ADDIS ABABA

INSTITUTE OF TECHNOLOGY

CIVIL ENGINEERING DEPARTMENT

SHEAR STRENGTH OF REINFORCED CONCRETE BEAM-COLUMN JOINT UNDER CYCLIC LOADING

By

Tsega Hailu

Advisor

Dr. Asnake Adamu

A Thesis Submitted to the School of Graduate Studies of Addis Ababa
University in Partial Fulfillment of the Requirements for the Degree of
Masters of Science in Civil Engineering

April, 2018

Addis Ababa, Ethiopia

Addis Ababa University
Addis Ababa Institute of Technology
School of Civil and Environmental Engineering

This is to certify that the thesis prepared by Tsega Hailu, entitled: S*hear Strength of Reinforced Concrete Beam-Column Joint under Cyclic Loading* and submitted in partial fulfillment of the requirements for the degree of Master of Sciences in Civil Engineering (Structures) complies with the regulations of the University and meets the accepted standards with respect to originality and quality.

Signed by the Examining Committee:

Dr. Asnake Adamu	________________	______________
Advisor	Signature	Date
Dr. Shifferaw Taye	________________	______________
External Examiner	Signature	Date
Dr. Esayas G/Youhannes	________________	______________
Internal Examiner	Signature	Date
Dr. Agizew Nigussie	________________	______________
Chairman	Signature	Date

ACKNOWLEDGMENT

First, I would like to praise God, Who is worthy to be praised for His grace and endless blessing.

I would also like to thank my beloved family for their support, love and understanding that have provided me throughout my life.

My deepest gratitude goes to my advisor Dr. Asnake Adamu for his unreserved assistance, useful suggestions and timely comments at all stages of my work and also I would like to thank Dr. Abrham Gebre for his support and encouragement.

I would like to express my gratitude to Addis Ababa University for sponsoring this postgraduate study through Female Scholarship.

I would like to express my heartfelt appreciation to all lecturers of engineering courses under structural stream for their endless effort to create efficient and outstanding engineers and also for enhancing my confidence to tackle any challenge.

Last but not least, I would like to thank my friends for their constructive opinions and support.

ABSTRACT

A beam-column joint is a critical region which contribute to the integrity of the structure and that transfer forces from one structural element to another. A beam- column joint need proper design and detailing to achieve adequate performance of reinforced concrete frame or else it causes large story drift and structural damage. Among the most common failure modes in beam-column joints, a shear-type of failure is the most sensitive to cause structural failure. When the joint is subjected to cyclic loading its shear strength will be affected by many factors such as concrete strength, column axial load, joint aspect ratio, transverse reinforcement and lateral beams. Predicting the joint shear behavior is difficult because of the combined effect of those different factors. Among those different factors, most researchers have different conclusion on the effect of column axial load on the joint shear strength, especially when the joint is designed and detailed according to capacity design philosophy. Therefore, this study mainly focused on the effect of column axial load on the shear strength of an exterior beam column joint; which is detailed according to capacity design philosophy.

The effect of column axial load was studied through finite element software when the end of the beam was subjected to cyclic loading. The numerical study was performed through ABAQUS and the joint was examined with different magnitude of column axial loads and their effect on the shear strength of the joint was evaluated. The results obtained from the analysis showed that the column axial load up to $0.2\mathrm{f}_\mathrm{c}' \times \mathrm{A}_\mathrm{g}$ provided confinement to the joint and enhanced the joint shear strength. On the other hand column axial load beyond $0.2\mathrm{f}_\mathrm{c}' \times \mathrm{A}_\mathrm{g}$ exhibited degradation of joint shear strength. This might be attributed to concrete, crushed in the joint region.

DECLARATION

I, the undersigned, declare that this thesis is my original work and has not been presented for a degree in any other University and that all sources of materials used for the thesis have been duly acknowledged.

Name Tsega Hailu

Signature

Place Addis Ababa University Institute of Technology

Date of submission April, 2018

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF SYMBOLS

A_g	Gross cross sectional area of a column
A_j	Effective horizontal area of the joint
A_{sh}	Area of the beam longitudinal reinforcement
b_c	Width of column
b_j	Effective joint width
b_w	Width of beam
d_c	Uniaxial compressive damage
d_t	Uniaxial tensile damage
E_c	Modulus of Elasticity of concrete
ε_{cr}	Cracking strain of concrete
ε_t^{pl}	Tensile plastic strain
ε_c^{pl}	Compressive plastic strain
ε^{el}	Elastic strain
ε_o	Strain at the peak stress
ε_u	Ultimate strain
E_o	Initial undamaged elastic stiffness
ε	Eccentricity
f'_c	Concrete compressive strength
f_{cd}	Design value of concrete compressive strength
f_{ck}	Characteristic compressive cylinder strength of concrete
f_{cr}	Cracking stress of concrete

f_y	Yielding stress
f_{yh}	Yielding strength of the beam longitudinal reinforcement
h_c	Depth of column
K_1	Numerical coefficient
K_C	Shape factor
N	Column axial load
U_X	Displacement in the x direction
U_Z	Displacement in the z direction
U_y	Displacement in the y direction
v_d	Normalized axial load
V_{jhd}	Joint nominal shear stress
V_n	Nominal joint shear stress
$v_{jh,l}$	Shear strength due to beam reinforcement
w_c	Compression recovery
w_t	Tension recovery
η	Reduction factor on concrete compressive strength
σ_{co}	Uniaxial state of concrete
σ_{bo}	Biaxial state of concrete
σ_c	Compressive stress
σ_t	Tensile stress
γ	Joint strength coefficient
Ψ	Dilation angle
μ	Viscosity

1. INTRODUCTION

1.1. Background

The seismic performance of reinforced concrete structures are highly affected by the critical regions like beam-column joint. A beam-column joint is a critical region and the main part of a structural system that creates integrity of the structural elements and transfer loads from one structural element to another (from beam to column). The proper design and detailing of joints may give adequate service and strength to the structure or else it will be the cause of structural failure. Among different failure modes of joint, a shear-type of failure is common to cause structural damage when the structure is subjected to seismic load. Shear strength of cyclically loaded joint is an important measure of seismic behavior of RC structure. Tests have shown that concrete in the joint is always seen to deteriorate and joint strength decreases notably with increase in number of load cycles and peak deflection of beam [1]. Joint shear strength, which is subjected to a cyclic load, affected by many parameters such as concrete strength, column axial load, joint aspect ratio, transverse reinforcement and lateral beam [1], [2].

There are several studies conducted to investigate the shear behavior of the joints with those different parameters that affect the shear strength of joint [2], [3], [4]. Among those different parameters, column axial load is still under investigation because of its unclear effect on the joint shear behavior [3], [4].Chandra Clyde et al. [4] and Haach, V.G. et al. [3] concluded that column axial load has positive influence on joint shear strength. On the other hand, some researchers like Park et al. [2] concluded that column axial load has insignificant role on the strength of a joint. Those two different ideas makes difficult to conclude the effect of column axial load on the shear strength of a joint. Some researchers also concluded that in the case of weak column–strong beam design, it is obvious that increasing the column axial load up to the balanced point (compression limit) improves the joint shear strength because the column moment capacity is positively affected by compression axial load. On the other hand, in the case of strong column–weak beam design, which is the case of most tests, there have been reports that column axial load gives joint shear strength both beneficial and detrimental effects and in the case of higher column axial load, more test data are needed [2] and generally Park et al. [2] concluded that the effect of column axial load on beam-column joint strength is not completely known. According to the above reasons, investigating the effect of column axial load will be vital because the current Ethiopian building code [5] is based on strong column weak beam design philosophy and also due to the increasing of high rise building in the country. When the number of story on the structure increases the column axial load also increase due to gravity load of structural members, similarly as joint is one part of column its behavior will change when the value of column axial load fluctuate, therefore this variation affect the behavior of the joint ether in positive or in negative way, especially this load will increase when the structure is subjected to seismic loading. Thus, the present research aims to provide some results and conclusions about the behavior of joint under the influence of column axial load using numerical simulation.

Literature showed [2] exterior joints are more vulnerable to the effect of column axial load and shear failure than interior joints because of their short span to depth ratio; therefore, studying exterior joint is more important than interior joint.

In this study, a building was analyzed and exterior beam-column joint is selected and detailed in accordance with the current code ES EN1998-1:2015 [5] . The joint was modeled using finite element software ABAQUS [6] and evaluated under the influence of compression column axial loads.

1.2. Statement of the Problem

The shear strength of cyclically loaded joint is affected by many factors such as joint aspect ratio, transverse reinforcement, concrete strength and column axial load. Those factors have great contribution to the shear strength of joint and their effect on the joint is precisely concluded rather than column axial load. Thus, this thesis targeted to answer whether column axial load affect the shear strength of a joint and if it does, up to what certain level of compression column axial load?

The magnitude of column axial load will be fluctuate when the joint is subjected to seismic loading. Therefore, studying the behavior of a joint under the influence of cyclic loading will be important to consider the influence of seismic load on the variation of axial load magnitude.

1.3. Objectives

1.3.1. General objective

The general objective of this research is to determine the shear strength and behavior of beam-column joint under cyclic loading.

1.3.2. Specific objective

The specific objective is to investigate the effect of column axial load on the shear strength of exterior beam-column joint.

1.4. Scope

RC exterior beam-column joint was studied under constant displacement cycles. The main variable was the magnitude of column axial load. The modeling and analysis of RC exterior beam-column joint was studied by using finite element software. The contribution of floor slab to the joint shear strength and also interior and corner joints were not considered.

1.5. Organization of the Thesis

The thesis is organized into the following six chapters:

- The first chapter is discussed about the background, statement of the problem, objective of the thesis work, scope and limitations, as well as organization of the thesis.
- Chapter two is devoted to literature review, summarizes the previous researches which are concerned with the behavior of beam column joint.
- Chapter three discussed about design and detailing of a beam-column joint in accordance with current Ethiopian Building Code Standard [5].
- The fourth chapter discussed about modeling and analysis of exterior beam column joint using finite element software. This chapter included the assumption that used to simulate the geometrical and material property of selected joint.
- In chapter five the results obtained from numerical analysis are discussed (and interpreted).
- The last chapter, chapter six, contains conclusions based on the finding of the research and recommendations for further work.

2. LITERATURE REVIEW

Research on beam-column joint under different types of loading systems including earthquake loading were carried out in the last six decades with main goal was to improve the performance and capacity of the joint through new design concept and different detailing mechanisms. In this chapter those various literatures are reviewed and discussed.

2.1. Behavior of a Moment-Resisting Frame

Moment resisting frames are consisting of assemblage of beams and columns, with the beams rigidly connected to the columns. Those types of structures resist applied forces primarily by bending of their members and joints. If the joints are not designed and detailed properly, their performance can significantly affect the overall response of the moment-resisting frames. Due to the restriction of space available in the joint block, the detailing of joint reinforcement assumes more significance than elsewhere. One of the basic assumptions of the frame analysis is that the joints are strong enough to sustain the forces (i.e., moments, axial, shear forces and torsion) generated by the loading, and to transfer the forces from one structural element to another, (i.e., from beams to columns). The analysis with the assumption of joint being rigid fails to consider the effects of high shear forces developed within the joint because under seismic loading, shear forces introduced to the joint of frames are usually very much larger than those occurring under gravity loading [7].

Severe reverse cyclic loading due to earthquakes causes large inelastic deformations in the beam-column joints of high-rise buildings. It has recently been reported that many beam-column joint failures been observed in the 1980 El Asnam earthquake. Shear and anchorage failures, particularly at exterior joints, have also been identified after the 1985 Mexico, the 1986 San Salvador, and the 1989 Loma Prieta earthquakes [8].

2.1.1. Strong Column – Weak Beam Design Concept

Nowadays the seismic design philosophy is to provide the structure with properties that ensure the dissipation of the energy induced by an earthquake. The more energy dissipated, the less strength required by the structure. This means not only safer structures but also more economic ones. Regions of the primary lateral force resisting mechanism are carefully selected, designed and detailed so that they can dissipate as much as possible the energy transmitted to the structure by the base motions. In frames these regions are generally known as plastic hinges and together they form the energy dissipation mechanism of the structure. The energy is dissipated taking advantage of the ductile properties of the plastic hinges, i.e. their ability to maintain strength in the inelastic range and absorb energy by hysteretic behavior [9].

The successful performance of the structure in sustaining large imposed base motions depends mainly on the ability of the energy dissipation mechanism of the structure to hold during the entire seismic action. This is achieved assuring that:

- Each plastic hinge is designed to have strength as close as possible to the required strength and is carefully detailed to maintain its ductility.
- The only mode of failure of a member containing a plastic hinge is the one corresponding to the development of the capacity of the plastic hinge. Therefore all the other modes of failure are inhibited by providing them with strength greater than the capacity of the plastic hinge;
- In the same way, regions not suited to dissipate energy in a stable manner are protected by ensuring that their strengths exceed the requirements from the development of the plastic hinge strength. Therefore these regions are designed to remain elastic.

These three requirements are the basis for the so-called capacity design procedure and their applicability is exemplified in the following to the simple case of a multi-story two-dimensional frame. The plastic hinges of the mechanisms to be considered for this frame are supposed to dissipate energy by means of inelastic rotations in the end-section of members.

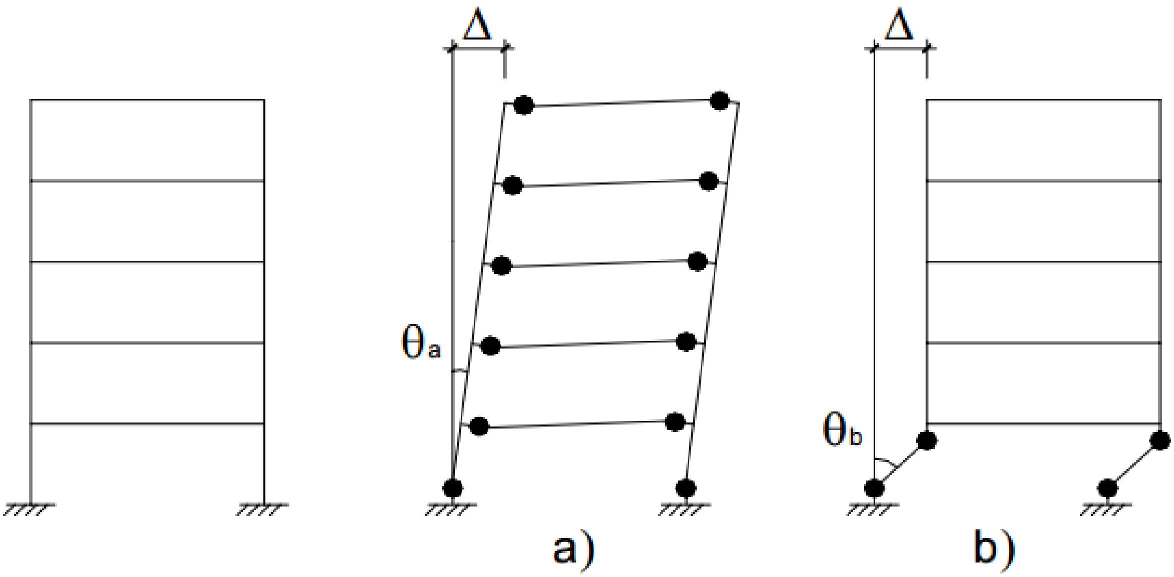


Figure 2-1 Application of the capacity design procedure to a multi- storey two-dimensional frame [8]

The major steps are summarized in the following [8]:

i. A kinematical admissible plastic mechanism at failure is chosen so that the maximum energy may be dissipated. Two fundamental criteria are used to define the most effective mechanism: in the first place the overall displacement of the structure should be achieved with the smallest inelastic deformation of the plastic hinges. Therefore the mechanism in Figure 2-1a is preferable since the inelastic rotations of the plastic hinges are considerably less than in the mechanism of Figure 2-1b for the same overall displacement Δ. In the second place in order to dissipate as much of the energy transmitted by the earthquake as possible a significant number of plastic hinges should form before collapse. For the present case it is obvious that the mechanism in Figure 2-1a is more advantageous.

ii. Parts of the structure intended to remain elastic are designed with respect to the situation of feasible action causing the development of the strength of the plastic hinges. Considering the frame in Figure 2-1, this means that the strength of the regions that are not plastic hinges must well exceed the required strength corresponding to the onset of the plastic moments in the plastic hinges. To assure this, a factor larger than unity is used, the overstrength factor. The latter is to take into account the variability of the yield stress on the reinforcement and the probability of strain-hardening effects that increase the strength of the plastic hinge after yielding.

iii. The nature and quality of detailing must be clearly distinct between the regions assigned to be plastic hinges and those which are to remain in the elastic domain. Structural members with adequate reinforcement, flexural yielding mechanisms are the ones presenting a more reliable ductile behavior opposing to the shear and bond-slip mechanisms. Therefore the plastic hinges in a capacity-designed structure should dissipate energy by means of flexural yielding rather than exploring inelastic shear and bond-slip deformations. This means that the designer should always assure that failure at the plastic hinge takes place by flexure before the exhaustion of shear capacity as well as bond strength. Capacity design is an important design tool which allows the engineer to choose and implement a satisfactory response despite the characteristics of the earthquake to which the structure is going to be submitted.

The strong column–weak beam concept is the corollary of the capacity design procedure and its fundamental importance in the design of structures whose seismic resistance system is composed by ductile frames. Considering the structural functions and modes of behavior of beams and columns, this concept establishes that the energy dissipation mechanism of the structure is composed by flexural plastic hinges taking place in beams and avoided in columns. Therefore, it is obvious that the strength of the beams is limited to the plastic hinge capacity and the columns are supposed to remain in the elastic domain. Column design moments are, according to this concept, derived at beam-column joints with respect to the actual resisting moments of the plastic hinges in the beams [9].

ES EN 1998-1:2015 [5] the requirement of strong column-weak beam is ensured by the following expression:

$$\sum \boldsymbol{M_{Rc}} \geq \mathbf{1.3} \sum \boldsymbol{M_{Rb}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots . 2\text{-}1$$

where,

$\sum \mathrm{M_{Rc}}$ is the sum of the design values of the moments of resistance of the columns framing the joint and

$\sum \mathrm{M_{Rb}}$ is the sum of the design values of the moments of resistance of the beams framing the joint.

The joint must follow the above design procedure and mechanisms to avoid plastic hinge in the joint region.

2.1.2. Classification of Joint in a Moment-Resisting Frame

A joint is defined as the portion of a column within the depth of the deepest beam that frames into the column. In a moment-resisting frame, joints are classified in terms of their location as interior joints, exterior joints and corner joints. When four beams frame into the vertical faces of a column, the joint is called as an interior joint. When one beam frames into a vertical face of the column and two other beams frame from perpendicular directions into the joint, then the joint is called as an exterior joint. When a beam frames into two adjacent vertical faces of a column, then the joint is called a corner joint [10].

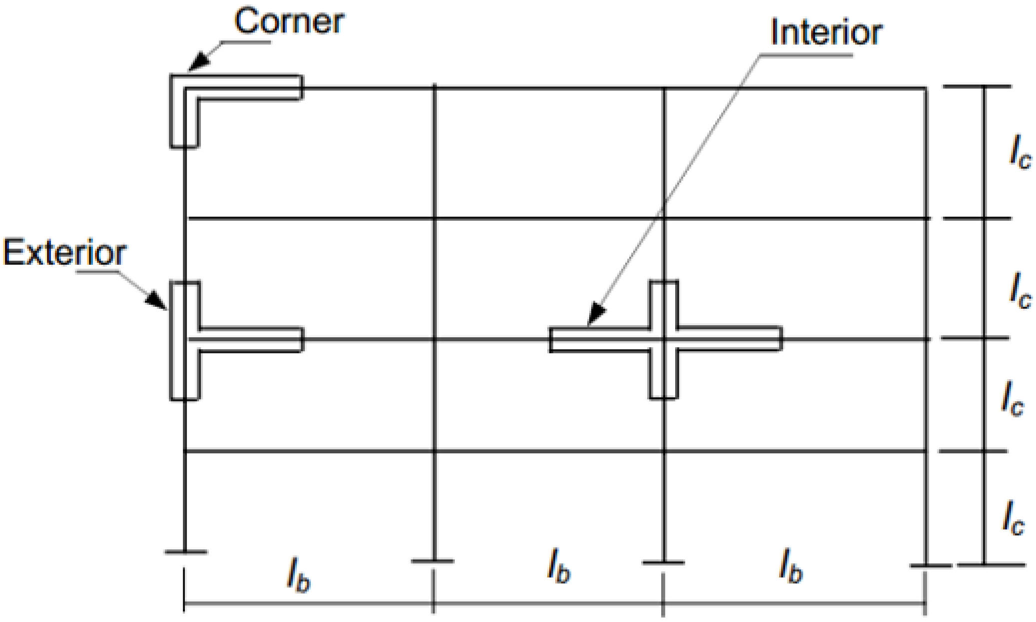


Figure 2-2: Typical planar view of frame with beam column joints [10]

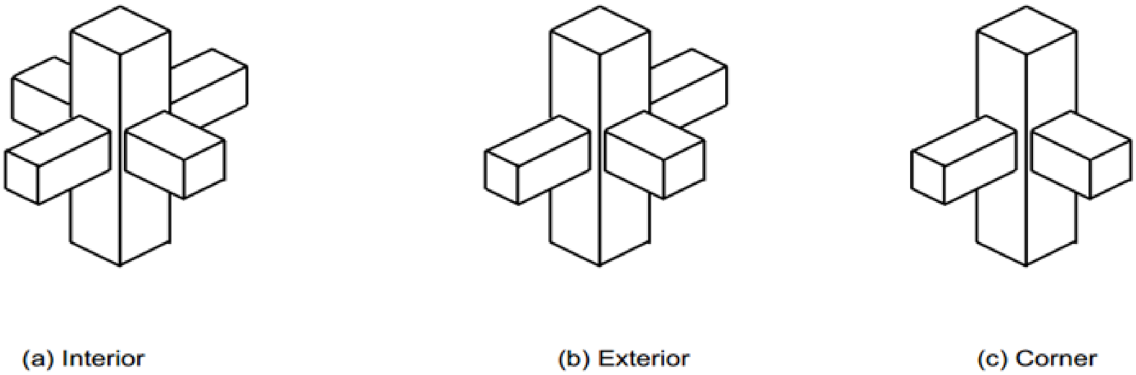


Figure 2-3 : Types of joint in a beam-column frame [10]

2.2. Modes of Joints Failure

A moment-resisting frame is expected to obtain ductility and energy dissipating capacity from flexural yield mechanism at the plastic hinges. A beam-column joint behavior is controlled by bond quality and shear failure mechanisms, which are weak sources for energy dissipation. The joint should have sufficient strength to enable the maximum capacities to be activated in the adjoining flexural members and the degradation of joints should be so limited such that the capacity of the column is not affected in carrying its design loads. A joint zone is one of the weak and critical sections in one structure system [11].

There are five types of failure that can occur at beam-column joint [12]. The first type represents connection failure by beam hinging as shown in Figure 2.3(a). This failure happens due to the formation of plastic hinges at the end of the beam in the joint zone. This condition occurs when the beam could not resist higher load and the reinforcement failed with the development of cracks. Although the joint zone may still resist the load; the beam failure will result in the failure of the joint core. The second type of failure is represent column-hinging failure as shown in Figure 2.3(b). This failure happens when the plastic hinges occur the column, either caused by shear force action or compressive force. As in the case of beam failure, a lot of cracks also can be seen in column failure. These kinds of failure have to be avoided because it can cause the frame to sway and hard to be repaired. The third failure type is caused by the spalling of concrete cover at a joint zone as shown in Figure 2.3(c). This happen because of cracks that occur at the joint face, where the cracked concrete burst when the load is increased. The spall-off concrete cover must be avoided because this can cause the decreasing in compressive load capacity of the column.

The fourth failure type, which is failure of anchorage bar inside the joint is shown in Figure 2.3(d). This failure takes place of exterior column joint. Reinforcements that have to resist negative moment in the beam must be anchored into the column as anchorage length. Inadequate anchorage length or poor detailing will cause this failure. Small radius of bend bars will produce high bearing stresses and contribute to anchorage failure. Frame structure cannot transfer the biaxial shear and decreased the structure capacity to absorb energy. The fifth failure type is joint shear failure as shown in Figure 2.3(e).

Loading at the beam will cause the horizontal shear force in column especially in the exterior beam-column joint. The combination of shear stresses and tensile forces also compressive forces at the beam reinforcement and column produced tensile stress and compression stress in the joint zone. The value of tensile stress may be greater when the joint segments reaches the ultimate capacity limitation, and this will cause cracks at the joint zone. This failure is almost same as anchorage bar failure, where the frame structure cannot transfer the biaxial shear and decreases the structure capacity to absorb energy [10].

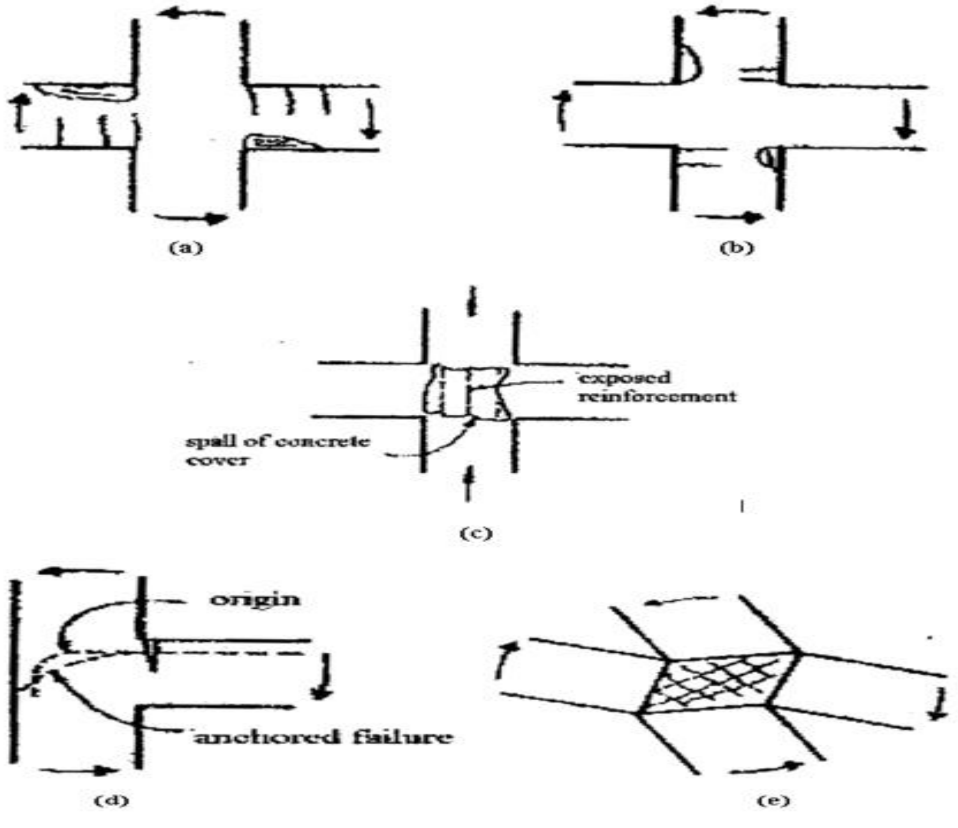


Figure 2-4: Types of failure mode in connection zone (a) Beam hinging failure, (b) Column hinging failure, (c) Failure by spall of concrete cover, (d) Anchorage failure, (e) Joint shear failure [12]

2.3. Shear Strength of Exterior Beam Column Joint under Cyclic Loading

Beam-column joints are critical regions in reinforced concrete frames designed for inelastic response to sever seismic attack. Beam-column joints of RC framed structure subjected to cyclic loads such as earthquakes experience large internal forces. Recent earthquakes have demonstrated that even when the beams and columns in a reinforced concrete frame remain intact, the integrity of the whole structure is compromised when the joints fail, where these members are connected [13]. Beam-column joints are susceptible to failure earlier than the adjacent members due to the destruction of joint zone. This failure is mainly for the external joints. Therefore, ductility and energy absorption capacity of the beam-column joints are of paramount importance in the seismic resistance of structures. Further, reinforcing bars have to meet the requirements of strength and ductility under the repeated reversed deformations. Also, while designing the joint core, it is necessary to verify the shear resistance and anchorage conditions of the reinforcement passing through the joint region [13] , [14].

As a consequence of opposite sign seismic moments in the columns immediately above and below the joint, and similarly beam moment reversal across the joint, the joint region is subjected to horizontal and vertical shear forces which magnitude is typically many times higher than in the adjacent beams and columns [8], [15]. This high shear force that is produced by adjacent members causes joint shear failure as well as structural damage.

In designing a building to withstand a severe earthquake, it is necessary that seismic energy be absorbed and dissipated through large but controllable inelastic deformations of the structure. The sources of potential brittle failure must be eliminated. Thus, it is necessary to prevent premature crushing and shearing of concrete as well as sudden loss of bond and anchorage. To utilize the energy-dissipating capacity of structural members, a joint connecting the beams and columns must function without having brittle failure and without excessive loss of stiffness [1].

However, experimental test results shows that joints collapsed showing an appreciable diagonal crack opening because of the cyclically alternating force at the tip of the beam. Due to increased crack width, the distance of opposite corners of the joint panel increases with horizontal load or with the number of cycles in the inelastic range [15].Therefore, a joint may loss the shear strength capacity, when it’s subjected to severe reverse cyclic loading due to earthquake.

2.3.1. Influence of Cyclic Loading on Shear Strength of the Joints

The monotonic shear resistance should be significantly higher than the flexure one, because under cyclic conditions shear strength and stiffness deteriorate much faster than the flexural, so shear deformations may become dominant with cycling and failure may take place at interesting inclined cracks. Cycling causes a degradation of strength with respect to the envelope provided by the virgin loading curves. This strength degradation is more evident between one cycle of deformation and the next, at the same level of peak deformation [16].

The key point of a seismic performance for beam–column joints is to ensure and maintain the energy absorption capacity of plastic hinges of adjoining members in structure designed using strong column weak beam concept, plastic hinges may form in the beam. The hinges may limit the shear imposed on the joint and avoiding any shear or anchorage failure in the joint core. By contrast, due to the small value of shear span to depth ratio, the strength of external beam–column joints can be governed by shear rather than flexural strength [1], [17].

Therefore, joints must be capable of resisting repeated reversal load or cyclic load that are caused by earthquake without excessive damage and shear failure. But under earthquake shaking the beams adjoining a joint are subjected to moments in the same (clockwise or counterclockwise) direction Figure 2.4 Under these moments, the top bars in the beam-column joint are pulled in one direction and the bottom ones in the opposite direction Figure 2.5a These forces are balanced by bond stress developed between concrete and steel in the joint region. If the column is not wide enough or if the strength of concrete in the joint is low, there is insufficient grip of concrete on the steel bars. In such circumstances, the bar slips inside the joint region, and beams lose their capacity to carry load [18].

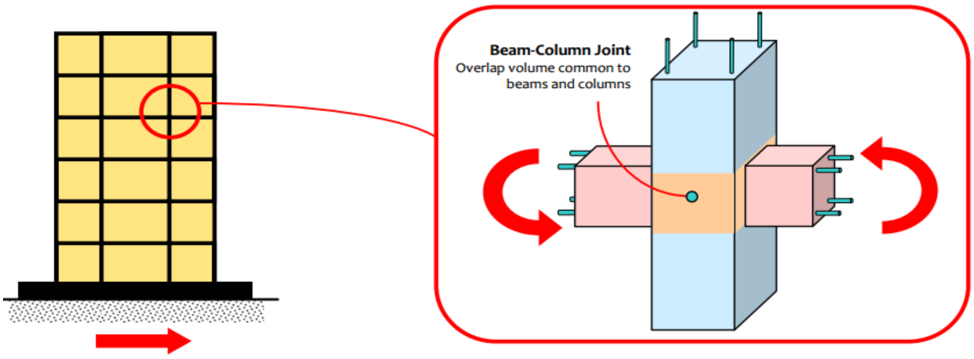
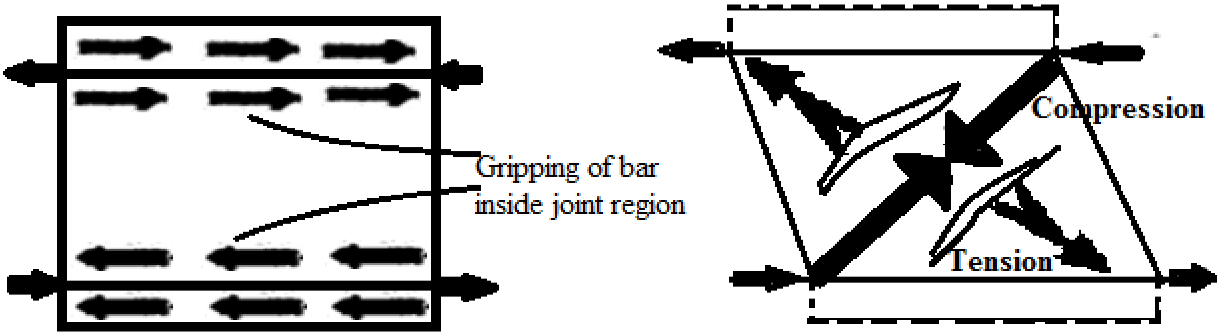


Figure 2-5: Beam –Column Joints are critical parts of a building [18]

Further, under the action of the above pull-push forces at top and bottom ends, joints undergo geometric distortion; one diagonal length of the joint elongates and the other compresses and this force causes irreparable damage in joints under strong seismic shaking as shown in Figure 2.5(b).If the column cross-sectional size is insufficient, the concrete in the joint develops diagonal cracks [18].

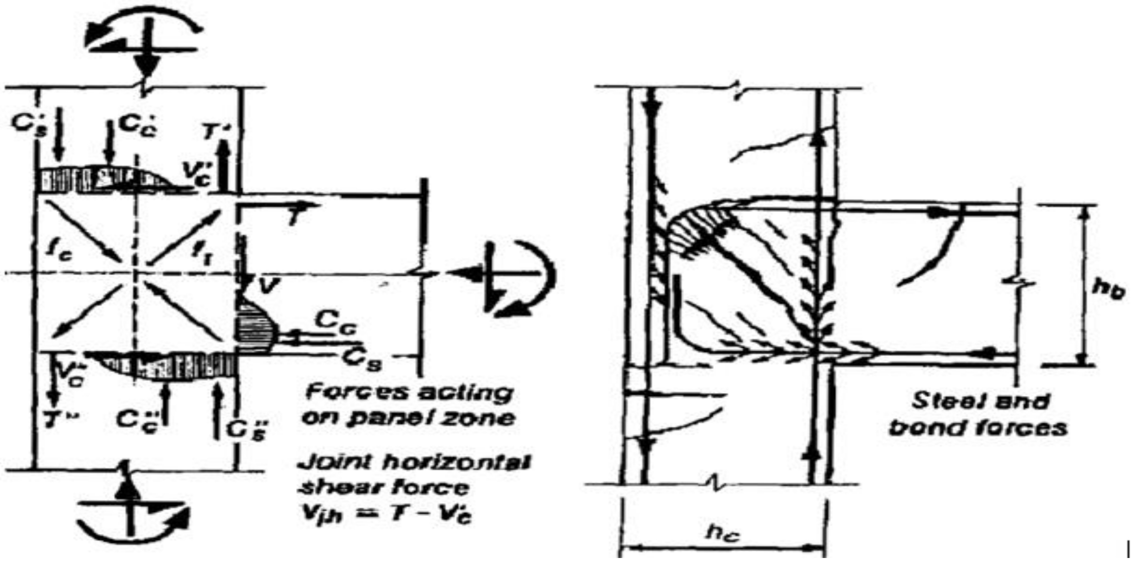


(a) Loss of grip on beam bars in joint region: large column width and good concrete in holding the beam bars.

(b) Distortion of joint: causes diagonal cracking and crushing of concrete.

Figure 2-6 : Cause of pull-push forces on joint

The internal forces acting on a reinforced concrete exterior beam-column joint under cyclic loading (or seismic action) is shown in Figure 2.6 (a). After diagonal tension cracking in the joint core, the beam and column forces are transferred across the joint core by a diagonal compressive strut and a truss mechanism consisting of a concrete diagonal compression field and horizontal and vertical reinforcement in the joint core, as shown in Figure 2.6 (b) [19].



(a) Force from beams and columns acting on joint core

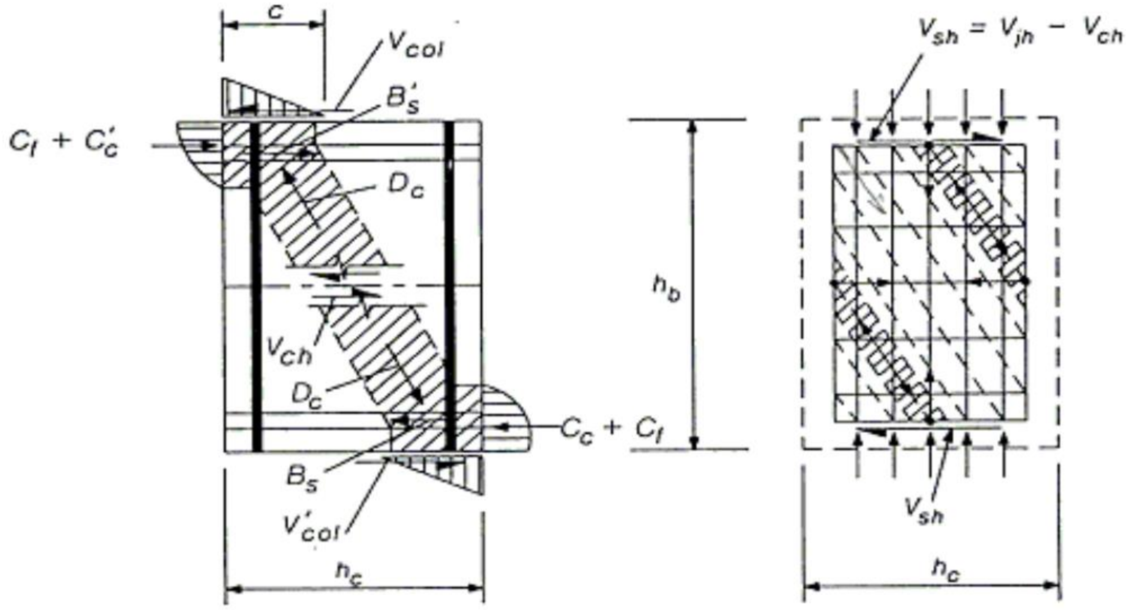
(b) Crack pattern and bond bearing forces after diagonal tension cracking

Figure 2-7: Cyclic behavior of exterior beam-column joint

2.3.2. Shear Transfer mechanisms

Under seismic action large shear forces may be introduced into beam-column joints irrespective of whether plastic hinges develop at column faces or at some other section of the beams. These shear forces may causes a failure in the joint core due to the breakdown of shear or bond mechanisms or both [8]. Therefore, the shear strength of joint depends on the capacity of shear resisting mechanisms.

This mechanism consists of a diagonal concrete strut action and a truss action as shown in Figure 2.7.



(a) Strut Mechanism (b) Truss Mechanism

Figure 2-8: Shear resisting mechanisms [10]

- **Strut mechanism**

The diagonal concrete strut mechanism is formed by the major diagonal concrete compression force in the joint. This force is produced by the vertical and horizontal compression stresses and the shear stresses on concrete at the beam and column critical sections, the strength of the strut mechanism depends on the compressive strength of concrete [10]. In exterior joint, the beam compression zone at one end of the strut must be developed by the anchored reinforcement [1].

- **Truss Mechanism**

The truss mechanism is formed by a combination of the bond stress transfer along the beam and column longitudinal reinforcement, the tensile resistance of lateral reinforcement and compressive resistance of uniform diagonal concrete struts in the joint panel and that of the truss mechanism on the tensile yield strength of the lateral reinforcement crossing the failure plane [10].

In resisting the joint shear, the diagonal strut mechanism can exist without any bond stress transfer along the beam and column reinforcement within the joint, while the truss mechanism can develop only when a good bond transfer is maintained along the beam and column reinforcement. Under seismic loading conditions, the bond along the beam reinforcement inevitably deteriorates especially after beam flexural yielding takes place unless the strength and size of the reinforcement is strictly restricted. With the outset of bond deterioration, the truss mechanism starts to diminish and the diagonal strut mechanism must resist the most dominant part of the joint shear. Under these conditions, the tension force in the beam reinforcement not transferred to the joint concrete by bond must be resisted by the concrete at the compression face of the joint, thus increasing the compression stress in the main strut. The concrete strut is progressively weakened by the reversed cyclic loading. At the same time, the compressive strength of the concrete is reduced by the increasing tensile strain perpendicular to the direction of main strut. The combination of these two phenomenon results in the failure of the concrete strut in shear compression. The principal role of the lateral reinforcement in this case is to confine the cracked core concrete [10].

2.3.3. Factors Affecting the Shear Strength of Exterior Beam Column Joint

Shear strength of the joint maintained by many parameters, those parameters have significant contribution on resisting mechanisms to joint expansion. To study the Shear strength of the joint under cyclic loading; identifying the effect of those parameters are important.

- **Concrete contribution**

Concrete strength has a direct influence on the joint shear strength using the strut mechanisms. Researchers demonstrated that the strength of the joint could be controlled by failure of the concrete at compression value analogous to that of an axially loaded concrete column. Under cyclic reversed loads on the joint, the shear strength gradually decreased due to exhaustion of the compressive concrete strength in the inclined direction .Therefore, the degradation of concrete strength results in a decrease in joint shear strength [1].

- **Stirrups contribution**

There is general agreement that transverse reinforcement improves the shear capacity of joints. From a large number of experimental data researchers concluded that joint reinforcement provides Confinement and shear resistance to the joint and increases the anchorage characteristics of beam Bars in the joint [1].

- **Column axial load contribution**

Because of limited amount of available experimental data; no discernable correlation was established between the maximum joint shear stress and the nominal column axial stress [4].Researchers have different perspective on the effect of column axial load on joint shear strength. Some researchers concluded that column axial load have insignificant effect on the strength of the joint. On the other hand, other experimental reports show that, high axial load on columns increases joint strength. Based on the previous research the effect of column axial load on the joint is summarized below as positive and negative,

- **Positive effect of axial load**

Clyde et al. [4] investigated the behavior of reinforced concrete exterior beam –column joint in a shear-critical mode. Experimentally, the effect of axial load on the shear strength of joint is studied by testing four half –scale RC exterior joint, which is subjected to quasi-static cyclic loading. They concluded that the joint strength coefficient, γ, changes with the variation of the column axial load. As column axial load increases the joint strength coefficient γ also increases. In other words increasing the value of axial column load improves the shear strength of the joint.

$$\gamma = \frac{V_n}{\sqrt{f'_c}A_j} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2\text{-}2$$

Where;

γ is joint strength coefficient,

f'_c is concrete compressive strength,

V_n is nominal joint shear force, and

A_j is effective horizontal area of the joint

Haach et al. [3] studied the influence of the column axial load on the joint shear strength through numerical simulations. The numerical study is performed through the software Abaqus, based on Finite Element Method. A comparison of the numerical and experimental results is presented in order to validate the simulation. The results showed that the column axial load made the joint more stiff but also introduced stresses in the beam longitudinal reinforcement.

Park and Mosalam [2] stated that from the aspect of the diagonal strut mechanism in the joint region, a compressive diagonal strut Width is determined by the compression block depth of the column and the beam. The column compression block depth is obviously increasing with the increase of the column axial load. Thus, increasing the column axial load has a positive effect on improving joint strength. Applying Principal tensile stress as the failure criterion, it is also concluded that high column axial load decreases this principal tensile stress and, accordingly, increases joint shear strength. As another Positive effect, high column axial load improves bond strength between the beam reinforcing bars and the surrounding concrete. This leads to increasing joint shear strength because the horizontal shear force is transferred into the joint by bond and anchorage of the beam reinforcement.

- **Negative effect of axial load**

Park and Mosalam [2] studied semi-empirical model and an analytical model is suggested to predict the shear strength of reinforced concrete (RC) exterior beam-column joints without transverse reinforcement (denoted as unreinforced) in the joint region. A large experimental data set of unreinforced exterior beam-column joints from published literature were collected using consistent criteria. The effect of joint aspect ratio, the effect of beam reinforcement and the effect of column axial load are the parameter that they studied. They investigated from a large expermental database and proposed that the column axial load has little influence on the joint shear strength of unreinforced beam column joints.

Bindhu et al. [14] studied the performance of exterior beam-column joint under seismic type loading. A six story RC building in zone III is analyzed and one of the exterior beam column joint at intermediate stories is designed for adequate shear strength in the joint and to satisfy the strong column-weak beam theory. The spacemen were tested under a reverse cyclic loading and under two different axial load then the performance of exterior beam column joint and the effect of column axial load on the behavior of the joint is studied and concluded that an increase in the column axial load improves the load carrying capacity and stiffens the joints. However, this reduces the energy absorption capacity and ductility of the joint.

Another negative effect of axial loading is the development of the well-known second-order moments (P-Δ effects as it is generally known). It is obvious that as the level of axial loading rises, the more important are these effects and therefore, the larger are the strength requirements. The designer should avoid high levels of ductility demand in members subjected to high axial loading in order to minimize the risk of failure due to large second order moments. Underestimating the P-Δ effects is a frequent cause of failure as it leads to structural collapse due to lateral instability, particularly in buildings in which sides way mechanisms are supposed to form [9].

- **Anchorage contribution**

In exterior joints the beam longitudinal reinforcement that frames into the column terminates within the joint core. After a few cycles of inelastic loading, the bond deterioration initiated at the column face due to yield penetration and splitting cracks, progresses towards the joint core. Repeated loading will aggravate the situation and a complete loss of bond up to the beginning of the bent portion of the bar may take place. The longitudinal reinforcement bar, if terminating straight, will get pulled out due to progressive loss of bond. The pull out failure of the longitudinal bars of the beam results in complete loss of flexural strength. This kind of failure is unacceptable at any stage. Hence, proper anchorage of the beam longitudinal reinforcement bars in the joint core is of utmost importance. The pull out failure of bars in exterior joints can be prevented by the provision of hooks or by some positive anchorage [10].

- **Longitudinal beam reinforcement**

Being beam reinforcement subjected to large alternate cyclic actions making nonlinear steel behavior, it is difficult to evaluate the maximum stress acting on the two reinforcement levels (at top and bottom), as far as the maximum confinement effect due to these bars. It is assumed that the joint shear resistance due to the confinement action provided by beam longitudinal reinforcement, should be proportional to the yielding strength and to the average area of the beam longitudinal reinforcement, $\mathrm{f_{yh}}$ and $\mathrm{A_{sh}}$.Consequently the shear strength due to beam reinforcement is

$$v_{jh,l} = k_1 * \frac{f_{yh} . A_{sh}}{A_g} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{2-3}$$

Where, k_1 is a numerical coefficient to be determined, Ag is the area of the joint transverse section, $\mathrm{f_{yh}}$ and $\mathrm{A_{sh}}$ are yielding strength and average area of the beam longitudinal reinforcement respectively [15].

2.3.4. Column Axial Load Effect on Shear Strength of Beam Column Joint

The failure criteria for joint are usually associated with principal stresses either in terms of diagonal tensile stresses or diagonal compressive stresses of strut exceeding certain values. The significance of representing joint capacity in terms of principal stresses has been discussed which recognizes the axial load acting on the column. A critical situation where the axial load on the column is very large, diagonal compression failure of strut occurring before the first diagonal tensile cracking in the joint has been cautioned. Therefore, it is very essential to account for the axial load effect in limiting the joint nominal shear stress [20] , [21] .

The nominal shear stresses associated with such principal stresses are expected to vary with respect to the axial load on the column. Figure 2.8 shows different code provisions for nominal shear stress for varying axial load ratios for both interior and exterior joints. It can be seen that ACI code allows higher nominal shear stress and NZS code limits to a lesser value, and both are not affected by axial loads. On the other hand, Limiting value of nominal shear stress as per EN code decreases as the axial load increases. Especially for exterior joints, where the variation of axial loads acting on the column could be high during seismic event [22].

The formula used to calculate the shear strength of the joint are differs from code to code. The three well established codes regarding to design and detailing aspects of beam column joints such as ACI 318M-02, NZS 3101: Part 1:1995 and the Eurocode 8 of EN1998-1:2003, those codes are evaluates the nominal shear capacity based on strut mechanism and express it as a function of concrete strength irrespective of the amount of shear reinforcement. However, the nominal shear capacity is influenced by the confinement provided by the adjoining members. A beam member that frames into a face is considered to provide confinement to the joint if at least the framing member covers three-quarters of the joint. ACI code suggests $1.7\sqrt{f_c'}\,A_j$ if confined on four faces, $1.25\sqrt{f_c'}\,A_j$ if confined on three faces and $1.0\sqrt{f_c'}\,A_j$ for other cases. The NZS has suggested a limiting of $0.2f_c'A_j$, with respect to strut mechanism irrespective of confinement offered by the framing members [22]. EN code has limited the nominal shear stress V_{jhd}

$$V_{jhd} \leq \eta f_{cd}\sqrt{1-\frac{v_d}{\eta}}\;b_j\,h_c \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2\text{-}4$$

Where;

v_d is the normalized axial force in the column above the joint; and

η denotes the reduction factor on concrete compressive strength due to tensile strains in transverse direction.

$\eta = 0.6\left(1-\frac{f_{ck}}{250}\right)$;

f_{ck} is given in MPa.

The shear strength of exterior joints is taken as 80% of the value given by Eq. 2.3

where:

The effective joint width b_j is:

a) if $b_c > b_w$: $b_j = \min\{b_c; (b_w + 0.5 * h_c)\}$;

b) if $b_c < b_w$: $b_j = \min\{b_w; (b_c + 0.5 * h_c)\}$

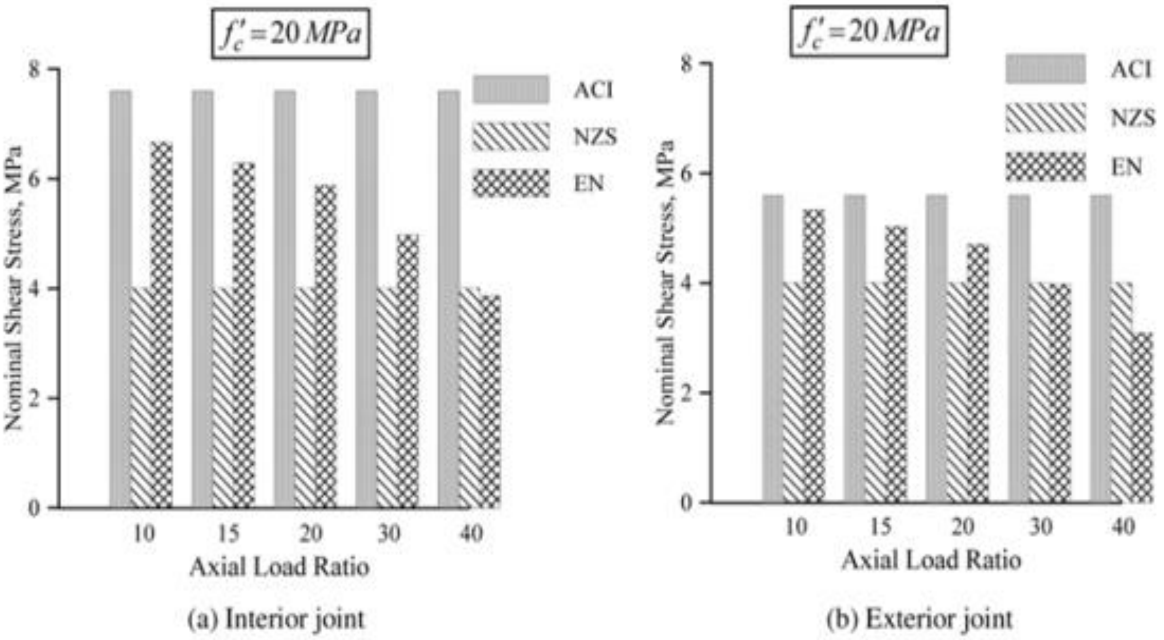


Figure 2-9 : Effect of axial load on nominal shear stress [22]

The magnitude of column axial load also determine the mode failure of beam column joint. If the shear forces transmitted from beam and column to the joint are large and little or no axial load is exerted on the column, shear failure may occur at the joint. If the shear forces are small and large axial load is exerted, compression failure may occur at the joint but sometimes it is hard to distinguish between shear failure and compression failure and it’s difficult to predict the exact failure mode of a specimen because of the influence of many factors [1].

Therefore, the relationship between the effects of column axial load and the capacity of shear strength of joint is still the issue that needs special consideration on design and detailing of beam column joint especially for earthquake prone area.

2.4. Abaqus modeling of Reinforced Concrete Beam-Column Joint

2.4.1. General

The use of FEA tools has become widespread due to increased computation power and the ability of FEA software packages to simulate incredibly complicated components, structures and systems under a wide variety of situations and loading conditions. Numerical simulation is very useful to study elements that are difficult to investigate by experiments, such as joints with high column axial load. Some variables such as stress distribution in concrete are difficult to measure during the experiments and can easily be evaluated by numerical simulations [3].

Abaqus is a Finite Element Analysis (FEA) software package with different products designed for different applications within its own suite. In pre-processing stage, geometry and boundary conditions, element types, material properties and nonlinear analysis solution are defined and in post-processing stage visualizations of result is obtained. Finite element simulation of the complex behavior of concrete as a non-homogeneous and anisotropic material is a challenge in the finite element analysis of reinforced concrete structures and their components. Among constitutive models defining concrete nonlinear behavior as a quasi-brittle material available in ABAQUS the Concrete Damage Plasticity (CDP) model is better to define the concrete nonlinear behavior [23].

Concrete damaged plasticity model (CDP) is used for defining concrete in plastic range. The concrete damaged plasticity model provides a general capability for modelling concrete and other quasi-brittle materials in all types of structures. This model uses concepts of isotropic damaged elasticity in combination with isotropic tensile and compressive plasticity to represent the inelastic behavior of concrete. The concrete damaged plasticity model is based on the assumption of scalar (isotropic) damage and is designed for applications in which the concrete is subjected to arbitrary loading conditions, including cyclic loading. The model takes into consideration the degradation of the elastic stiffness induced by plastic straining both in tension and compression. It also accounts for stiffness recovery effects under cyclic loading [16].

When used CDPM some parameters is must be defined such as shape factor($\mathrm{K_C}$), Eccentricity (Ɛ), dilation angle (ψ), stress ratio ($\frac{\sigma_{bo}}{\sigma_{co}}$) and viscosity parameters (μ). Parameter $\mathrm{K_C}$ is interpreted as a ratio of the distances between the hydrostatic axis and respectively the compression meridian and the tension meridian in the deviatoric cross section. This ratio is always between 0.5 and 1 and it's taken as 0.667 in most of literatures. Second parameter is eccentricity (plastic potential eccentricity). It is a small positive value which expresses the rate of approach of the plastic potential hyperbola to its asymptote. In other words, it is the length (measured along the hydrostatic axis) of the segment between the vertex of the hyperbola and the intersection of the asymptotes of this hyperbola (the center of the hyperbola). Parameter eccentricity can be calculated as a ratio of tensile strength to compressive strength. The CDP model recommends to assume e = 0.1. Another parameter describing the state of the material is the point in which the concrete undergoes failure under biaxial compression. It is a ratio of the strength in the biaxial state to the strength in the uniaxial state $(\frac{\sigma_{bo}}{\sigma_{co}})$ and dilation angle ψ is interpreted as a concrete internal friction angle. The ABAQUS user's manual specifies default value of $(\frac{\sigma_{bo}}{\sigma_{co}})$ =1.16, dilation angle ψ = 35° and viscosity parameters μ=0 respectively [1].

The main two failure mechanisms of the concrete material are tensile cracking and compressive crushing according to fundamental assumptions of damage plasticity model. The evolution of the yield (or failure) surface and the degradation of elastic stiffness in damage plasticity model are controlled by two hardening variables which are tensile and compressive equivalent plastic strains $\varepsilon_t{}^{pl}$ and $\varepsilon_c{}^{pl}$ and the behavior of concrete both in compression and in tension is defined by the users. Stiffness recovery is an important aspect of the mechanical response of concrete under cyclic loading. Abaqus allows direct user specification of the stiffness recovery factors w_t and w_c .The experimental observation in most quasi-brittle materials, including concrete, is that the compressive stiffness is recovered upon crack closure as the load changes from tension to compression. On the other hand, the tensile stiffness is not recovered as the load changes from compression to tension once crushing micro-cracks have developed. This behavior, which corresponds to tension recovery w_t =0 and compression recovery w_c =1, is the default used by Abaqus [6].

When the concrete specimen is unloaded from any point on the strain softening branch of the stress-strain curves, the unloading response is weakened: the elastic stiffness of the material appears to be damaged (or degraded). The degradation of the elastic stiffness is characterized by two damage variables, dt and dc , which are assumed to be functions of the plastic strains, temperature, and field variables:

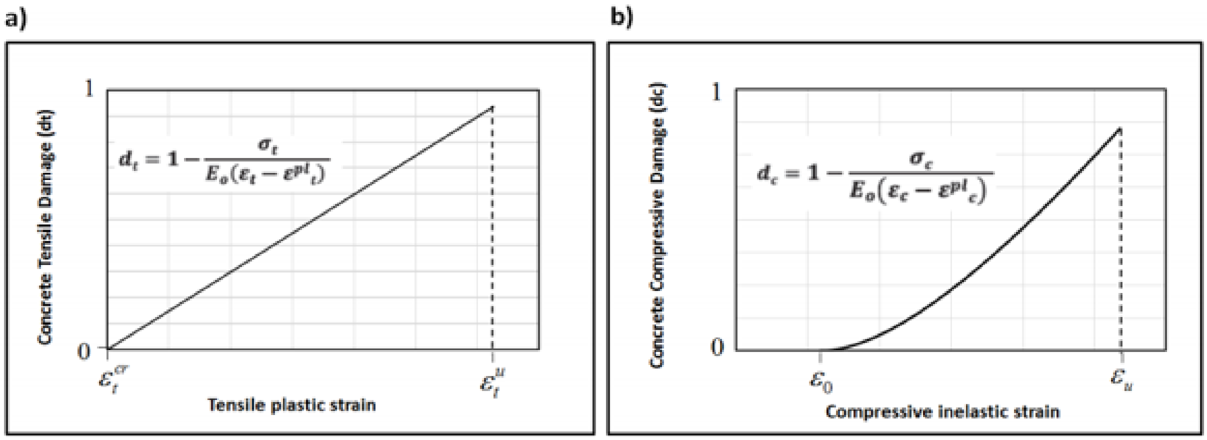
$$dt = dt(\varepsilon_t^{pl}, \theta . f_i); \quad 0 \le d_t \le 1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2\text{-}5$$

$$dc = dc(\varepsilon_t^{pl}, \theta . f_i); \quad 0 \le d_c \le 1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2\text{-}6$$

The damage variables can take values from zero, representing the undamaged material, to one, which represents total loss of strength. If E_o is the initial (undamaged) elastic stiffness of the material, the stress-strain relations under uniaxial tension and compression loading are, respectively:

$$\sigma_t = (1 - dt)E_o\,(\varepsilon_t - \varepsilon_t^{pl}), \ldots \ldots \ldots 2\text{-}7$$

$$\sigma_c = (1 - dc)E_o\,(\varepsilon_c - \varepsilon_c^{pl}), \ldots \ldots \ldots 2\text{-}8$$



a. uniaxial tensile damage b. uniaxial compressive damage

Figure 2-10: Definition of damage parameter in CDP model [6]

In Abaqus reinforcement in concrete structures is typically provided by means of rebar's, which are one-dimensional rods that can be defined singly or embedded in oriented surfaces. Rebar's are typically used with metal plasticity models to describe the behavior of the rebar material and are superposed on a mesh of standard element types used to model the concrete. With this modeling approach, the concrete behavior is considered independently of the rebar. Effects associated with the rebar/concrete interface, such as bond slip and dowel action, are modeled approximately by introducing some "tension stiffening" into the concrete modeling to simulate load transfer across cracks through the rebar [6].

The embedded element technique can be used to model rebar reinforcement. ABAQUS searches for the geometric relationships between nodes of the embedded elements and the host elements. If a node of an embedded element lies within a host element, the translational degrees of freedom at the node are eliminated and the node becomes an "embedded node."

The translational degrees of freedom of the embedded node are constrained to the interpolated values of the corresponding degrees of freedom of the host element. Embedded elements are allowed to have rotational degrees of freedom, but these rotations are not constrained by the embedding [16].

For cyclic analysis abaqus explicit is more efficient than other types of Abaqus product. This product provides nonlinear, transient, dynamic analysis of solids and structures using explicit time integration. Its powerful contact capabilities, reliability, and computational efficiency on large models also make it highly effective for quasi-static applications involving discontinuous nonlinear behavior. For quasi static simulation incorporating rate-independent material behavior, the natural time scale is generally not important.to achieve an economical solution, it is often useful to reduce the period of analysis or to increase the mass of the model artificially (mass scaling).Both alternatives yield similar result for rate independent material, although mass scaling is the preferred means of reducing the solution time if rate dependencies are included in the model because the natural time scale is preserved [6].

2.4.2. RC Beam-Column Joint Performed Using Abaqus

Now a days solving problems using numerical simulation is adapted. Abaqus is one of commercial software that can be used for analysis based on nonlinear behavior of the structures. There are many researches conducted using abaqus and the results obtained from analysis is nearly the same as that conducted by experiments [3], [12], [23].

Haach et al. [3] evalute the influnce of column axial load on the joint strength by taking monotonically loaded exterior beam-column joint.This study is made by using finite element software Abaqus.They compare the numerical and expermental result in order to validated the simulation and the numeriacal result confirmed the expermental observation and allowed analysing other aspects that could not evaluted in the test.They generalized that the numerical (Abaqus) model represented adequately the global behavior of experimental tests and it allow parametrical study that can’t be reached using experment.

M.A. Najafgholipour et al. [23] they investigated the joint shear failure mode in terms of joint shear capacity, deformations and cracking pattern using nonlinear finite element analysis (FEA) software Abaqus. A 3D finite element model capable of appropriately modeling the concrete stress-strain behavior, tensile cracking and compressive damage of concrete and indirect modeling of steel-concrete bond is used.They used concrete damage plasticity model in order to define nonlinear behavior of concrete material and define concrete compression behaiour by using Hognestad type parabola and concrete tensile bahaviour was defining using linear stress-strain relationships which is proposed by Belarbi and Thomas (1994).Finite element model is then verified against experimental results of two non-ductile beam-column connections (one exterior and one interior) which are vulnerable to joint shear failure. The comparison between experimental and numerical results indicates that the FE model is able to simulate the performance of the beam-column connections and is able to capture the joint shear failure in RC beam-column connections.

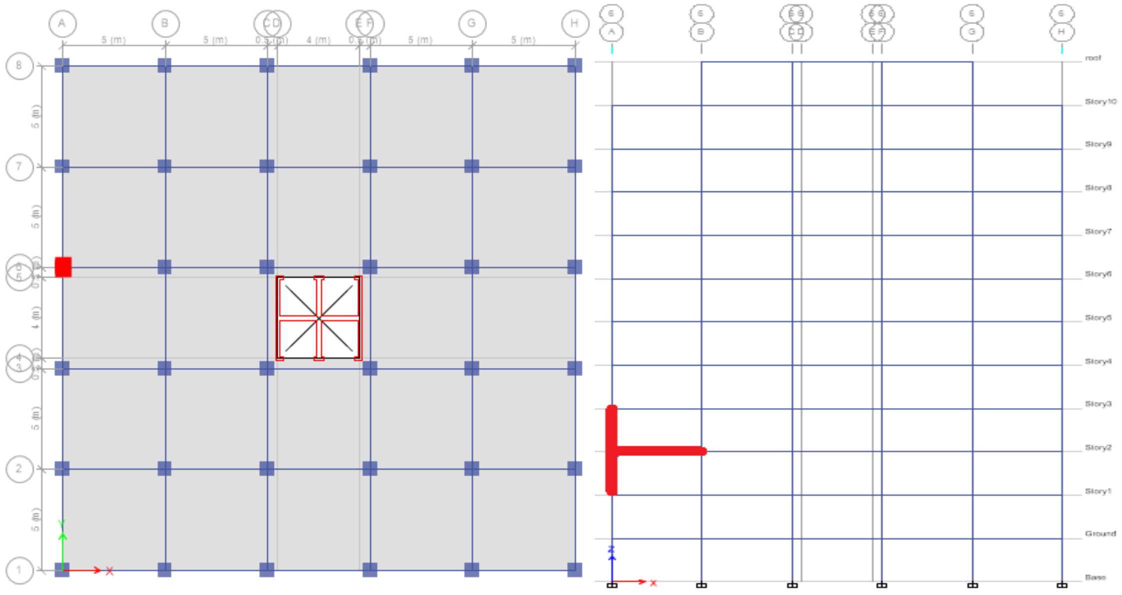
3. ANALYSIS AND DESIGN OF BEAM COLUMN JOINT

3.1. Analysis of Frame using ETABS Software

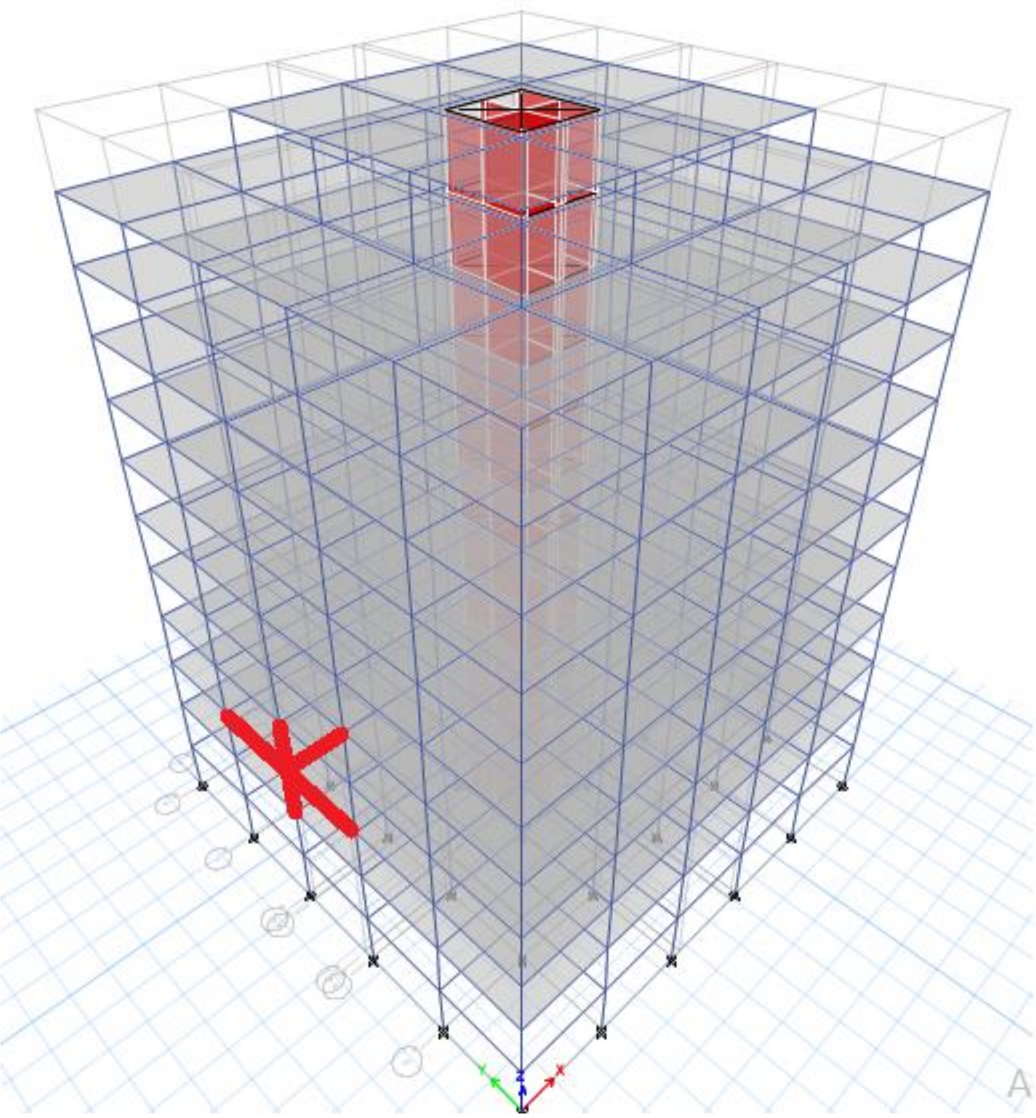
G+10 moment resisting frame is considered for this analysis which is located in Addis Ababa for office purpose and covers 625m^2 area. The frames are designed to carry the vertical loading as well as the moments resulted from seismic. Thus, frames are designed to resist an anticipated ground acceleration of 0.1g. The assumed size of the columns are 700mmx700mm and the size of the beams are 400mmx500mm. The floor heights are taken as 3m each floor throughout the building and the foundation assumed to be structurally rigid.

Concrete compressive strength considered is C25/30 and the reinforced steel f_{yk}=400MPa.Static lateral force method is used for Earthquake analysis because the structure is regular in plan and in elevation and the height of the building is less than 40m and also the fundamental period of the building is less than 2 seconds. The structure is designed and detailed for ductility class DCM.

After analysis of the frame, the design moment, shear and axial load from the critical load combination for earthquake ground motion in the X direction are estimated for the selected exterior beam column joint as shown in the Figure 3.1 (b) and the joint is detailed for capacity design philosophy as per ES EN 1998-1:2015 [5].



(a) (b)



(c)

Figure 3-1: Structural layout: (a) plan of the building, (b) elevation of the frame with exterior beam column joint, (c) 3D model of the building

3.2. Detailing of Beam-Column Joint according to ES EN 1998-1:2015

The specimen dimension and reinforcement detail of selected exterior beam column joint is shown in the Figure 3.2. The horizontal confinement reinforcement in joints of primary seismic beam with columns is provided according to ES EN 1998-1:2015 [5].

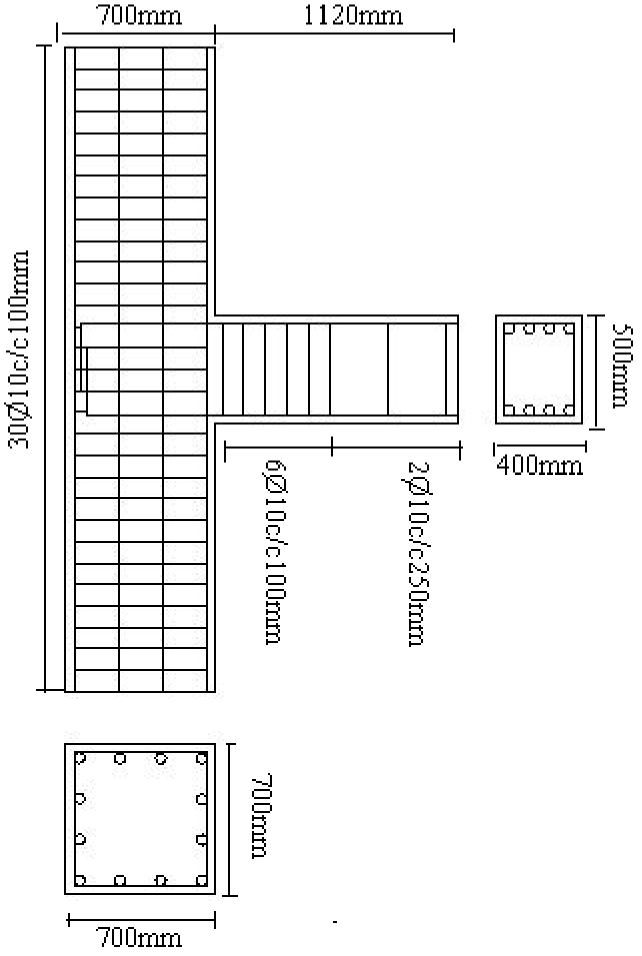


Figure 3-2: Specimen dimensions and reinforcement details

The beam is 400mm wide and 500 deep. It is symmetrically reinforced with 4ø24 bars for both the positive and negative reinforcement. Each longitudinal bar has a 300mm hook bent at 90°. The bottom beam reinforcement is bent up into a hook at the joint, and the top reinforcement is bent down into a hook at the joint. The hooks overlap approximately 190mm as shown in Figure 3.2. The transverse reinforcement is ø10 bar closed stirrup with 135° bend and 100mm extension on both ends as shown in Figure 3.3. The stirrups are spaced at 100mm along the beam critical region and 250mm outside the critical region.

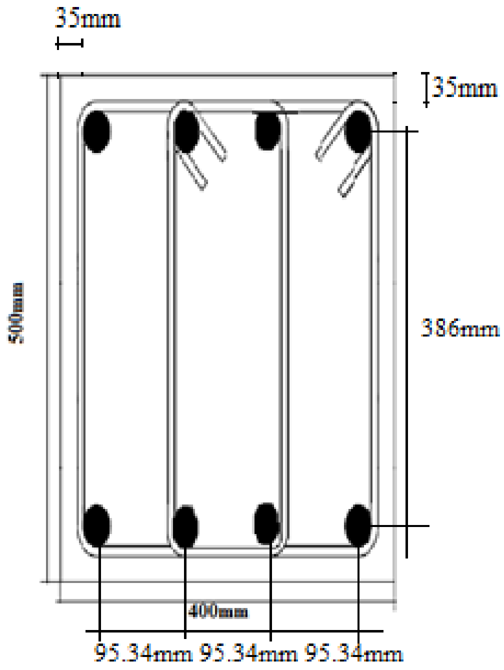


Figure 3-3: Beam cross section

The column is 700mm wide and 700 deep. It is reinforced with 12ø24 bars evenly distributed around the perimeter of the column. The transverse reinforcement in the column consists of ø10 bar closed stirrups with 135° bends and 100mm extensions on both ends; this is shown in Figure 3.4. The spacing of the stirrups are 100mm throughout the column as shown in the Figure 3.2.

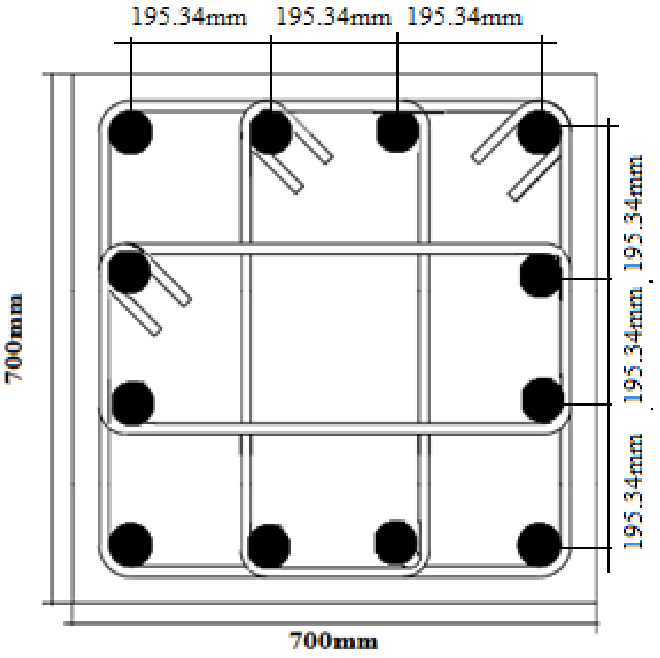


Figure 3-4: Column cross section

4. MODELING AND ANALYSIS OF EXTERIOR BEAM COLUMN JOINT USING ABAQUS

Abaqus have different modules, each module contain only those tools that are relevant to a specific portion of the modeling task. The order of the modules in the menu and in the model tree corresponds to the logical sequence that the user follow to create a model. The order of the modules is part used to create individual part by sketching or importing their geometry, property used to create section and material definitions and assign them to regions of parts. Assembly used to create and assemble part instances, Step module is used create and define the analysis steps and associated output requests, Interaction module is used to specify the interaction, such as contact, between regions of a model, Load module used to specify loads, boundary conditions, and fields, Mesh module is used to create finite element mesh, Job module is used to submit a job for analysis and monitor its progress, the last module is Visualization used to view analysis results. This modules is presented below in detail how it's used for this analysis.

4.1. Geometric Properties

The geometrical properties of exterior beam column joint as shown on the Figure 3.2 were kept constant for all analysis for the study of the joint behavior under cyclic loading. Point of contraflexure is assumed at the mid height of the column and the beam length are selected by considering span to depth ratio of the beam. The column has a cross section of 700 mm x 700mm with an overall length of 3000mm and the beam has a cross section of 400mm x 500mm and the length of the cantilevered portion is 1120mm.The beam is reinforced using 4Ø24 as top and bottom longitudinal bars and ø 10 c/c 100mm is used as transverse reinforcement. The column was reinforced with 12 ø 24 longitudinal bars and ø 10 ties with spaced 100mm as transverse reinforcement.

4.2. Materials Properties

4.2.1. Concrete

Concrete damage plasticity model (CDPM) is used in order to define the nonlinear behavior of concrete. The CDPM parameters as shown in the Table 4.1 are taken by default from Abaqus User's Manual [6].

Table 4-1: CDPM parameters

Dilation angle(Ψ)	Eccentricity($\mathcal{E}$)	Stress ratio($\frac{\sigma_{bo}}{\sigma_{co}}$)	Shape factor(K_C)	Viscosity parameter(μ)
35	0.1	1.16	0.6667	0

The compressive and tensile stiffness recovery are defined by factors w_t and w_c through the load reversal. The set values for these factors were as default i.e. w_t =0 and w_c =1 and the degradation of the elastic stiffness is characterized by two damage variables, dt and dc also calculated using the Eqs. 2.6 and 2.7.

The other variables which is relevant to capture the nonlinear behavior of concrete are concrete compressive strength and concrete tensile strength with their corresponding compressive plastic strain and tensile plastic strain are shown in Table 4.2 and table 4.3 respectively.

The Hognestad [24] model was selected for both the pre and post-peak compression response of the concrete. This model is a simple compression response curve and it's suitable for normal concrete strength (<40MPa).

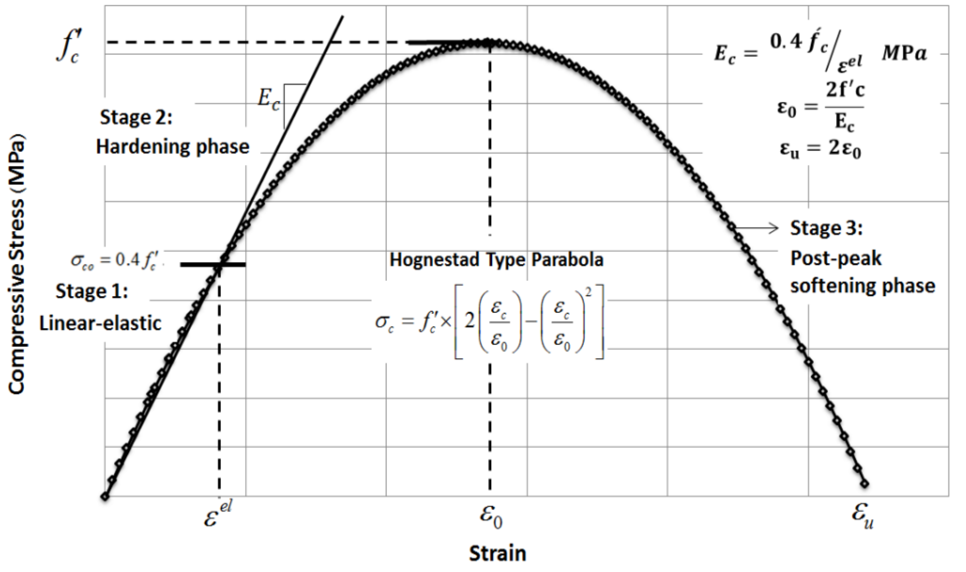


Figure 4-1: Hognestad parabolic pre- and post-peak concrete compression response [24]

Abaqus input data for concrete compression hardening for f'_c =25MPa. The inelastic strain is defined as the total strain minus the elastic strain. Where ε_0 is the strain at the peak stress f'_c, and is usually taken as 0.002.

Table 4-2: Concrete compressive strength

Yield stress (MPa)	Inelastic strain (mm/mm)	Concrete compression damage (d_c)
9.002903226	0	0
16.01032258	0.000322425	0
21.02032258	0.000644745	0
24.03096774	0.000966962	0
25.04032258	0.001289075	0
24.04645161	0.001611085	0.038142
21.04741935	0.00193299	0.158103
16.04129032	0.002254792	0.358348
9.026129032	0.002576491	0.638955

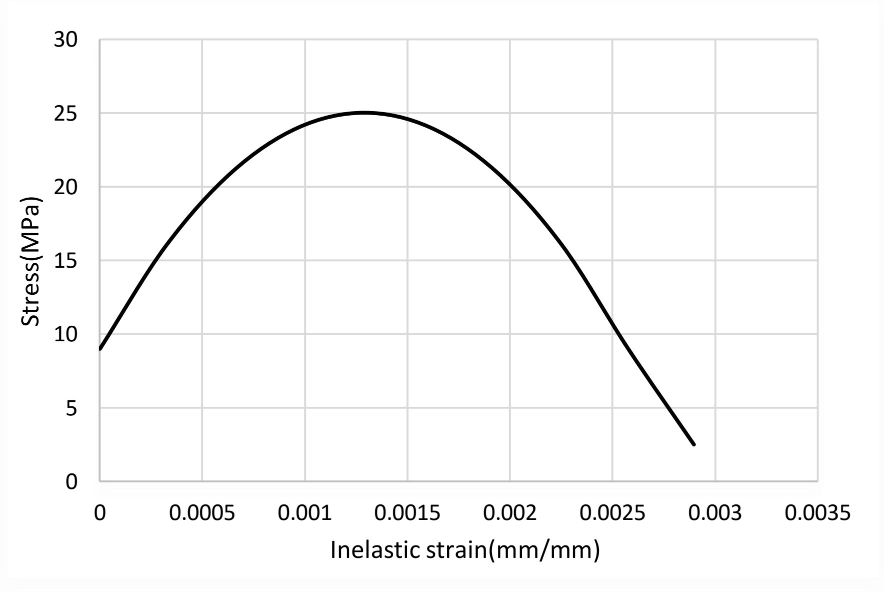


Figure 4-2: Compressive strength of concrete

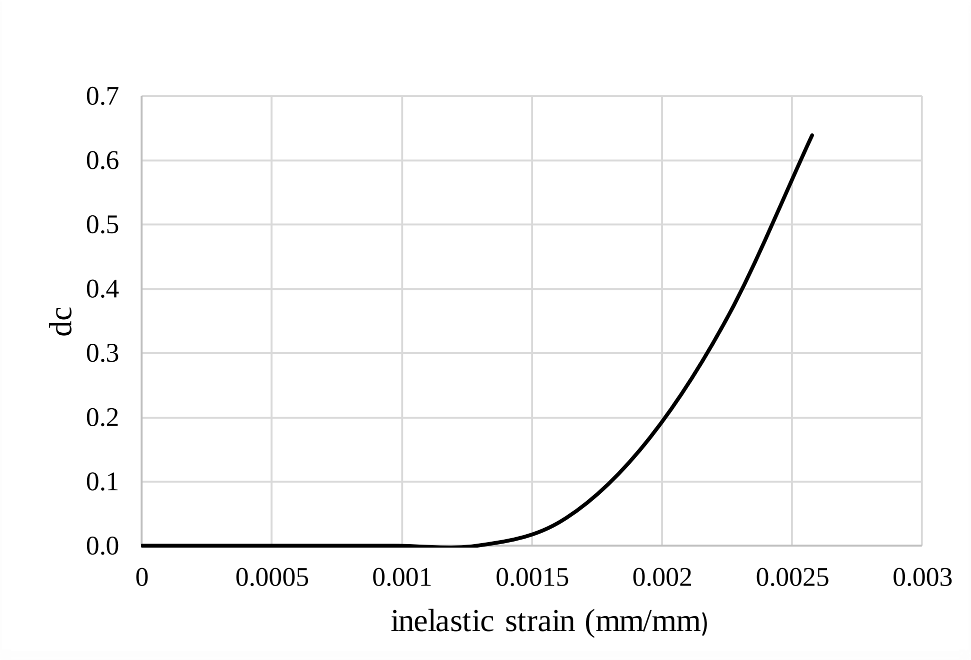


Figure 4-3 : concrete compression damage

The tensile behavior of the concrete is defined by using the formula recommended by Belarbi and Hsu (1994) and Pang and Hsu (1995) [25].

For Ascending branch ($\varepsilon_1 \leq \varepsilon_{cr}$)

$$\sigma_1^c = E_C * \varepsilon_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{4-1}$$

where:

E_C = modulus of elasticity of concrete, taken as 3875 $\sqrt{f'c}$, where f'_c and $\sqrt{f'c}$ are in MPa;

ε_{cr} = cracking strain of concrete, taken as 0.00008 mm/mm,

For descending branch ($\varepsilon_1 > \varepsilon_{cr}$)

$$\sigma_1^c = f_{cr}\left(\frac{\varepsilon_{cr}}{\varepsilon_1}\right)^{0.4} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{4-2}$$

f_{cr} is cracking stress of concrete, taken as 0.31 $\sqrt{f'c}$ MPa

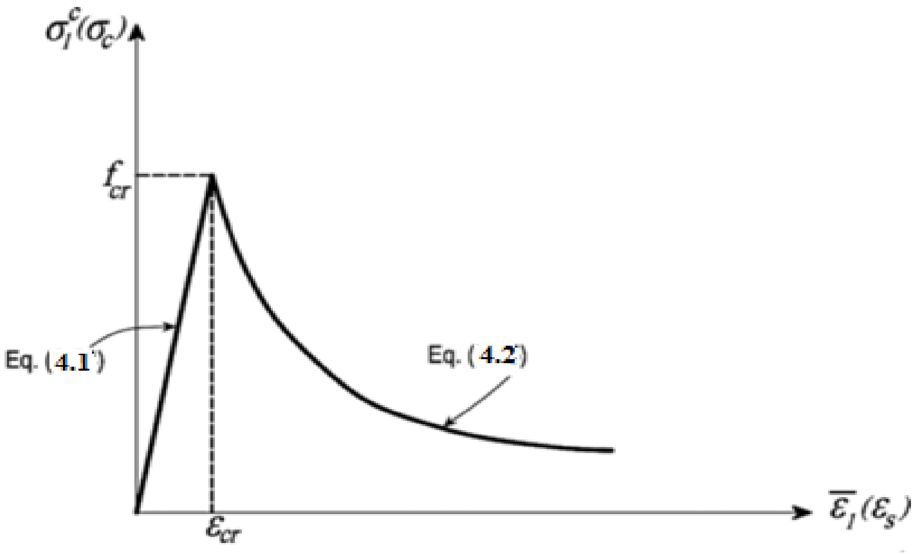


Figure 4-4 : Tensile stress-strain curve of concrete

Abaqus input data for concrete tension stiffening for f'_c=25 MPa. The cracking strain is defined as the total strain minus the elastic strain.

Table 4-3: Concrete tension stiffening

Yield stress(MPa)	Inelastic strain (mm/mm)	Concrete tension damage (d_t)
0.77503	0	0
1.55012	4E-05	0
1.17487	0.00012	0.24202
0.99905	0.0002	0.355451
0.89053	0.0002799	0.425467
0.81455	0.0003599	0.474484
0.75732	0.0004399	0.511406
0.71209	0.0005198	0.540586
0.67511	0.0005998	0.564446
0.64409	0.0006797	0.584457
0.61756	0.0007597	0.601574

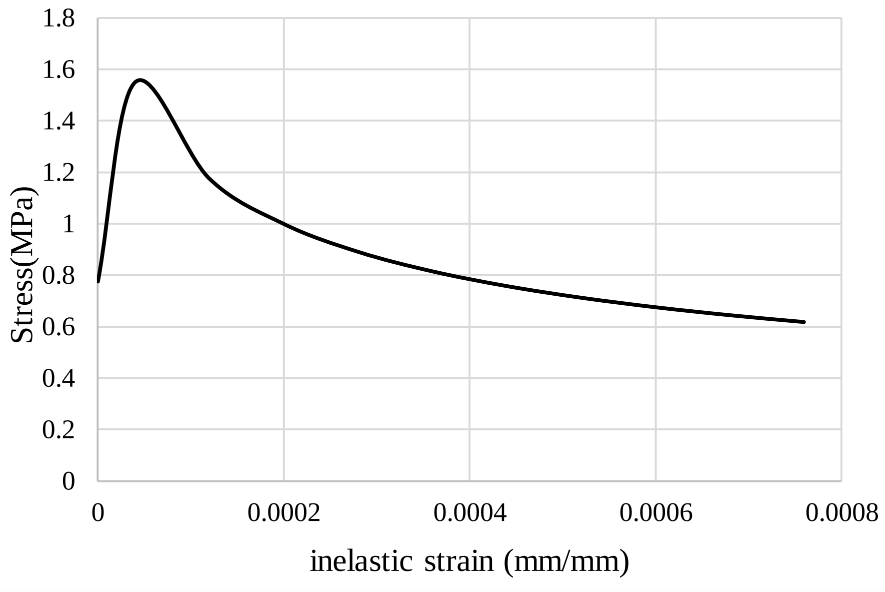


Figure 4-5: Concrete tension stiffening

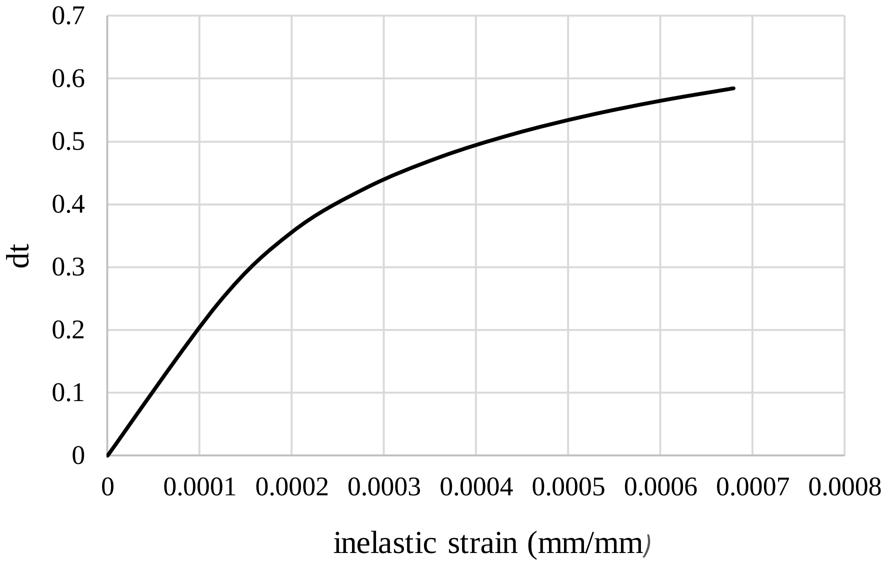


Figure 4-6 : concrete compression damage

4.2.2. Reinforcement

Since steel is homogenous material than concrete it is possible to use typical stress strain curve to define steel plasticity therefore the yield and ultimate stress of reinforcement bar 400MPa and 600MPa respectively are used for this study. The Longitudinal and transverse steel reinforcement behavior was defined as an elastic-plastic material using a bilinear curve as shown in the Figure 4.3.

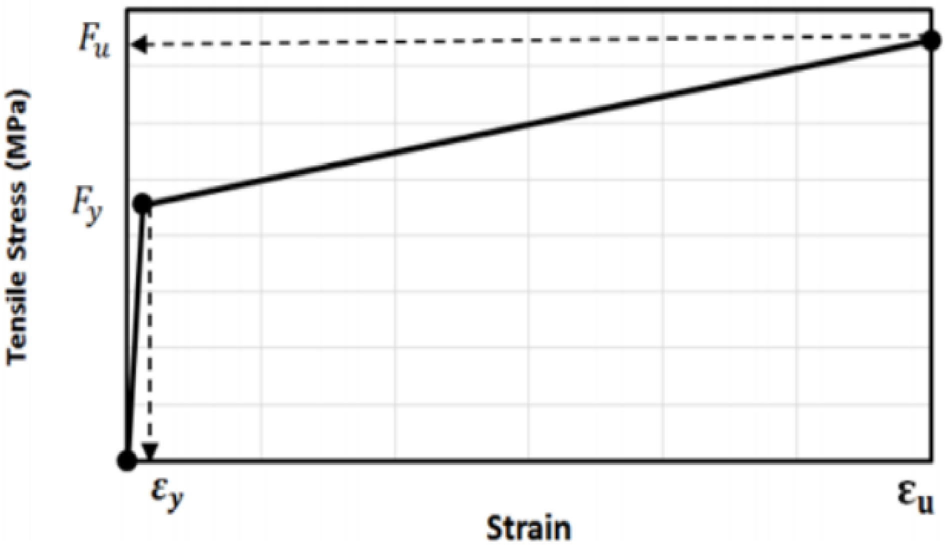


Figure 4-7 : Typical uniaxial stress-strain behavior of reinforcements introduced to the numerical model *[23]*

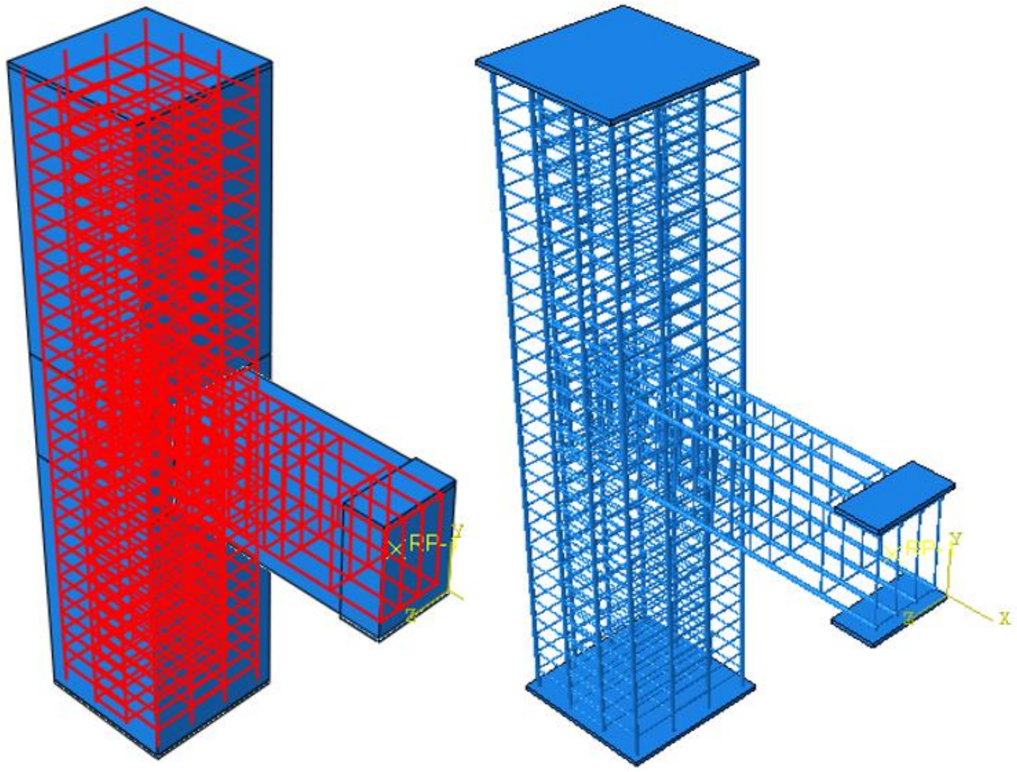


Figure 4-8: Finite element model of concrete and reinforcement detail

4.3. Loading and Boundary Condition

Non-linear analysis of exterior beam column joint was carried out with two loading steps. The column axial load is applied to the top of column corresponding to $0.1f_c'$*Ag, $0.2f_c'$*Ag, $0.3f_c'$*Ag, $0.4f_c'$*Ag and $0.5f_c'$*Ag, respectively, in the first step for six different analysis and also one additional analysis is performed without axial load as a controlling model. The magnitude of axial load was calculated by the formula 4-3 [5].

$$\nu = \frac{N}{A_g * f_c'} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots . 4\text{-}3$$

where:

A_g is the gross cross-sectional area of the column,

f_c' is the compressive strength and

N is the column axial load

The lateral load was carried out using displacement control in the second step at the tip of the beam through a loading plate in a quasi-static cyclic manner. The capacity of the joint is evaluated under service load. Constant displacement were performed at each number of cycles which contained positive and negative segments of 5mm. The displacement history is constant for all specimens as shown in the Figure 4.5. A steel loading plate are tied up with the concrete beam and column to avoid the stress concentrations around the points of loading.

Displacement/rotation boundary condition is used to constrain the movement of the selected degrees of freedom to zero or to prescribe the displacement or rotation for each selected degree of freedom. In the present study the bottom of the column is restrained in three degrees of freedom at the U_x, U_y and U_z directions and the top of the column is restrained in two degrees of freedom at the U_x and U_z directions as shown the Figure 4.5.

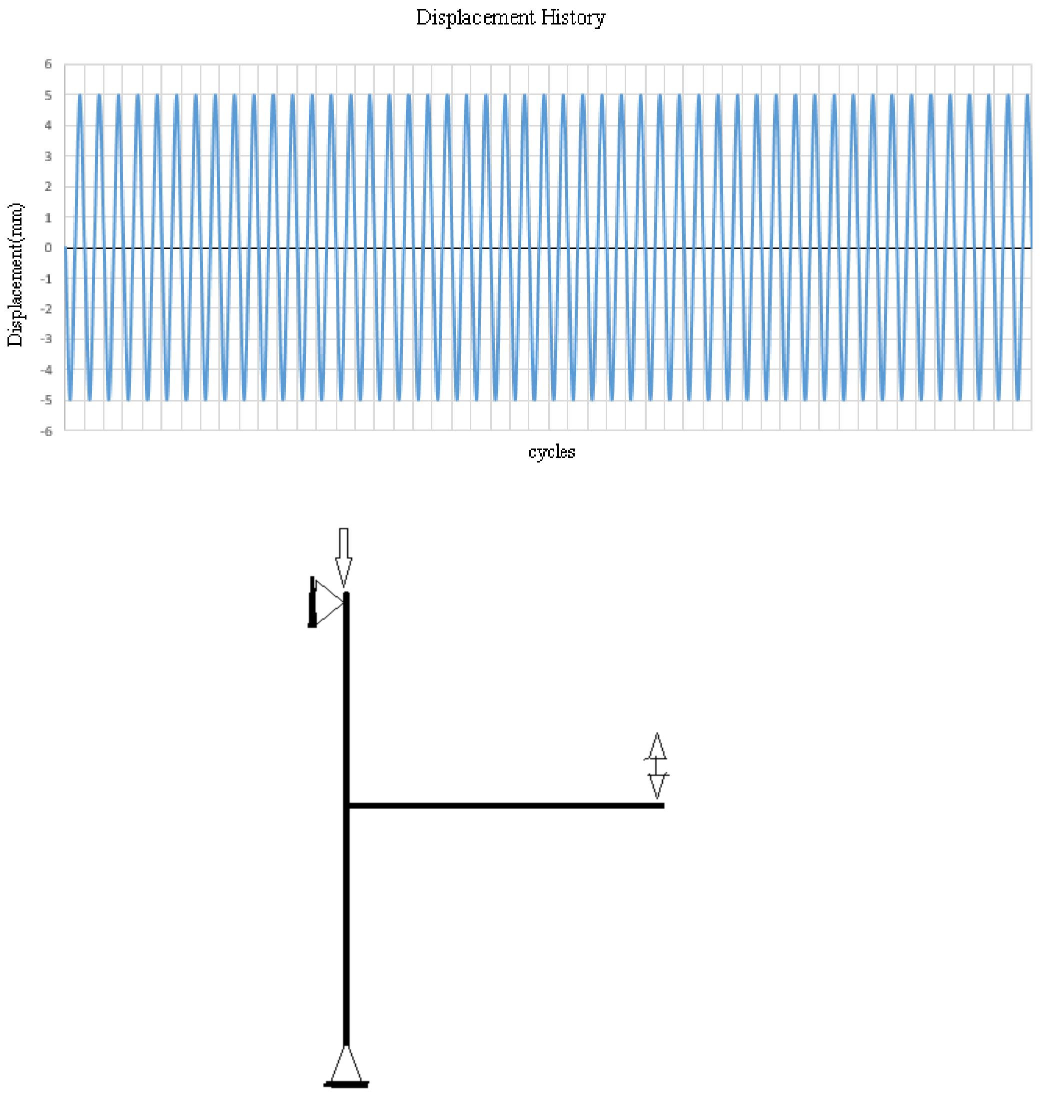


Figure 4-9: Loading and boundary condition

4.4. Mesh and Finite Elements

The element type is selected and assigned to the mesh by choosing the element family, geometric order and shape along with specific element controls. In this study 8-node brick element (C3D8) and 2-node truss element (T3D2) is used for modelling concrete and reinforcement, respectively.

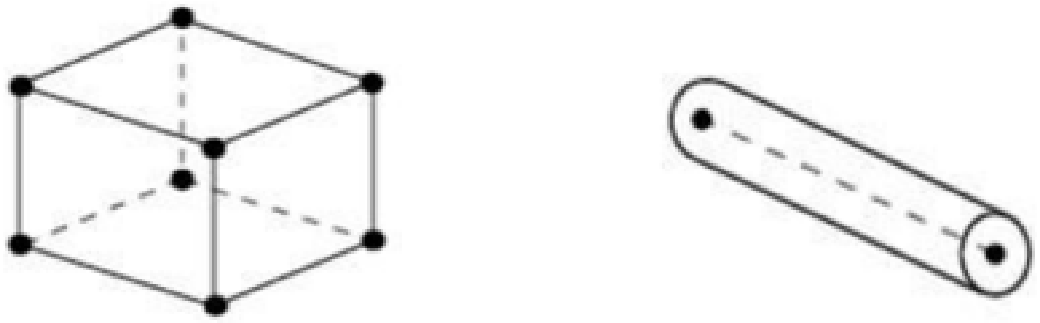


Figure 4-10 : Linear Element (8-Node Brick Element) and Truss Element (T3D2)

The mesh size of 60 mm are taken for the beam and column. Tie constraint is used to tie the interface between beam and column. A tie constraint ties two separate surfaces together to make the translational and rotational motion as well as all other active degrees of freedom equal for a pair of surfaces. The embedded element technique [6] is used to specify that an element or groups of elements are embedded in host elements. Among different type of Abaqus product Abaqus/Explicit is used for this research because of the advantages that are discussed in the literature.

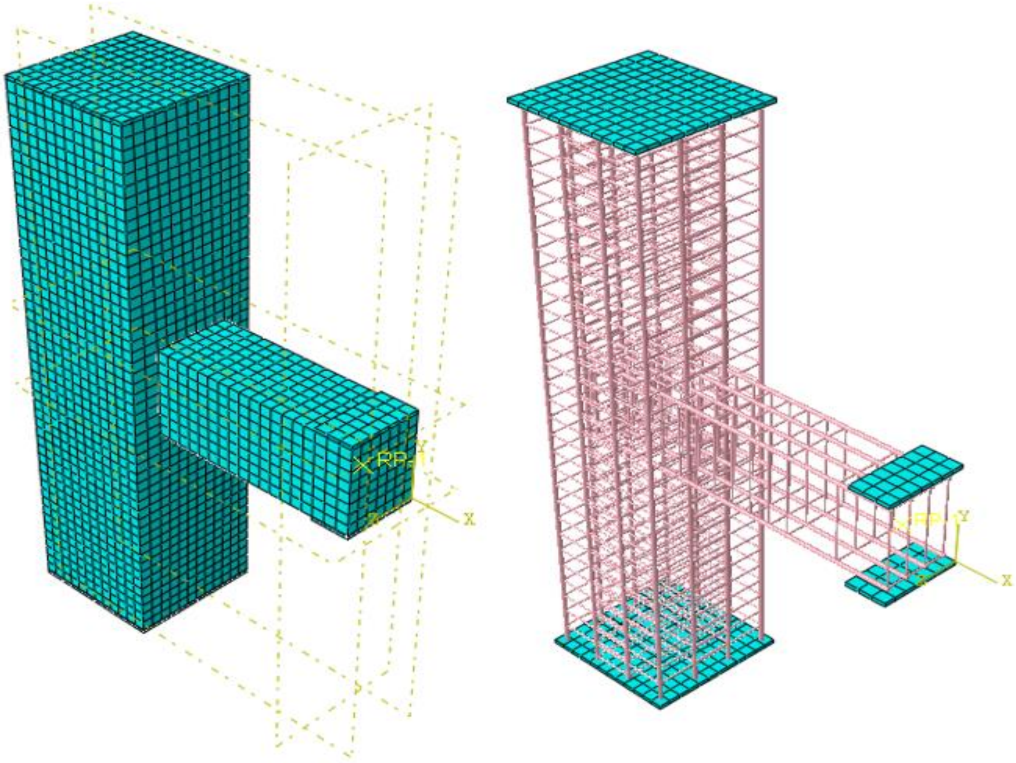


Figure 4-11: Concrete and reinforcement element mesh

5. ANALYSIS RESULT AND DISCUSSION

The shear strength and behavior of beam-column joint is presented in this study in terms of:

- Load-displacement graph and
- Joint shear stress vs. displacement

The joint shear strength examined under five different column axial load levels and the analysis results of each beam-column joint specimen are discussed as followed:

5.1. Shear Strength of Beam-Column Joint under Cyclic Loading

This model was selected as the control beam-column joint without compression column axial load to compare the response of other model with different level of axial load and to establish the effect of column axial load on the shear strength of exterior beam column joint.

Figure 5.1 shows the relationship between the applied displacement and the corresponding force at the same point and the result shows that as the number of cycle increase the load carrying capacity of the beam column joint decreases because the reversed cyclic loading form a cross inclined shear cracks at the joint region. The maximum load obtained from the analysis is 310kN at displacement of 5mm.

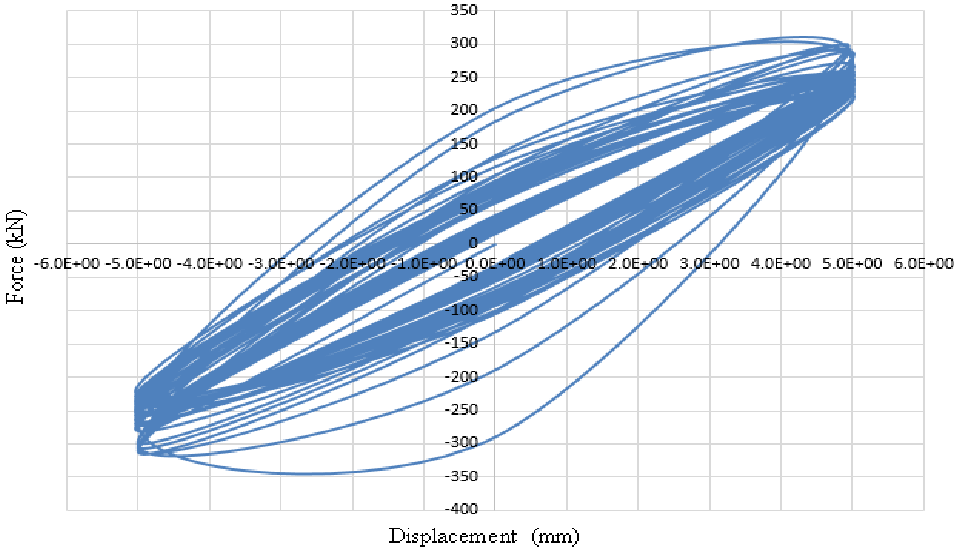


Figure 5-1: Lateral load-displacement diagram

At the end of cyclic loading the load carrying capacity of a joint decreases from 310kN to 222kN, the load had dropped 28% from the peak at the same displacement value. However the applied displacement is constant for each number of cycles the capacity of the beam-column joint is seen to deteriorate at each number of cycles. This deterioration may be due to the fact that the repeated reversal cyclic loading initiate full depth crack at the beam-joint interface. This has to do with the fact that cycling not only contributes to increase the damage already induced in the members on the previous cycle by the successive opening of the cracks, but also spreads further damage as new cracks form [9]. In order to visualize the cracking pattern of the model the maximum principal strain contour is displayed as shown in Figure 5.2. The crack initiates from the root of a beam and propagates to the joint and this may also the cause of stiffness degradation between the first and the subsequent cycles.

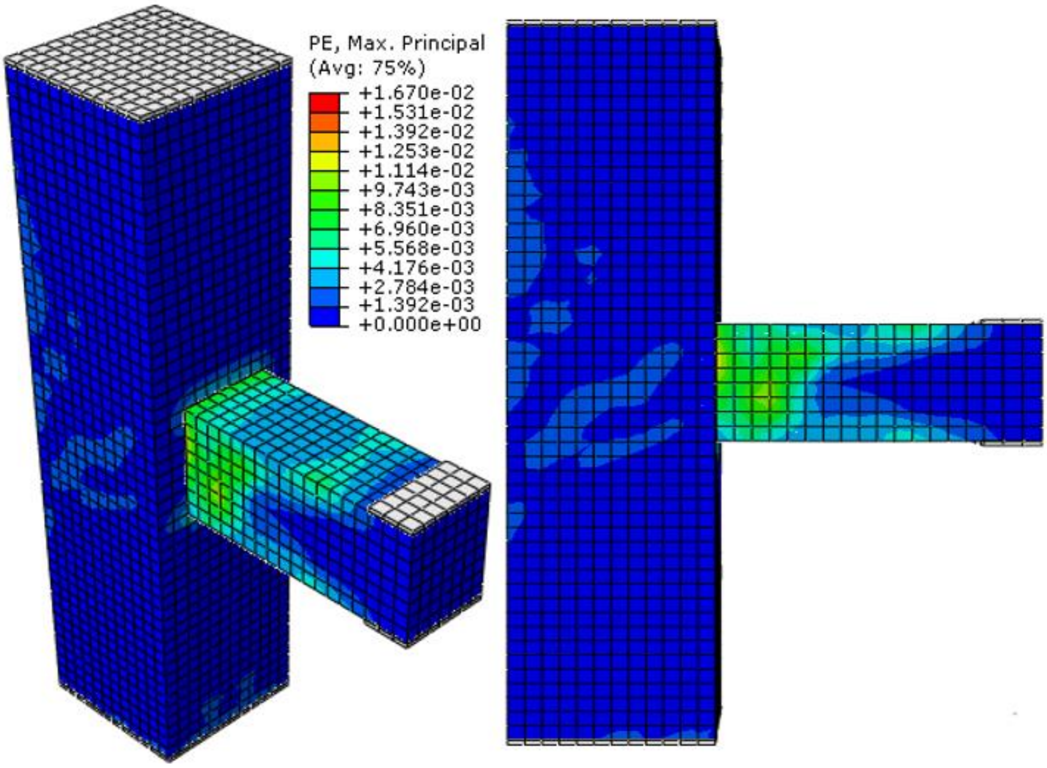


Figure 5-2: Developing of full depth cracks at beam-joint interface

The maximum joint shear stress obtained from the analysis without axial load is 2.15MPa at the first cycle and after 50cycles the capacity of the joint dropped to 1.24MPa which is 42% of the peak value as shown in the Figure 5.3.

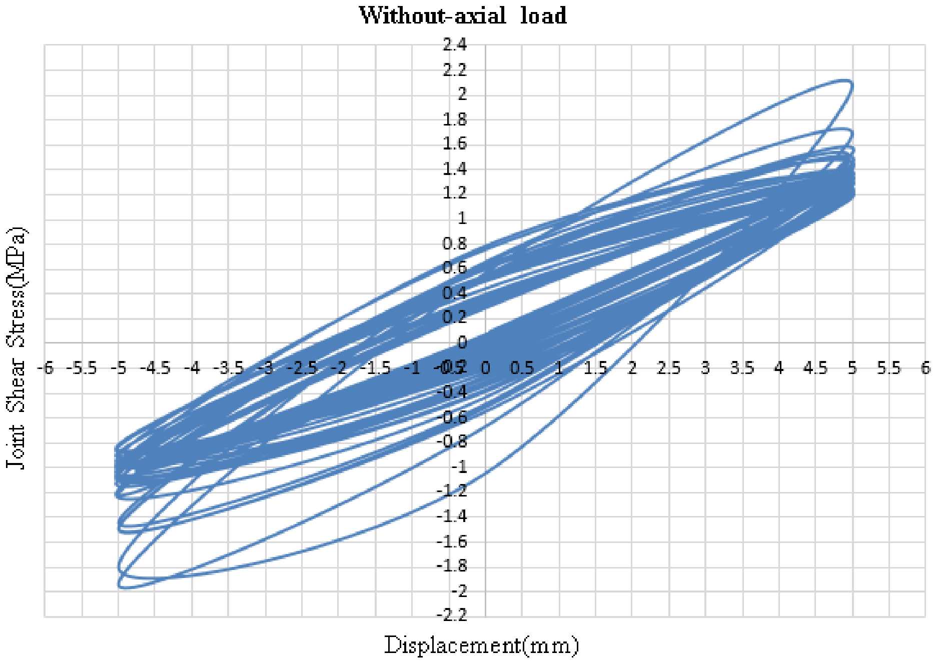


Figure 5-3: Joint shear stress verses applied beam displacement for joint without axial load

5.2. Shear Strength of Beam-Column Joint under Different level of Axial Load

Five different levels of column axial load from $0.1f'_c \times A_g$ to $0.5f'_c \times A_g$ were analyzed and the results obtained from the software for each specimen is discussed below separately and then comparison is made by taking the maximum value of joint shear stress.

5.2.1. Analysis result of 10% axial load

The maximum shear stress obtained with 10% of axial load is 2.48 MPa at the first cycle and after 50cycles the capacity of the joint dropped to 0.96MPa which is 61% of the peak value as shown in the Figure 5.4.

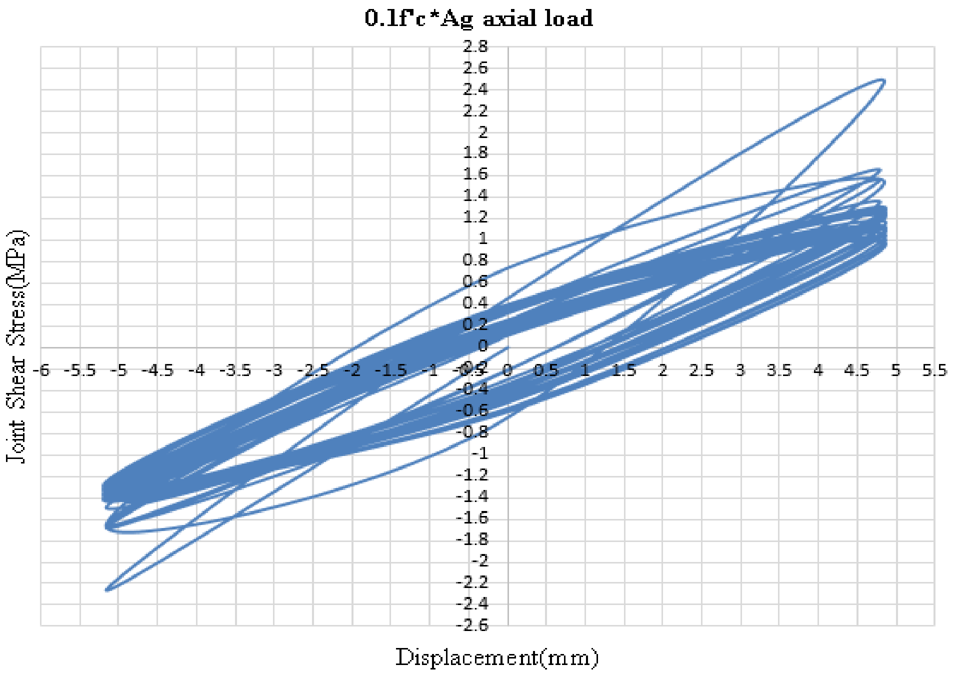


Figure 5-4: Joint shear stress verses applied beam displacement for 0.1f’c*Ag axial load

5.2.2. Analysis result of 20% axial load

The maximum shear stress obtained with 20% of axial load is 2.51 MPa at the first cycle and after 50cycles the capacity of the joint dropped to 0.95MPa which is 62% of the peak value as shown in the Figure 5.5.

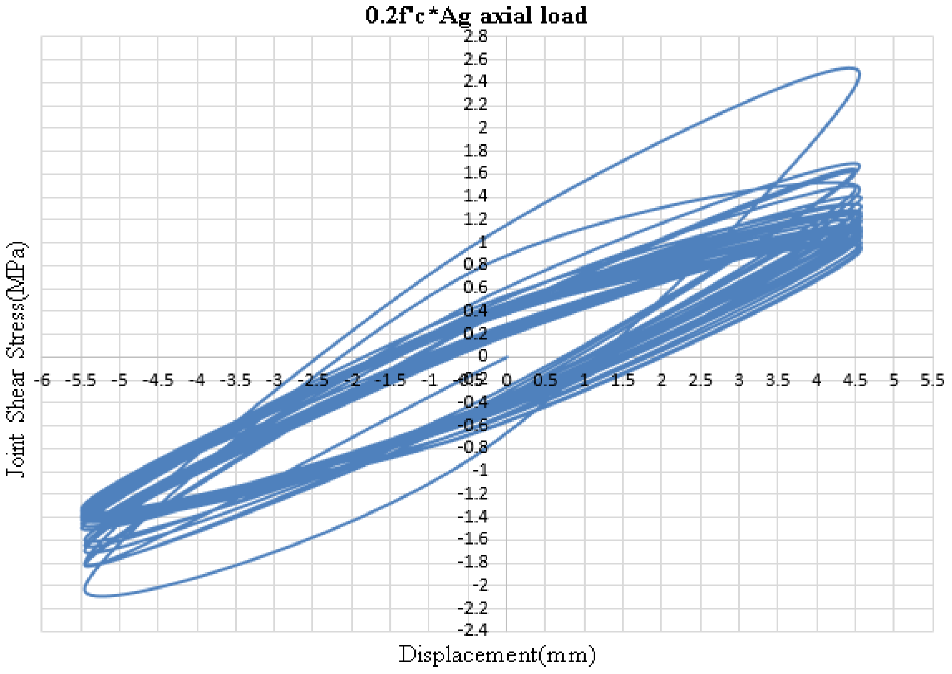


Figure 5-5: Joint shear stress verses applied beam displacement for 0.2f’c*Ag axial load

5.2.3. Analysis result of 30% axial load

The maximum shear stress obtained with 30% of axial load is 1.82 MPa at the first cycle and after 50cycles the capacity of the joint dropped to 0.95MPa which is 48% of the peak value as shown in the Figure 5.6.

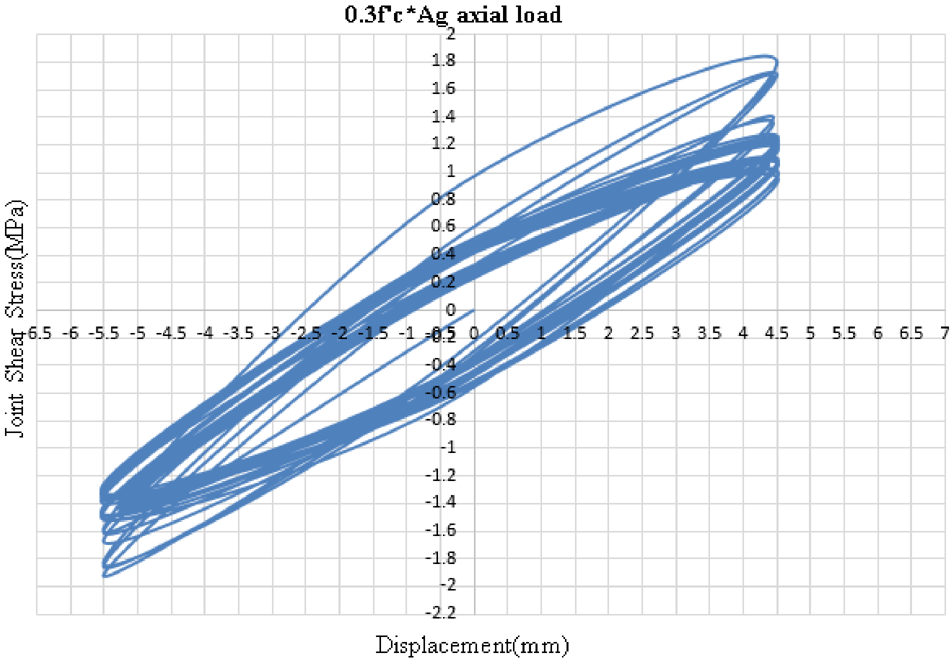


Figure 5-6: Joint shear stress verses applied beam displacement for 0.3f'c*Ag axial load

5.2.4. Analysis result of 40% axial load

The maximum shear stress obtained with 40% of axial load is 1.8 MPa at the first cycle and after 50cycles the capacity of the joint dropped to 0.954MPa which is 47% of the peak value as shown in the Figure 5.7.

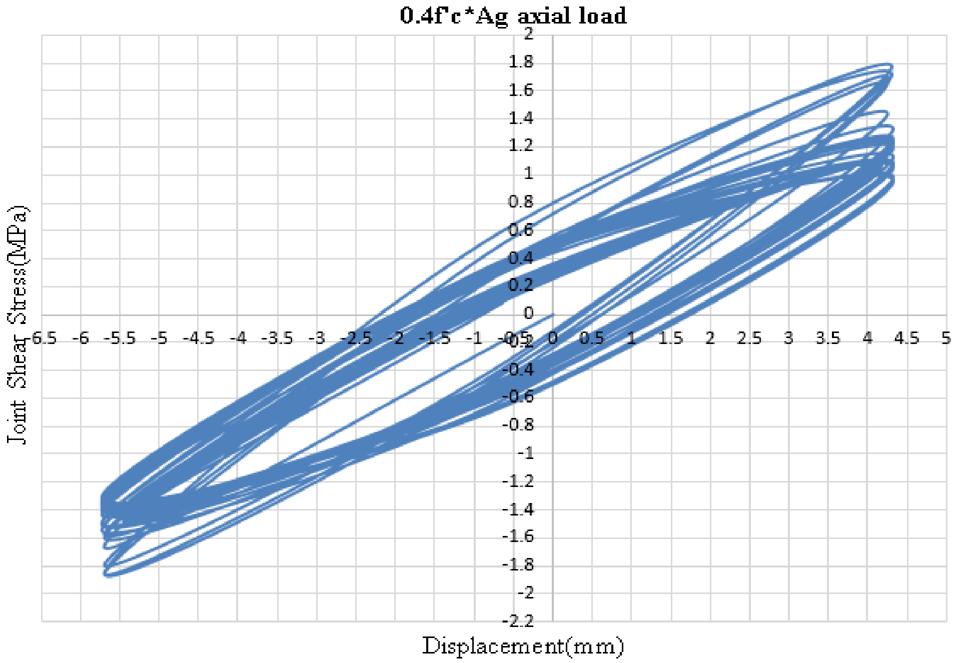


Figure 5-7: Joint shear stress verses applied beam displacement for 0.4f'c*Ag axial load

5.2.5. Analysis result of 50% axial load

The maximum shear stress obtained with 50% of axial load is 1.69MPa at the first cycle and after 50cycles the capacity of the joint dropped to 0.93MPa which is 45% of the peak value as shown in the Figure 5.8.

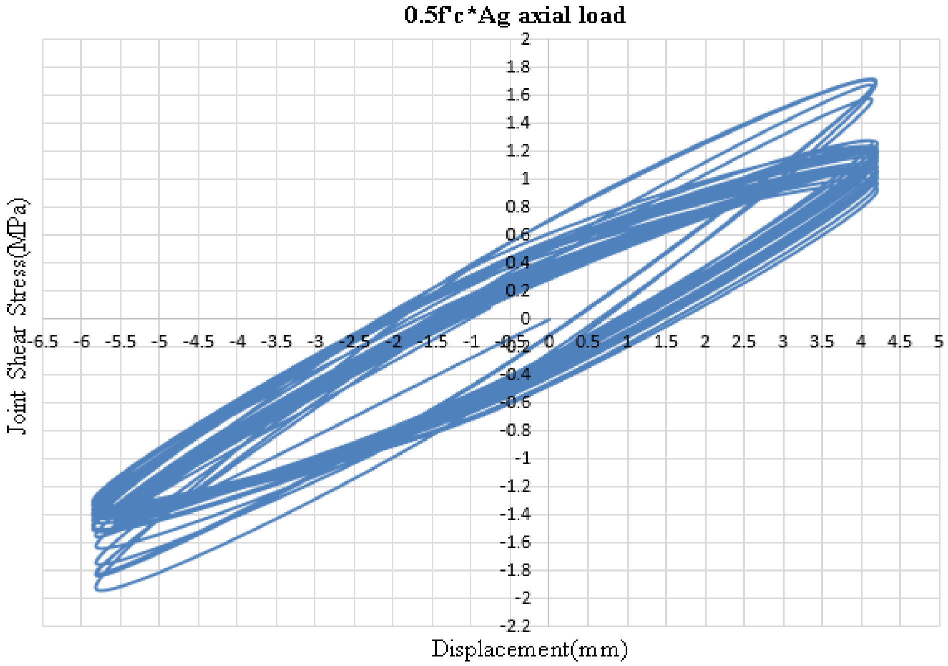


Figure 5-8: Joint shear stress verses applied beam displacement for 0.5f'c*Ag axial load

Figure 5.9 shows the envelopes of joint shear stress versus axial load ratio and this analysis result leads to an important conclusion about the effect of column axial load on the shear strength of the joint. As shown in the graph the column axial load corresponding to $0.1f_c' \times A_g$ and $0.2f_c' \times A_g$ axial loads shows positive effect on the joint shear strength but axial load greater than $0.2f_c' \times A_g$ shows detrimental effect on the joint shear strength.

The maximum shear stress obtained from the specimen with $0.1f_c' \times A_g$ and $0.2f_c' \times A_g$ axial loads greater than the maximum shear stress obtained from zero axial load. Which is an increases of 13.3% and 14.3% in the shear capacity of a joint respectively. And it is a result of increasing the confinement due to the axial compressive load. On the other hand the other subsequent specimen with axial load $0.3f_c' \times A_g$, $0.4f_c' \times A_g$ and $0.5f_c' \times A_g$ showed a reduction in joint shear capacity by 15.35%, 16.28%, 21.4% respectively when compared to the specimen with zero axial load. This degradation of capacity may be due to the fact that the concrete within the joint region is crushed.

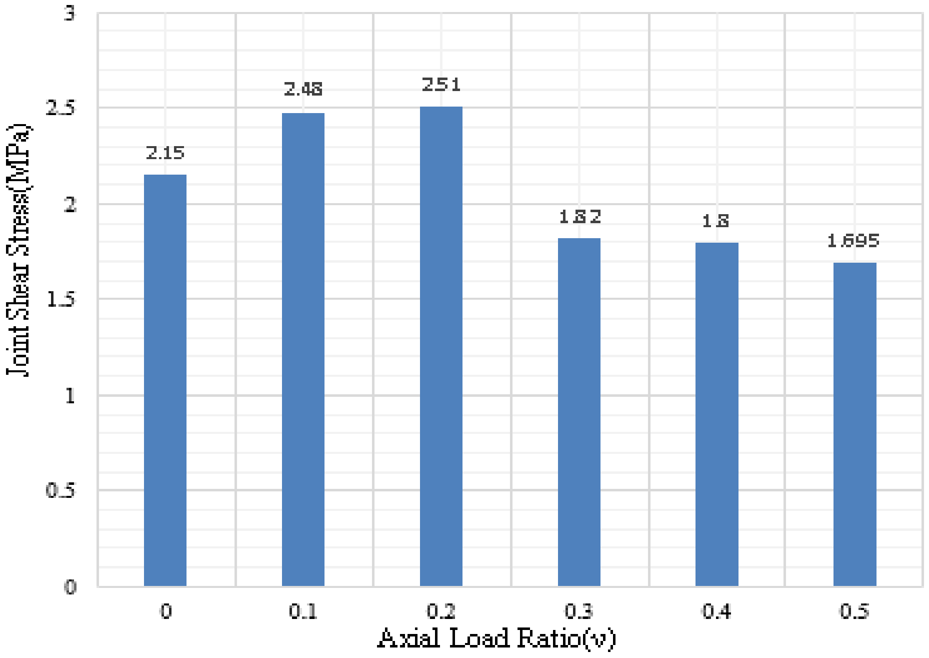


Figure 5-9 : Envelopes of joint shear stress versus axial load ratio

Generally, from this analysis result, it is reasonable to conclude that the column axial load magnitude changes the shear behavior of the joint. As the current study showed that the normal stress less than 20% of concrete compressive strength enhance the shear capacity of the joint this is due to the fact that column axial load was provided confinement to the concrete which placed around the joint region and it may be significantly increase the bond between the concrete and reinforcement.

This was briefly discussed by Costa J. L. D [9] the presence of low-to medium compressive forces increases the ultimate strength of a low shear span ratio member. This is due to the increase in the depth of the compression zone and so, in this zone, the shear may be transferred by inclined compression. The contribution of the diagonal compressed strut is significant during the first cycle in the inelastic range. However it deteriorates with the increase of the inelastic loading cycles. This is due to the fact that cycling at high levels of inelastic deformation causes permanent elongation on the beam bars and leads to full depth open cracks at the beam-joint interface. Under these conditions flexural compression from the beams become negligible. The compressive forces are then transmitted to the longitudinal bars of the beams, which significantly increase the bond stresses along the horizontal perimeters of the joint core. This leads to a drastic reduction in the contribution of the concrete strut to the transfer of horizontal joint shear and a consequent increase in the contribution of the truss mechanism. The mobilization of the truss mechanism depends intimately on the effectiveness of bond between the steel bars and the surrounding concrete. Bond has a very poor response in terms of energy dissipation, stiffness and strength degradation under inelastic cycling. Thus, it can be concluded that the development of plastic hinges in the end sections of the beams seriously affects the ability of the joint to resist in a stable manner the induced shear forces. Again, joints whose columns are low axially loaded are the most sensitive to bond deterioration since compression helps to maintain the bond mechanism.

The foregoing serves to emphasize the need to take special precautions to prevent premature bond deterioration in joints under seismic loads. Adequate confinement of the joint core significantly improves the bond performance under seismic conditions [8]. Confinement may be provided by axial compression of the column and/or by means of reinforcement using the intermediate column bars as these members are supposed to remain in the elastic domain. Moreover, confinement improves the performance of the compressed diagonal strut. [9].

On the hand when the column normal stress greater than 20% of concrete compressive strength showed detrimental effect on a joint shear strength and this is due to the fact that the concrete is crushed in the joint region. This result may be reasonable according to Hegger et al. [26] and other researcher which is discussed in the literature review. Hegger et al. [26] reported that in columns, normal stresses higher than 40% of the concrete compressive strength reduce the shear strength of a joint.

6. CONCLUSION AND RECOMMENDATION

6.1. Conclusion

The objectives of this study are to evaluate the behavior and to investigate the effect of column axial load on the shear strength of beam column joint under cyclic loading. To address this objectives column size of 700mm*700mm reinforced with 12ø24 bar and beam size of 400mm*500mm which was symmetrically reinforced with 4ø24 bar was used and also the horizontal confinement reinforcement in joint was provided according to ES EN 1998-1:2015. Then by using computational software Abaqus, the joint is examined under column axial load from zero to $0.5f_c' \times A_g$ when the end of the beam was subjected to displacement control cyclic loading. Based on the result obtained from numerical analysis, the following conclusion can be drawn.

1. Whenever the applied displacement was kept constant for each number of cycles, the capacity of the beam-column joint is seen to deteriorate at each number of cycles. This deterioration is attributed to the fact that the repeated reversal cyclic loading initiate full depth crack at the beam-joint interface.
2. The analysis result showed that the column axial load corresponding to $0.1f_c' \times A_g$ and $0.2f_c' \times A_g$ axial load showed positive effect on the joint shear strength whereas, axial load greater than $0.2f_c' \times A_g$ showed detrimental effect on the shear strength of the joint. Generally column axial load have positive and negative effect on the shear strength of the joint depending on its magnitude.

6.2. Recommendation

1. Since joint shear strength is affected by different parameters; investigating the effect of each parameters individually on the shear strength of joint will be vital to give a reliable conclusion.
2. Since this study covers the effect of axial load on the shear strength of exterior beam column joint it’s also important to examine its effect on other types of joint, like interior and corner.
3. Further research regarding to shear strength of beam-column joint needs to be conducted experimentally to identify the influence of compression column axial load on the behavior of the joint.

REFERENCES

[1] Zhang, L. and Jirsa, J.O., "A Study of Shear Behavior of Reinforced Concrete Beam-Column Joints," University of Texas,Austin, 1982.

[2] Park,S. and Mosalam,K.M., "Shear Strength Models of Exterior Beam-Column Joints without Transverse Reinforcement," Pacific Earthquake Engineering Research Center,Berkeley;California, 2009.

[3] Haach,V.G., El Debs, L .H.C. and El Debs ,M.K., "Evaluation of the Influence of the Column Axial Load on the Behavior of Monotonically Loaded R/C Exterior Beam–Column Joints through Numerical Simulations," Elsevier, no. 30, p. 965–975, 2008.

[4] Clyde,C., Pantelides,C.P. and Reaveley,L. D., "Performance-Based Evaluation of Exterior Reinforced Concrete Building Joints for Seismic Excitation," Pacific Earthquake Engineering Research Center,Berkeley;California, 2000.

[5] "ES EN 1998-1:Design of Structures for Earthquake Resistance," Ministry of Construction,Addis Ababa,Ethiopia, 2015.

[6] "ABAQUS:Analysis user's manual 6.14-EF," Dassault Systems Simulia Corp.Providence,RI,USA, 2014.

[7] Subramanian, N. and Rao, D.S.P., "Seismic Design of Joints in RC Structures—A Review," Indian Concrete Journal, vol. 77, no. 2, p. 883–892, 2003.

[8] Paulay,T. and Priestley, M.J.N, "Seismic Design of Reinforced Concrete and Masonry Buildings," John Wiley & sons,Inc,USA, 1992.

[9] Costa,J.L.D, "Reinforced Concrete under Large Seismic Action," University of Denmark, 2003.

[10] Uma,S.R. and Prasad,A. M., "Seismic Behavior of Beam Column Joints in Reinforced Concrete Moment Resisting Frames - A Review," Indian Concrete Journal, vol. 1, no. 80, pp. 33-42, 2006.

[11] Park, R. and Paulay, T., "Reinforced Concrete Structures," John Wiley & Sons Inc,New York, p. 769, 1975.

[12] Meinheit, F.D and Jirsa, O.J., "Shear Strength of RC Beam-Column Connections," Journal of the Structural Division, ASCE, vol. 107, no. 11, 1981.

[13] Murthy, D.S.R., Gandhi, P., Sreedhar, D.S., Vaidhyanathan, C.V. and Mohanty, O.N., "Seismic Resistance of Reinforced Concrete Beam-Column Joints with TMT and CRS Bars," ICI Journal, vol. 1, no. 3, p. 19–26., 2000.

[14] Bindhu,K.R. ,Sukumar, P.M. and Jaya,K.P., "Performance of Exterior Beam-Column Joints under Seismic Type Loading," ISET Journal of Earthquake Technology, vol. 46, no. 2, p. 47–64, 2009.

[15] Russo,G. and Somma,G., "A Design Formula for Predicting the Shear Strength of Exterior Beam Column Joints under Seismic Loading," Vancouver, B.C., Canada, 2004.

[16] Patil,S. S., Konapure,C. G. and Manekari,S. S., "A Study of R. C. C. Beam Column Junction Subjected To Quasi-Static (Monotonic) loading," IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE), vol. 6, no. 5, pp. 61-74, 2013.

[17] Pauletta,M., Luca,D.D. and Russo,G., "Exterior Beam Column Joints – Shear Strength Model and Design Formula," Elsevier,Udine, Italy, 2015.

[18] Murty,C. V. R. ,Rupen G. ,Vijayanarayanan,A. R. and Mehta V.V. , "Earthquake Behaviour of Buildings," Gujarat State Disaster Management Authority(GSDMA),India, 2013.

[19] Osman, M. H., "Performance of Reinforced Concrete Beam-Column Joint Subjected to Repeated Reversed Load," Universiti tun hussein onn Malaysia,Malaysia, 2012.

[20] Hakuto,S., Park,R. and Tanaka,H., "Seismic Load Tests on Interior and Exterior Beam-Column Joints with Substandard Reienforcement Details," ACI Structural Journal, vol. 1, no. 97, pp. 11-25, 2000.

[21] Pampanin, S., Calvi, G.M. and Moratti, M., "Seismic Behaviour of RC Beam-Column Joints Designed for Gravity Loads," 12th European Conf. on Earthquake Engineering, p. 726, 2002.

[22] Uma,S.R. and Jain,S.K., "Seismic Design of Beam-Column Joints in RC Moment Resisting Frames – Review of Codes," Structural Engineering and Mechanics, vol. 23, no. 5, pp. 579-597, 2006.

[23] Najafgholipour,M.A., Dehghan,S.M.,Amin,D. and Arsalan,N, "Finite Element Analysis of Reinforced Concrete Beam-Column Connections with Governing Joint Shear Failure Mode," Latin American Journal of Solids and Structures, no. 14, pp. 1200-1225, 2017.

[24] Hognestad, E., "A Study of Combined Bending and Axial Load in Reinforced Concrete Members," University of Illinois, engineering experiment station, no. 399, 1951.

[25] Thomas, T. C. Hsu and Mo,Y. L. , "Unified Theory of Concrete Structures," John Wiley & Sons, Ltd,USA, 2010.

[26] Josef Hegger, Alaa Sherif and Wolfgang Roeser, "Nonseismic Design of Beam-Column Joints," ACI Structural Journal, vol. 5, no. 100, p. 654–664, 2003.

[27] "ES EN 1992-1:Design of Concrete Structures," Ministry of Construction,Addis Ababa,Ethiopia, 2015.

[28] Bond, A. J ., Brooker, O. ,Harris, A. J. ,Harrison, T. , Moss,R. M., Narayanan, R. S. and Webster,R., "How to Design Concrete Structures using Eurocode 2," A cementand concreteindustry publication, 2006.

[29] Elghazouli, A. Y., "Seismic Design of Buildings to Eurocode 8," Spon Press,New York, 2009.

APPENDICES

A. Data used for Analysis and Design of a Building

A.1. Data used to calculate dead load

Table A-1: Members cross section

COLUMNS		**BEAMS**	
Unit weight, ɣ $\left(\frac{\mathrm{kN}}{\mathrm{m}^3}\right)$	25	Unit weight, ɣ $\left(\frac{\mathrm{kN}}{\mathrm{m}^3}\right)$	25
Column height (m)	3	Beam length (m)	5
Column Depth(m)	0.7	Beam Depth(m)	0.5
Column Width(m)	0.7	Beam Width(m)	0.4
SLAB		**HCB**	
Unit weight, ɣ $\left(\frac{\mathrm{kN}}{\mathrm{m}^3}\right)$	25	Unit weight, ɣ $\left(\frac{\mathrm{kN}}{\mathrm{m}^3}\right)$	14
Slab thickness (m)	0.2	wall height (m)	3
Slab length(m)	5	Wall length(m)	5
Slab Width(m)	5	Wall thickness(m)	0.2
LIFT SHEAR WALL			
Unit weight, ɣ $\left(\frac{\mathrm{kN}}{\mathrm{m}^3}\right)$	25		
Shear wall height (m)	3		
Shear wall thickness (m)	0.2		

A.2. Input data for Earthquake analysis of building using ETABS

Building location = Addis Ababa

Seismic Zone =3

Ground acceleration=0.1g

Ground type= C

The assumed structure is regular in plan and elevation and also the fundamental period is less than 2second therefore according to ES EN 1998-1:2015, [5] static lateral force method is used for seismic analysis.

Fundamental period of the building

$T_1 = C_t * H^{\frac{3}{4}}$ =1.1sec Fundamental period of the building, in sec <2sec

Where

C_t is 0.075 for moment resistant space concrete frames

H is the height of the building, 36 m

Behavior factor

$q = q_o K_W \geq 1.5$

q_o is the basic value of the behavior factor, dependent on the type of the structural system and on its regularity in elevation. For Frame system q_o in DCM is $3.0\alpha u/\alpha_1$.For multistory, multi-bay frames $\alpha u/\alpha_1$ is 1.3

K_W is the factor reflecting the prevailing failure mode in structural systems with wall and is

1 for frame and frame-equivalent dual systems.

Analysis output

Figure A.1 and Figure A.2 shows shear force and moment diagram respectively that obtained from ETABS (2016) for the selected frame.

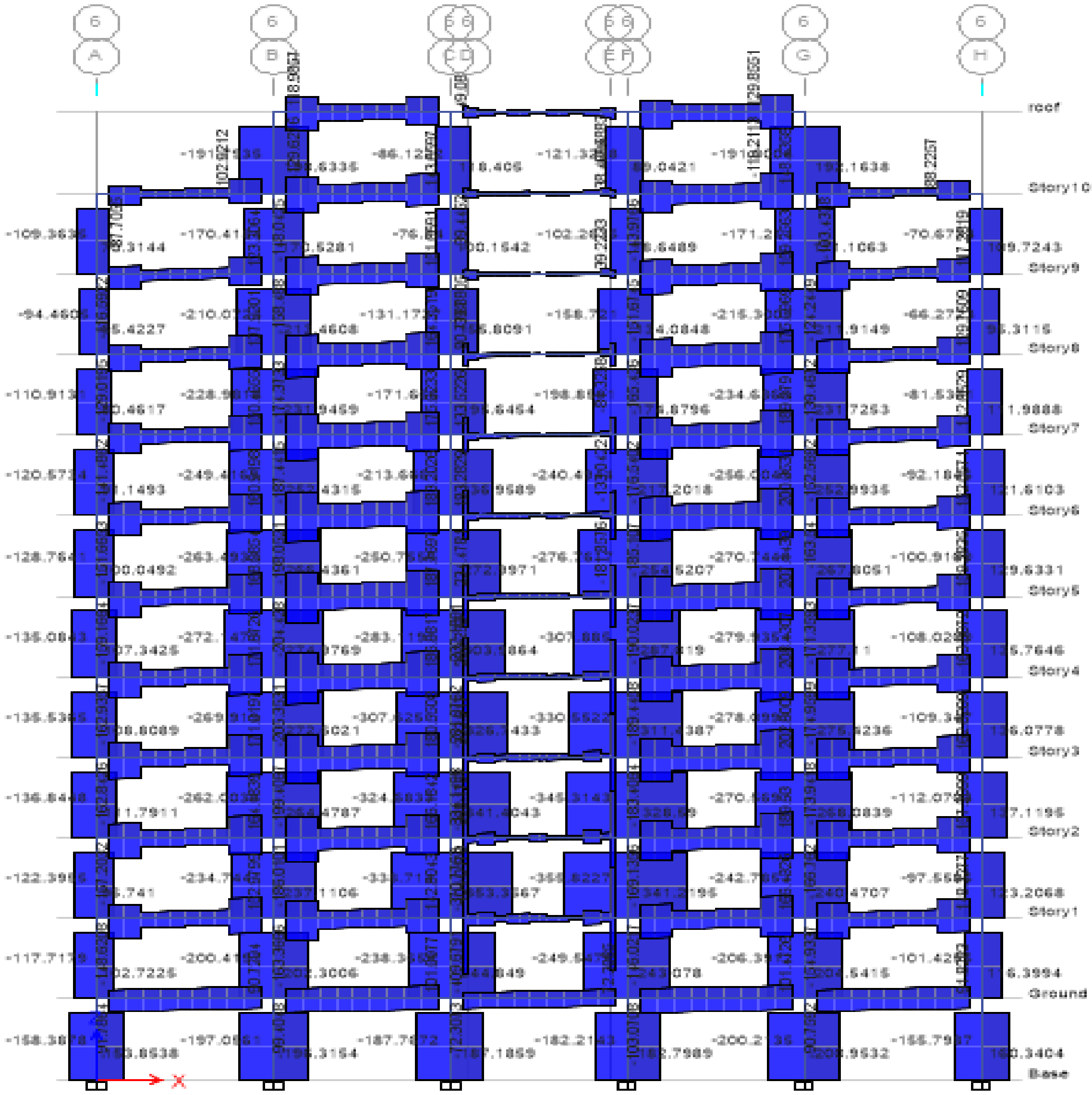


Figure A-1: Envelopes of shear force diagram for selected frame

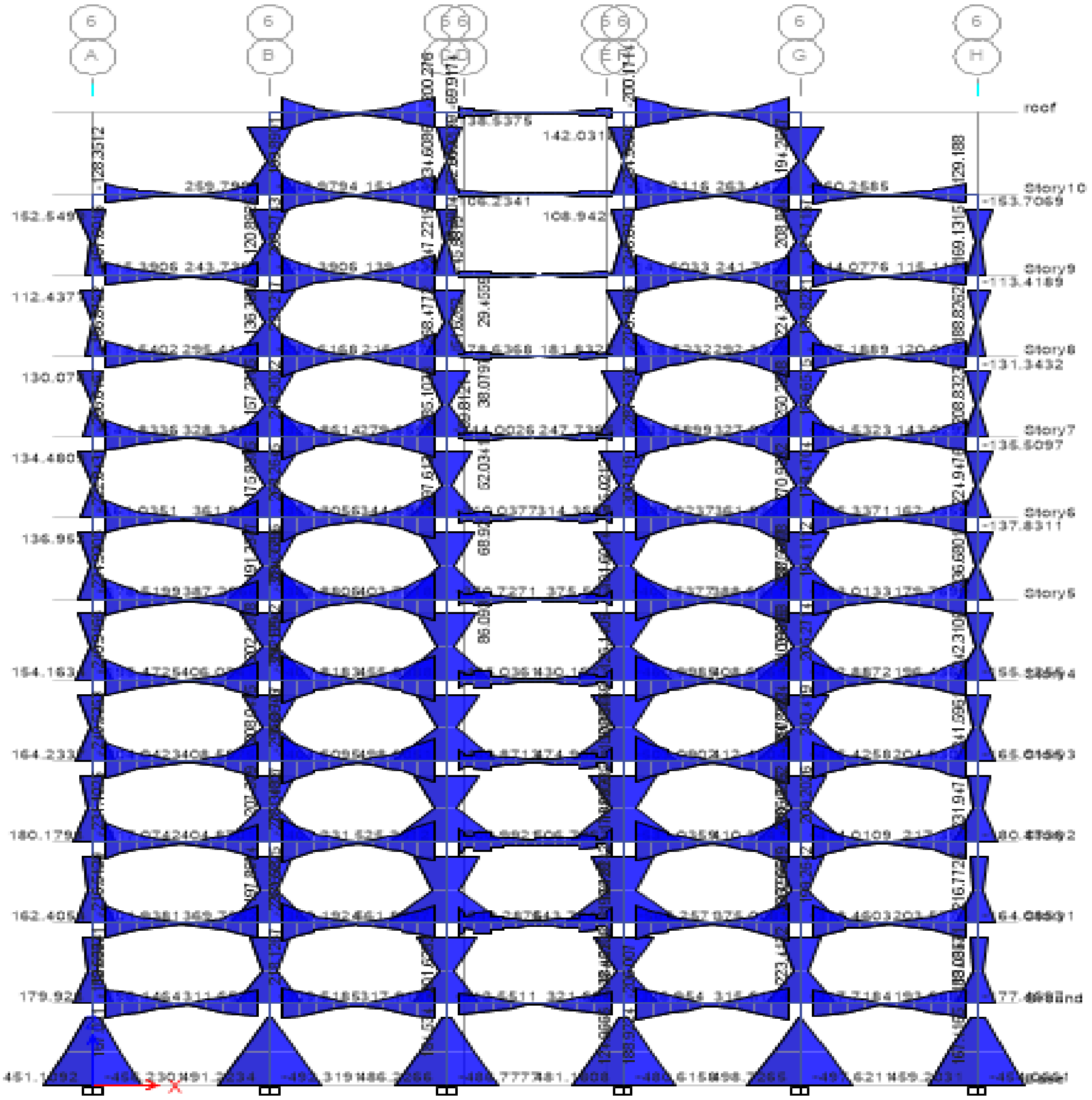


Figure A-2: Envelopes of bending moment diagram for selected frame

B. Beam-Column Joint Design

A Beam-Column joint was designed and detailed according to ES EN 1992-1-1:2015 [27] and ES EN 1998-1:2015 [5] and also Bond, A. J. et al. [28] and Elghazouli, A.Y. [29] are used as references.

The basic procedures and important principles are discussed below:

B.1. Design and Detailing of Beam

- ✓ **Carried out Analysis of Beam to Determine Design Moments (M)**

In order to promote an efficient transfer of moments between columns and beams, and reduce secondary effects, the offset of the beam center line from the column center line is limited to less than a quarter of the column width.

Also, to take advantage of the favorable effect of column compression on the bond of reinforcement passing through the beam-column joint: Width of beam was selected

$$\mathrm{b_w \le \min\{b_c + h_w; 2b_c\}} \qquad \text{B-1}$$

where

$\mathrm{h_w}$ is depth of the beam and

$\mathrm{b_c}$ is column width

- ✓ **Check deflection of the beam**

The span-to-depth ratio should ensure that deflection limited to $\frac{\mathrm{span}}{250}$

The limiting span-to-depth ratio may be estimated using Expressions B-2 and B-3 and multiplying this by correction factors to allow for the type of reinforcement used and other variables.

$$\mathrm{\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\,\frac{p_o}{p} + 3.2\sqrt{f_{ck}}\left(\frac{p_o}{p} - 1\right)^{3/2}\right] \quad if\ p \le p_o} \qquad \text{B-2}$$

$$\mathrm{\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\,\frac{p_o}{p - p'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{p'}{p_o}}\right] \quad if\ p > p_o} \qquad \text{B-3}$$

where:

$\frac{l}{d}$ is the limit span/depth

K is the factor to take into account the different structural systems

p_o is the reference reinforcement ratio = $10^{-3\sqrt{f_{ck}}}$

p is the required tension reinforcement ratio at mid-span to resist the moment due to the design loads (at support for cantilevers)

p' is the required compression reinforcement ratio at mid-span to resist the moment due to design loads (at support for cantilevers)

f_{ck} is in MPa units

Expressions (B-2) and (B-3) have been derived on the assumption that the steel stress, under the appropriate design load at SLS at a cracked section at the mid-span of a beam or slab or at the support of a cantilever, is 310 MPa,(corresponding roughly to f_{yk}= 500 MPa).

Where other stress levels are used, the values obtained using Expression B-2 and B-3 should be multiplied by $310/\sigma_s$

- ✓ **Determine**:

$$K = \frac{M}{bd^2 f_{ck}} \qquad \text{B-4}$$

$$K' = 0.6\delta - 0.18\delta^2 - 0.21 \text{ where } \delta \leq 1 \qquad \text{B-5}$$

where

K is factor to take account of the different structural systems

M is design moment

d is effective depth of column

b is width of beam

f_{ck} is characteristic cylinder strength of concrete

Case-1 $K \leq K'$: No compression reinforcement required

Obtain lever arm z using expression B-6

$$z = \frac{d}{2}\left[1 + \sqrt{1 - 3.53k}\right] \leq 0.95d \qquad \text{B-6}$$

Calculate tension reinforcement required from

$$A_s = \frac{M}{f_{yd}z} \qquad \text{B-7}$$

Check minimum reinforcement requirements

$$A_{s,min} = \frac{0.26 f_{ctm} b_t d}{f_{yk}} \text{ where } f_{ck} \geq 25 \qquad \text{B-8}$$

Check maximum reinforcement requirements $A_{smax} = 0.04A_c$ for tension or compression reinforcement outside lap locations.

Case-2 $K \geq K'$: Compression reinforcement required

Obtain lever arm z using expression B-9

$$z = \frac{d}{2}\left[1 + \sqrt{1 - 3.53k'}\right] \qquad \text{B-9}$$

Calculate compression reinforcement required from

$$A_{s2} = \frac{(k - k')f_{ck}bd^2}{f_{sc}(d - d_2)} \qquad \text{B-10}$$

$$f_{sc} = 700\left[\frac{x - d_2}{x}\right] \leq f_{yd} \qquad \text{B-11}$$

Calculate tension reinforcement required from

$$A_s = \frac{k' f_{ck} bd^2}{f_{yd}z} + A_{s2}\frac{f_{sc}}{f_{yd}} \qquad \text{B-12}$$

Check maximum reinforcement requirements $A_{smax} = 0.04A_c$ for tension or compression reinforcement outside lap locations.

- ✓ **Beam shear design**

$$V_{E,d} = \gamma_{Rd} \times [M_{Rd}(top) + M_{Rd}(bottom]/l_{cl} + V_g \qquad \text{B-13}$$

where

V_g is shear force from gravity load analysis

For DCM beams, $\gamma_{Rd} = 1$

$$A_{sw}/s = V_{E,d}/(z \times f_{ywd} \times \mathrm{Cot}\,\theta) \qquad \text{B-14}$$

$$f_{ywd} = f_{yk}/1.15 \qquad \text{B-15}$$

$$V_{Rd,max} = 0.3(1 - f_{ck}(MPa)/250)b_{wo}Zf_{cd}\sin 2\delta, 1 \leq \cot\delta \leq 2.5 \qquad \text{B-16}$$

- ✓ **Definition of critical regions**

$$l_{cr} = h_w \qquad \text{B-17}$$

where

h_w is depth of the beam

- ✓ **check spacing for DCM beam**

For critical regions:

$$s \leq \{8d_{bL}; h_w/4\,; 24d_{bw}; 225mm\} \qquad \text{B-18}$$

For outside the critical regions:

$$s \leq 0.75d \qquad \text{B-19}$$

where

d_{bL} is the minimum longitudinal bar diameter (in millimeters)

h_w is beam depth (in millimeters)

d_{bw} is diameter of the hoops (in millimeters) and shall be not less than 6

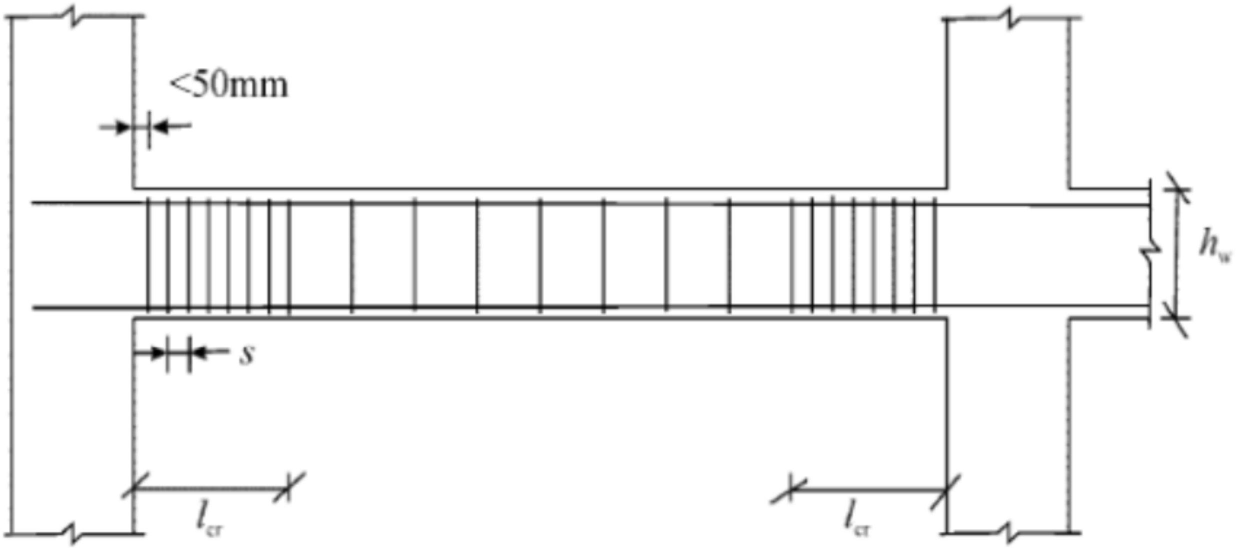


Figure B-1 : Transverse reinforcement in critical region of beams

- ✓ **Check local ductility demand using the following expression**

For critical regions

$$\rho_{max} = \rho' + \frac{0.0018}{\mu_{\phi}\varepsilon_{sy,d}} \cdot \frac{f_{cd}}{f_{yd}} \qquad \text{B-20}$$

For tension side

$$\rho_{min} = \frac{0.5 f_{ctm}}{f_{yk}} \qquad \text{B-21}$$

The reinforcement ratios of the tension zone and compression zone, ρ and ρ’, both normalized to bd.

- ✓ **Check bars crossing interior joint and anchorage at exterior joint using the expression B-22 and B-23 respectively**

For interior joint

$$\frac{d_{bL}}{h_c} \leq \frac{7.5 . f_{ctm}}{\gamma_{Rd} . f_{yd}} \cdot \frac{1 + 0.8 . v_d}{1 + 0.75 k_D . \rho'/\rho_{max}} \qquad \text{B-22}$$

For exterior joint

$$\frac{d_{bL}}{h_c} \leq \frac{7.5.\, f_{ctm}}{\gamma_{Rd}.\, f_{yd}}.\,(1 + 0.8.\, v_d) \qquad \text{B-23}$$

where

h_c is the width of the column parallel to the bars;

f_{ctm} is the mean value of the tensile strength of concrete;

f_{yd} is the design value of the yield strength of steel;

v_d is the normalized design axial force in the column, taken with its minimum value for the seismic design situation ($\nu_d = \frac{N_{Ed}}{f_{cd} \cdot A_c}$);

k_D is the factor reflecting the ductility class equal to 1 for DCH and to 2/3 for DCM;

ρ' is the compression steel ratio of the beam bars passing through the joint;

ρ_{max} is the maximum allowed tension steel ratio

γ_{Rd} is the model uncertainty factor on the design value of resistances, taken as being equal to1.2 or 1.0 respectively for DCH or DCM (due to overstrength owing to strain-hardening of the longitudinal steel in the beam).

B.2. Design and Detailing of Column

- ✓ **Determine the actions (ultimate axial load is N_{Ed} and the ultimate moments are M_{top} andM_{bottom}) on the column using an appropriate analysis method.**

- ✓ **Determine the effective length (lo)**

For braced frame

$$l_o = 0.5l\sqrt{\left(1+\frac{K_1}{0.45+K_1}\right).\left(1+\frac{K_2}{0.45+K_2}\right)} \qquad \text{B-24}$$

Unbraced members

$$l_o = l_{max}\left\{\sqrt{1+10.\frac{K_1K_2}{K_1+K_2}}\ ;\left(1+\frac{K_1}{1+K_1}\right).\left(1+\frac{K_2}{1+K_2}\right)\right\} \qquad \text{B-25}$$

In this case, the frame is assumed braced, therefore, equation B-24 was used.

where:

K_1 and K_2 are the relative flexibilities of rotational restraints at ends

$$K = (\theta/M).(EI/l) \qquad \text{B-26}$$

where

θ is the rotation of restraining members for bending moment M

EI is the bending stiffness of compression member

l is the clear height of compression member between end restraints

- ✓ **Determine design bending moment**

$$M_{Ed} = \text{Max}\{M_{02}, M_{0e}+M_2, M_{01}+0.5M_2\} \qquad \text{B-27}$$

where:

$$M_{01} = \text{Min}\{|M_{top}|, |M_{bottom}|\} + e_iN_{Ed} \qquad \text{B-28}$$

$$M_{02} = \text{Max}\{|M_{top}|, |M_{bottom}|\} + e_iN_{Ed} \qquad \text{B-29}$$

$$e_i = \text{Max}\{l_0/400\,, h/30\,, 20\} \qquad \text{B-30}$$

$\mathrm{M_{top}}$ and $\mathrm{M_{bottom}}$ are moments at the top and bottom of column respectively

$$\mathrm{M_{0e} = 0.6M_{02} + 0.4M_{01} \geq 0.4M_{02}} \tag{B-31}$$

$$\mathrm{M_2 = N_{Ed} \times e_2} \tag{B-32}$$

where

$\mathrm{N_{Ed}}$ is the design axial load and

$\mathrm{e_2}$ is deflection due to second order effects

$\mathrm{M_{02}}$ and $\mathrm{M_{01}}$ should be positive if they give tension on the same side.

A non-slender column can be designed ignoring second order effects and therefore the ultimate design moment for this study taken as $\mathrm{M_{Ed} = M_{02}}$

- ✓ **Determine slenderness**

$$\lambda = \frac{\mathrm{l_0}}{\mathrm{i}} \tag{B-33}$$

where

i is radius of gyration and

λ is slenderness

$$\lambda_{\mathrm{lim}} = \frac{\mathrm{20ABC}}{\sqrt{\mathrm{n}}} \leq \frac{\mathrm{15.4c}}{\sqrt{\mathrm{n}}} \tag{B-34}$$

$\mathrm{A} = {}^{1}/_{(1 + 0.2\varphi_{\mathrm{ef}})}$; (If φ_{ef} is not known, A= 0.7 may be used)

$\mathrm{B} = \sqrt{1 + 2\omega}$; (if ω, reinforcement ratio, is not known, B=1.1 may be used)

$\mathrm{C} = 1.7 - \mathrm{r_m}$; (if $\mathrm{r_m}$ is not known, C= 0.7 may be used)

$\mathrm{n} = {}^{\mathrm{N_{Ed}}}/_{\mathrm{A_c}} \times \mathrm{f_{cd}}$

$\mathrm{r_m = M_{01}/M_{02}}$

$\mathrm{M_{01}}$, $\mathrm{M_{02}}$ are the first order end moments, $|\mathrm{M_{02}}| \geq |\mathrm{M_{01}}|$

If the end moments $M_{01}$and M_{02} give tension on the same side, r_m should be taken positive and if slenderness (λ) is less than the slenderness limit (λ_{lim}), then second order effects may be ignored.

- ✓ **check biaxial bending using expression B-35**

$$\left(\frac{M_{Edz}}{M_{Rdz}}\right)^a + \left(\frac{M_{Edy}}{M_{Rdy}}\right)^a \leq 1.0 \qquad \text{B-35}$$

where

$M_{Edz,\, y}$= Design moment in the respective direction including second order effects in a slender column

$M_{Rdz,\, y}$= Moment of resistance in the respective direction

Value of a =1 is used for this study by considering rectangular sections and because of

$\frac{N_{Ed}}{N_{Rd}} = 0.1$

If the frame was to be designed as a moment frame in both directions, it may be designed for uniaxial bending about each direction in turn rather than considering biaxial bending, provided the uniaxial capacity is reduced by 30 per cent as per ES EN 1998-1:2015 and check normalized axial compression load for DCM,

$$\nu_d = \frac{N_{Ed}}{f_{cd} \cdot A_c} < 0.65 \qquad \text{B-36}$$

- ✓ **Use column chart to find $A_s$required for $N_{Ed}$and M_{Ed}**

The total longitudinal reinforcement ratio $ƒ_1$ shall be not less than 0.01 and not more than 0.04 and In symmetrical cross-sections symmetrical reinforcement should be provided ($ƒ= ƒ'$).

- ✓ **Definition of critical regions**

$$l_{cr} = \max\left\{h_c;\ {}^{l_{cl}}/_{6}\,; 0.45\right\} \qquad \text{B-37}$$

where

h_c is the largest cross-sectional dimension of the column (in meters)

l_{cl} is the clear length of the column (in meters)

- ✓ **Check for the requirement of strong column-weak beam using expression B-38**

$$\sum M_{Rc} \geq 1.3 \sum M_{Rb} \qquad \text{B-38}$$

$\sum M_{Rc}/\sum M_{Rb} = 2.08$ and the requirement of strong column-weak beam is satisfied.

- ✓ **Check column shear** $(\mathbf{V_{Ed}} < \mathbf{V_{Rd}})$

For a conservative design, the column shear could be based upon the flexural capacity at maximum compression. However, ES EN 1998-1:2015 allows the column flexural capacities to be multiplied by the ratio $\sum M_{Rb}/\sum M_{Rc}$ on the basis that yielding may develop initially in the beams and hence does not allow the development of the column overstrength moments.

$$V_{E,d} = \gamma_{Rd} \times \left({\sum M_{R,b}}\big/{\sum M_{R,C}}\right) \times \frac{(M_{c,top} + M_{c,bottom})}{l_{cl}} \qquad \text{B-39}$$

For DCM columns, $\gamma_{Rd} = 1.1$

$$A_{sw}/s = V_{E,d}/(z \times f_{ywd} \times \operatorname{Cot}\theta) \qquad \text{B-40}$$

$$f_{ywd} = f_{yk}/1.15 \qquad \text{B-41}$$

$$V_{Rd,max} = 0.3(1 - f_{ck}(MPa)/250) b_{wo} Z f_{cd} \sin 2\delta, 1 \leq \cot\delta \leq 2.5 \qquad \text{B-42}$$

where

$V_{Rd,max}$ is the maximum design shear resistance

A_{sw} is the cross-sectional area of the shear reinforcement

s is the spacing of the stirrups

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

z is the internal lever arm, equal to 0.9d or to the distance between the tension and the compression reinforcement, (d-d1)

b_{wo} is web thickness equal to the max of (150mm,h_{storey}/20), h storey is storey height.

- ✓ **Check spacing for DCM column**

For critical regions:

$$s \leq \{b_0/2\,;175;8d_{bL}\} \qquad \text{B-43}$$

For outside the critical regions:

$$s \leq \{12d_{bL}\,,0.6h_c,0.6b_c,240mm\} \qquad \text{B-44}$$

where

b_0 is the minimum dimension of the concrete core to the centerline of the hoops(in millimeters)

d_{bL} is the minimum diameter of the longitudinal bars (in millimeters)

h_c and b_c are depth and width of the column cross section respectively

And also check the distance between consecutive longitudinal bars engaged by hoops or cross-ties does not exceed 200 mm.

- ✓ **Check the confinement of reinforcement within a beam-column joint and in the critical regions at the base of a column meet the expression B-45**

$$\alpha\omega_{wd} \geq 30\mu_{\varphi}\nu_d.\mathcal{E}_{sy,d}.\frac{b_c}{b_o} - 0.035 \qquad \text{B-45}$$

where

ω_{wd} is the mechanical volumetric ratio of confining hoops within the critical regions $\left[\omega_d = \frac{\text{volume of confining hoops}}{\text{volume of concrete core}}.\frac{f_{yd}}{f_{cd}}\right]$;

μ_{φ} is the required value of the curvature ductility factor;

ν_d is the normalized design axial force (ν_d= $N_{Ed}/A_c,f_{cd}$);

$\mathcal{E}_{sy,d}$ is the design value of tension steel strain at yield;

h_c is the gross cross-sectional depth

h_o is the depth of confined core (to the centerline of the hoops);

b_c is the gross cross-sectional width;

b_o is the width of confined core (to the centerline of the hoops);

α is the confinement effectiveness factor, equal to $\alpha = \alpha_n \cdot \alpha_s$, with:

For rectangular cross-section:

$$\alpha_n = 1 - \sum_n b_i^2 / 6 b_o h_o \qquad \text{B-46}$$

$$\alpha_s = (1 - s/2b_o)(1 - s/2h_0) \qquad \text{B-47}$$

where

n is the total number of longitudinal bars laterally engaged by hoops or cross ties; and

b_i is the distance between consecutive engaged bars(see Figure B-2; also for b_o, h_o, s)

A minimum value of ω_{wd} equal to 0.08 should be provided within the critical region at the base of the primary seismic columns

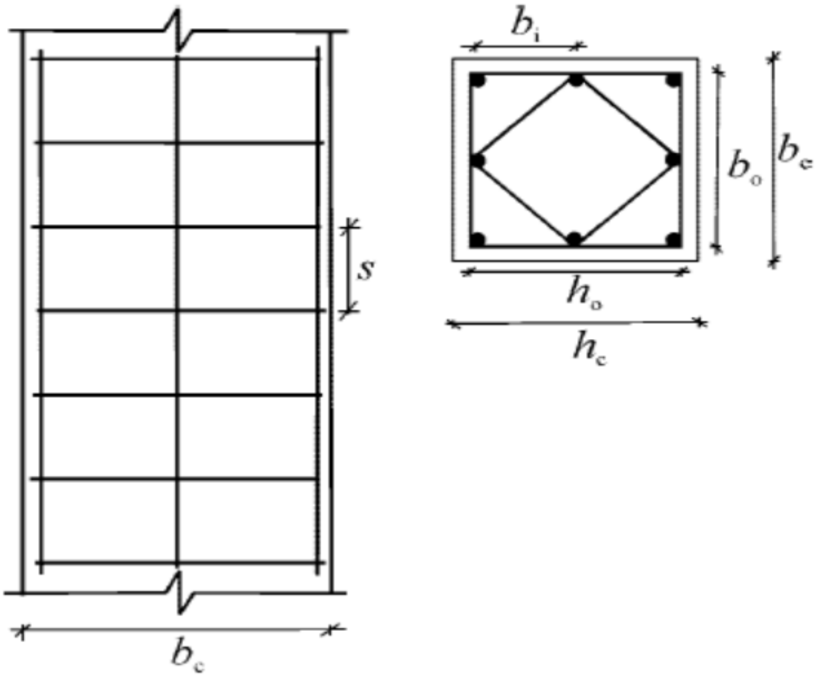


Figure B-2: confinement of concrete core

B.3. Beam-Column Joint

In DCM the horizontal confinement reinforcement in the joints of primary seismic beams with columns provided as per ES EN 1998-1:2015 for the critical regions of columns by considering the case listed in the following paragraph.

1. To ensure that there is adequate bond between reinforcement and concrete, the diameter of the main beam bars passing through the joint must be limited as given in Eqs. B-22 and B-23
2. At least one intermediate column bar is provided between each of the corners of the columns.
3. Hoops must continue unreduced through the joint from the critical region of the column, or must meet the confinement requirements of Equation B-45 if greater, unless the joint is confined on all four sides by beams. In this case, the hoop spacing may be doubled (but must not exceed 150 mm).