 that the ductility is of class medium, meaning the joint can resist deformations under moderate seismic actions without collapse. Finally, to increase the ductility of the joint especially when the height of the beam framing into the column is greater than the width of the column, it is recommended that both transversal and horizontal confining hoops be placed at the joint with a spacing equal to or less that the minimum spacing in the column.

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ANNEXES

Table A 1. Categories of used of the building (EC 1, Part 1, BS)

Category	Specific Use	Example
A	Areas for domestic and residential activities	Rooms in residential buildings and houses; bedrooms and wards in hospitals; bedrooms in hotels and hostels kitchens and toilets.
B	Office areas	
C	Areas where people may congregate (with the exception of areas defined under category A, B, and D[1])	**C1**: Areas with tables, etc. e.g. areas in schools, cafés, restaurants, dining halls, reading rooms, receptions. **C2**: Areas with fixed seats, e.g. areas in churches, theatres or cinemas, conference rooms, lecture halls, assembly halls, waiting rooms, railway waiting rooms. **C3:** Areas without obstacles for moving people, e.g. areas in museums, exhibition rooms, etc. and access areas in public and administration buildings, hotels, hospitals, railway station forecourts. **C4**: Areas with possible physical activities, e.g. dance halls, gymnastic rooms, stages. **C5**: Areas susceptible to large crowds, e.g. in buildings for public events like concert halls, sports halls including stands, terraces and access areas and railway platforms.
D	Shopping areas	**D1:** Areas in general retail shops **D2**: Areas in department stores
[1] Attention is drawn to 6.3.1.1(2), in particular for C4 and C5. See EN 1990 when dynamic effects need to be considered. For Category E, see Table 6.3		
NOTE 1 Depending on their anticipated uses, areas likely to be categorised as C2, C3, C4 may be categorised as C5 by decision of the client and/or National annex. NOTE 2 The National annex may provide sub categories to A, B, C1 to C5, D1 and D2 NOTE 3 See 6.3.2 for storage or industrial activity		

Table A 2. Imposed loads on floors, balconies and stairs in buildings (EC 1 Part 1)

Categories of loaded areas	q_k [kN/m^2]	Q_k [kN]
Category A		
- Floors	1,5 to2,0	2,0 to 3,0
- Stairs	2,0 to4,0	2,0 to 4,0
- Balconies	2,5 to 4,0	2,0 to 3,0
Category B	2,0 to 3,0	1,5 to 4,5
Category C		
- C1	2,0 to 3,0	3,0 to 4,0
- C2	3,0 to 4,0	2,5 to 7,0 (4,0)
- C3	3,0 to 5,0	4,0 to 7,0
- C4	4,5 to 5,0	3,5 to 7,0
- C5	5,0 to 7,5	3,5 to 4,5
category D		
- D1	4,0 to 5,0	3,5 to 7,0 (4,0)
- D2	4,0 to 5,0	3,5 to 7,0

Table A 3. Values of Minimum cover, **$C_{min,dur}$**, requirements with regard to durability for reinforcement steel (EC2)

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

Figure A 1. *NASPW Block H building back view*

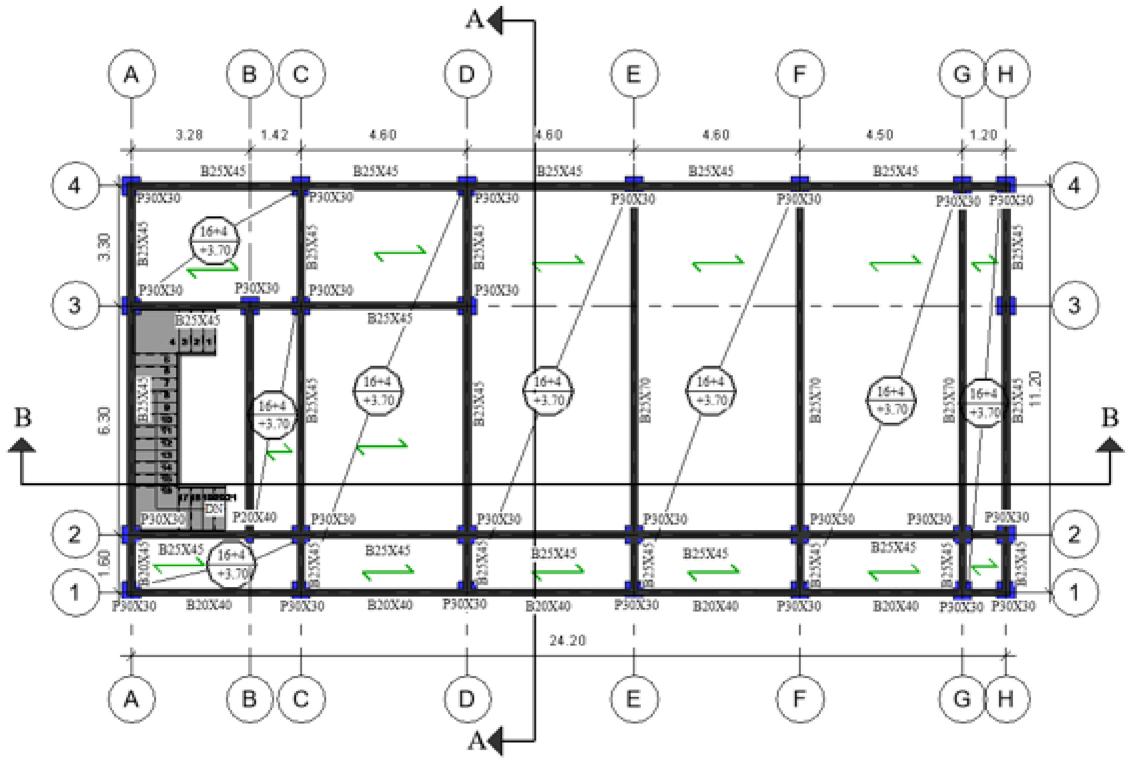

Figure A 2. Formwork plan of the second floor

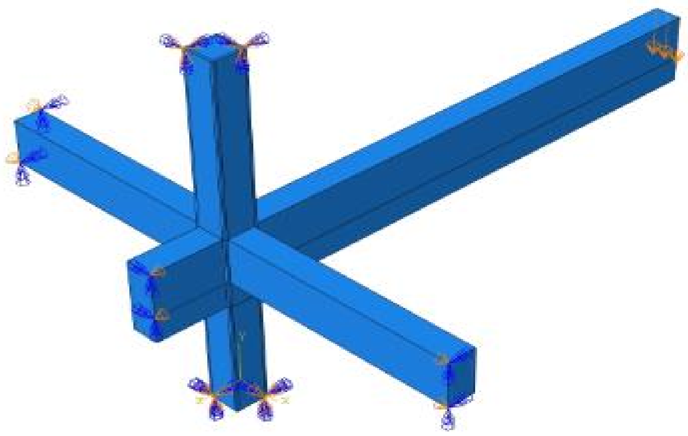

Figure A 3. Boundary condition applied to the joint

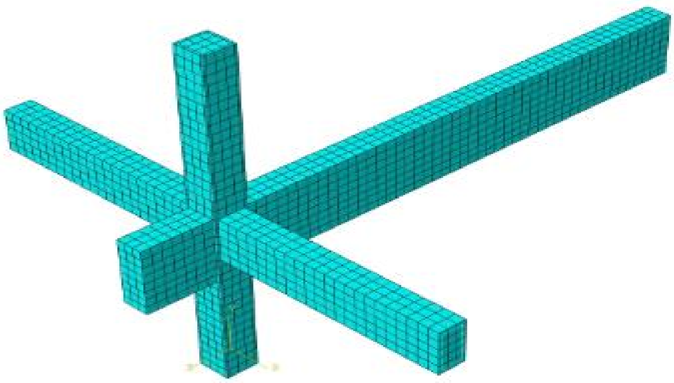

Figure A 4. Meshing applied to the joint

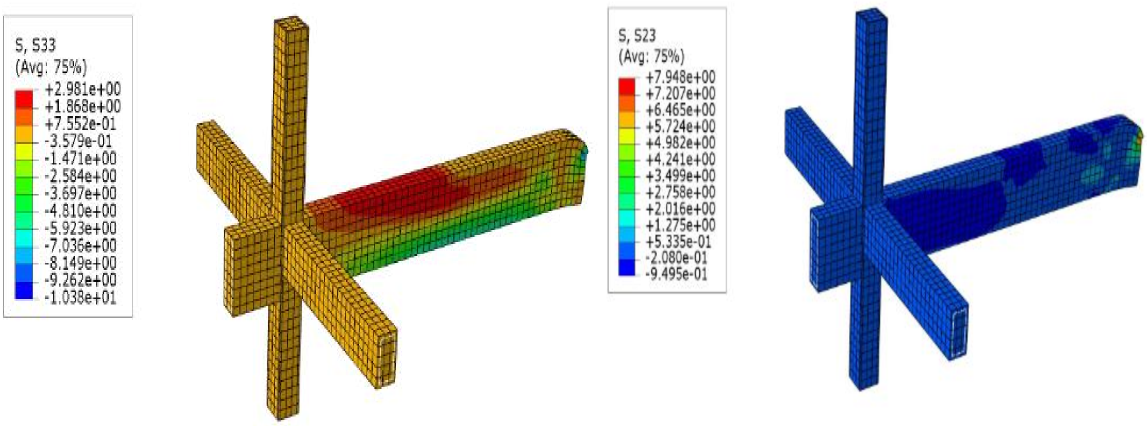

Figure A 5. Principal stresses in the joint