AN ANALYTICAL STUDY ON THE BEHAVIOUR OF EXTERIOR BEAM- COLUMN JOINT UNDER CYCLIC LOADING

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BONAFIDE CERTIFICATE

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TABLE OF CONTENTS

CHAPTER NO.	TITLE	PAGE
	ACKNOWLEDGEMENT	iii
	ABSTRACT	iv
	LIST OF SYMBOLS	v
1.	**INTRODUCTION**	
	1.1 GENERAL	1
	1.2 OBJECTIVES AND SCOPE	2
2.	**LITERATURE REVIEW**	
	2.1 GENERAL	4
	2.2 INFLUENCES OF CONCRETE STRENGTH	4
	2.3 INFLUENCES OF COLUMN LOAD ON JOINT SHEAR STRENGTH	4
	2.4 INFLUENCES OF JOINT ASPECT RATIO ON JOINT SHEAR STRENGTH	5
	2.5 INFLUENCE OF DETAILING OF REINFORCEMENT	5
	2.6 INFLUENCE OF JOINT STIRRUPS	6
	2.7 LIMITING JOINT SHEAR STRENGTH	6
	2.8 SEISMIC BEHAVIOUR OF BEAM COLUMN JOINT IN REINFORCED CONCRETE MOMENT RESISTING FRAME	7
3.	**STRUCTURAL BEHAVIOUR UNDER SEISMIC ACTION**	
	3.1 BEAM- COLUMN JOINTS	9
	3.2 FORCES ACTING ON BEAM-COLUMN JOINT	9
	3.3 PERFORMANCE CRITERIA	10
	3.4 JOINT MECHANISM	10
	3.5 BOND REQUIREMENTS	10

	3.6	EXTERIOR JOINT	11
	3.7	FACTORS AFFECTING BOND STRENGTH	12
	3.8	SHEAR REQUIREMENTS OF JOINT	12
	3.9	JOINT SHEAR AREA	13
	3.10	SHEAR RESISTING MECHANISM	13
4.	**CYCLIC BEHAVIOUR OF CONCRETE AND REINFORCEMENT**		
	4.1	PLAIN CONCRETE	18
	4.2	REINFORCEMENT	18
	4.3	REINFORCED CONCRETE	19
	4.4	DUCTILITY	19
	4.5	SIGNIFICANCE OF DUCTILITY	19
	4.6	VARIABLES AFFECTING DUCTILITY	20
	4.7	DESIGN FOR DUCTILITY	21
5.	**ANALYTICAL INVESTIGATION**		
	5.1	ASSUMPTIONS	23
	5.2	LOAD COMBINATION	23
	5.3	ANALYSIS OF JOINT	26
	5.4	DESIGN & DETAILING OF JOINT	27
	5.5	FINITE ELEMENT ANALYSIS	30
	5.6	STEPS INVOVVED IN FINITE ELEMENT ANALYSIS	31
6.	**RESULTS**		38
7.	**CONCLUSION**		43
8.	**BIBLIOGRAPHY**		44

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SYNOPSIS

The Behavior of beam-column joints is discussed, in the context of current design procedures for reinforced concrete ductile frames subjected to severe earthquake motions. As plastic hinges are expected to develop in beams, the beam-column joints must be capable of transferring large shear forces across the joint cores. The mechanisms of shear resistance of joint cores comprise a diagonal concrete strut mechanism and a truss mechanism. A considerable amount of joint core shear reinforcement is necessary to sustain the truss mechanism if bond failure of longitudinal bars is avoided. The Diameter of longitudinal beam reinforcement in joint cores needs to be restricted to ensure adequacy in anchorage in joint cores. In this thesis, a six storied RC building frame is analysed and the maximum shear forces and bending moment occurred in the beam-column joint is found and detailing of reinforcement as per IS:456-2000 and IS:13920-1993 is done. Finally analysis is carried out on the Exterior beam-column joint specimen, as per IS:456-2000 and also as per IS:13920-1993 using ANSYS-10 Software. The failure mechanism of beam column joint is studied.

LIST OF SYMBOLS

The symbols and notations given below apply to the provision of this thesis

A_h = Design horizontal seismic coefficient

D = Base dimension of the building, in meters, in the direction in which the seismic force is considered.

D = Over all depth of beam

d =Effective depth of member

DL = Response quantity due to dead load

ELx = Response quantity due to earthquake load for horizontal shaking along x- direction

ELy = Response quantity due to earthquake load for horizontal shaking along y- direction

ELz = Response quantity due to earthquake load for horizontal shaking along z- direction

F_i = Design lateral forces at the floor I due to all modes considered

f_{ck} = Characteristic compressive strength of concrete cube

F_y = Yield strength of steel

g = Acceleration due to gravity

h = Height of structure, in meters

h_i = Height measured from the base of the building to floor i

h_{st} = Storey height

I = Importance factor

IL = Response quantity due to imposed load

L_{AB} = Clear span of beam

$M^{Ah}_{u,lim}$ = Hogging moment of resistance of beam at end A

$M^{As}_{u,lim}$ = Sagging moment of resistance of beam at end A

$M^{Bh}_{u,lim}$ = Hogging moment of resistance of beam at end B

$M^{Bs}_{u,lim}$ = Sagging moment of resistance of beam at end B

$M^{bL}_{u,lim}$ = Moment of resistance of beam frame into column from the left

$M^{BR}_{u,lim}$ = Moment of resistance of beam frame into column from the right

n = Number of storey

Q_i = Lateral force at I floor

R = Response factor

S_a/g = Average response acceleration coefficient for rock or soil sites based on appropriate natural periods and damping of the structure

T = Undamped natural period of vibration of the structure

V_b = Design seismic base shear

V_b^{D+L} = Shear at end A of beam due to dead and live loads with a partial factor of safety of 1.2 on loads

V_b^{D+L} = Shear at end B of beam due to dead and live loads with a partial factor of safety of 1.2 on loads

W = Seismic weight of the structure

Wi = Seismic weight of floor i

Z = Zone factor

INTRODUCTION

CHAPTER-1

INTRODUCTION

1.1 GENERAL

In the analysis of reinforced concrete moment resisting frames, the joints are generally assumed as rigid. In Indian practice, the joint is usually neglected for specific design with attention being restricted to provision of sufficient anchorage for beam longitudinal reinforcement. This may be acceptable when the frame is not subjected to earthquake loads. There have been many catastrophic failures reported in the past earthquakes, in particular with Turkey and Taiwan earthquakes occurred in 1999, which have been attributed to beam-column joints. The poor design practice of beam column joints is compounded by the high demand imposed by the adjoining flexural members (beams and columns) in the event of mobilizing their inelastic capacities to dissipate seismic energy. Unsafe design and detailing within the joint region jeopardizes the entire structure, even if other structural members conform to the design requirements.

The first step of victory over such disastrous effects is the recognition that earthquakes are a natural phenomenon like droughts and floods and steps can be taken to ensure the damage and loss of life can be reduced. The powerful earthquake that struck the Kutch area of Gujarat on the early hours of 51st Republic day, 26 Jan 2001 has been the most devastating earthquake in the last five decades in India. After this, greater attention is given to the issues of structural design subjected to Seismic forces in India.

Earthquake engineering profession has learnt a lot from the performance of the man-made structure during the earthquake rather than from laboratory tests or from the analytical studies. The observation from the damages caused due to earthquake indicates that if the structures were properly designed and constructed for the earthquake loads most of the buildings would have been saved from the collapse, which would have reduced the total loss of life and property.

These indications made us to realize that the guidelines provided in the Seismic codes like IS :1893 -2002, "Criteria for Earthquake Resistant Design Structures' and IS:13920 -1993, "Ductile detailing of the RC structures subjected to earthquake forces" should be made mandatory for all the structures to be constructed in the future rather than being only as recommendations.

During a strong earthquake, beam-column connection is subjected to severe reversed cyclic loading. The exterior joints are not designed and detailed properly in the field. From the past study of earthquake in Gujarat and other places of world, Most of the building failed due to poor confinement in beam- column joint. The confinement was not provided in the column joint top and bottom up to a depth of 1.5 d to 2d. The critical zone of failure in column due to shear is 1d from the face of joint and similarly in beam up to distance of 2d from the face of the column.

The joint core undergoes two types of mechanism such as strut mechanism and beam mechanism. Practically no confinement provided in this region. Most of the designers not designed properly the beam-column joint since it is very tedious and complicated. There is no software available for analyzing and design of beam-column joint. No code provision for recommending joint design details except IS :1893 (part-I): 2002, IS:13920-1993 and ACI – ASCE which mentioned only beam and column design and detailing due to earthquake loading.

In recent years, large-scale general-purpose finite element codes have developed rapidly and their functions are more and more perfect. This makes it possible to apply finite element analysis on different structures. Now the higher version of ANSYS provides many new functions that can simulate, analyze and compute the mechanical behavior of structures more accurately. In this paper, exterior beam-column joint details are simulated and analyzed by using new version of ANSYS. Many new functions of this finite-element software are used to simulate each component of the joints more accurately. These joints are all originated from familiar multi-story concrete frames. During the finite element analysis, the interaction between the beam and column , as well as geometric and material nonlinearities have been considered, and the pretension force of joints are not applied by initial strains of the conventional method but a new and more appropriate method. Based on the results of FEA, behavior have been discussed.

1.2 OBJECTIVES AND SCOPE

The main objective of this study was to evaluate the performance of exterior beam – column joint which have less transverse reinforcement than required by the draft recommendation of committee 352 and IS: 13920-1993. To achieve these objectives, one exterior beam-column specimen were designed according to the provision of IS: 456-2000 and IS: 1893(part-I): 2002,from this one exterior beam column joint is analysed as per IS:456-2000 and other as per IS:13920-1993 for various detailing using ANSYS Software.

CHAPTER-2

REVIEW OF LITRATURE

2.0 GENERAL

Many authors have analysed all the test data known to them on monotonically-loaded, exterior beam-column joints to determine the influences on joint shear strength of concrete strength, column load, joint aspect ratio, reinforcement detailing, and stirrups. The effect of these parameters is discussed separately although in practice, they are interdependent.

2.1 INFLUENCES OF CONCRETE STRENGTH

The normalized joint shear strength of Parker & Bullman's beam column joint specimens was particularly low. This is thought to be due to bearing failure resulting from a combination of high bearing stresses and widely – spaced, large diameter main bars.

A regression analysis based on a power formula was carried out to determine a relationship between concrete strength and joint shear strength for the specimens without joint stirrups and joint aspect ratios between 1.33 and 1.4. The analysis showed that joint strength is almost proportional to the square root of the concrete compressive strength.

2.2 INFLUENCES OF COLUMN LOAD ON JOINT SHEAR STRENGTH

Beam- column joints can fail either flexure (in the beam or upper column) or shear. Some of Parker & Bullman's beam- column specimens failed because of flexure in the upper column. Back- analysis indicates that the upper column failed at lower loads than predicted by section analysis, assuming that plane section remain plane. This is consistent with measurements of column bar strains made by investigators, including Scott and Ortiz. Back- analysis of Scott and Ortiz data shows that the strains measured in the inner column bars at the top of the joint were greater than predicted by conventional analysis.

2.3 INFLUENCES OF JOINT ASPECT RATIO ON JOINT SHEAR STRENGTH

Joint aspect ratio is defined as the ratio between the height of beam to height of column i.e. h_b/h_c. The joint shear strength of specimens without significant joint stirrups reduces with increasing joint aspect ratio. Scott tested 12 specimens with a joint aspect ratio of 1.4 and three specimens with a joint aspect ratio of 2.0. All the specimens were reinforced with a single R6 stirrups at the centre if the beam, and the beams were reinforced with either L bars or U bars. Comparing the joint shear strength of the specimens with L bars and U bars can see the effect of joint aspect ratio. Specimens C7, c1AL, C4A and C4Al (see Table 1) were reinforced with L bars; specimen C7 had a joint aspect ratio of 2, and the other specimens had joint aspect ratios of 1.4. The mean normalized joint shear strength $V_j/b_c h_c \sqrt{f_c'}$ of specimen C7 was .78 N/mm^2 which is 26% less than the mean normalized joint strength of specimens C1AL, C4, C4A and C4AL. A similar difference in joint shear strength occurred for the specimens with U bars.

2.4 INFLUENCE OF DETAILING OF REINFORCEMENT

Taylor, Scott Scott & Hamil tested similar specimens with the beam steel detailed as both L bars and U bars. Comparation of the normalized joint shear strengths of similar specimens reinforced with L bars and U bars shows that the joint shear strength of the specimens with U bars was about 20 % less than that of specimens with L bars.

Ortiz varied the radius of bend in the beam reinforcement between four and eight bar diameters in tests on similar specimens without joint stirrups and found that it had little effect on joint shear strength. Ortiz also varied the percentage of column bars from 2.05% to 3.4 % in tests on similar specimens without joint stirrups and found that it had no significant effect on joint shear strength.

2.5 INFLUENCE OF JOINT STIRRUPS

Stirrups are considered effective (i.e. able to increase joint strength) only if positioned above the compressive zone of the incoming beam and below the main beam reinforcement. Stirrups are also considered to be more efficient at increasing joint shear strength if positioned above the centre line of the beam.

2.6 LIMITING JOINT SHEAR STRENGTH

The test data were analysed to determine the limiting joint shear strength. The maximum shear strength of the specimens tested by Talor(B3/41/24), Scott&Hamil (C4ALN3 and C4ALN5), Ortiz (BCJ7). And Kordina (RE7), ranged between 1.23 $b_c hc\sqrt{f_c}$ and 1.30 $b_c h_c\sqrt{f_c}$.Strriups strain was measured in the tests of Ortiz and Scott &Hamil (excludingC4ALN5). These showed that the stirrups did not yield in tests BCJ7 and C4ALN. It is not known whether the stirrups yielded in the other tests listed above. The aspect ratio h_b/h_c of the specimens listed above varied between 1.33 and 1.43. Consideration of the inclined stress field model proposed by Scott suggests that the maximum joint shear strength should be related to the joint aspect ratio. Further tests are required to confirm this.

2.7 SEISMIC BEHAVIOUR OF BEAM COLUMN JOINT IN A REINFORCED CONCRETE MOMENT RESISTING FRAME- S.R.UMA AND A.MEHER PRASAD

The beam column joint is the crucial zone in a reinforced concrete moment resisting frame. It is subjected to large forces during severe ground shaking and its behaviour has a significant influence on the response of the structure. The assumption of joint being rigid fails to consider the effects of high shear forces developed within the joint.The shear failure is always brittle in nature which is not an acceptable structural performance especially in seismic conditions. This paper presents a review of the postulated theories associated with the behaviour of joints. Understanding the joint behaviour is essential in exercising proper judgments in the design of joints. The paper discusses about the seismic actions on various types of joints and highlights the critical parameters that affect joint performance with special reference to bond and shear transfer.

STRUCTURAL BEHAVIOUR UNDER SEISMIC ACTION

CHAPTER-3

STRUCTURAL BEHAVIOUR UNDER SEISMIC ACTIONS

The seismic design philosophy relies on providing sufficient ductility to the structure by which the structure can dissipate seismic energy. The structural ductility essentially comes from the member ductility wherein the latter is achieved in the form of inelastic rotations. In reinforced concrete members, the inelastic rotations spread over definite regions called as plastic hinges. During inelastic deformations, the actual material properties are beyond elastic range and hence damages in these regions are obvious. The plastic hinges are "expected" locations where the structural damage can be allowed to occur due to inelastic actions involving large deformations. Hence, in seismic design, the damages in the form of plastic hinges are accepted to be formed in beams rather than in columns. Mechanism with beam yielding is characteristic of strong-column-weak beam behaviour in which the imposed inelastic rotational demands can be achieved reasonably well through proper detailing practice in beams.

Therefore, in this mode of behavior, it is possible for the structure to attain the desired inelastic response and ductility. On the other hand, if plastic hinges are allowed to form in columns, the inelastic rotational demands imposed are very high that it is very difficult to be catered with any possible detailing. The mechanism with such a feature is called column yielding or storey mechanism.

One of the basic requirements of design is that the columns above and below the joint should have sufficient flexural strength when the adjoining beams develop flexural overstrength at their plastic hinges. This column to beam flexural strength ratio is an important parameter to ensure that possible hinging occurs in beams rather than in columns.

3.1 BEAM COLUMN JOINT

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

TYPES OF JOINTS IN FRAMES

The joint is defined as the portion of the column within the depth of the deepest beam that frames into the column. In a moment resisting frame, three types of joints can be identified viz.interior joint, exterior joint and corner joint. 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 each frames into two adjacent vertical faces of a column, then the joint is called as a corner joint.

The severity of forces and demands on the performance of these joints calls for greater understanding of their seismic behaviour. These forces develop complex mechanisms involving bond and shear within the joint.

3.2 FORCES ACTING ON A BEAM COLUMN JOINT

The pattern of forces acting on a joint depends upon the configuration of the joint and the type of loads acting on it.

The shear force in the joint gives rise to diagonal cracks thus requiring reinforcement of the joint. The detailing patterns of longitudinal reinforcements significantly affect joint efficiency. The bars bent away from the joint core result in efficiencies of 25-40 % while those passing through and anchored in the joint core show

85-100% efficiency. However, the stirrups have to be provided to confine the concrete core within the joint.

3.3 PERFORMANCE CRITERIA

The moment resisting frame is expected to obtain ductility and energy dissipating capacity from flexural yield mechanism at the plastic hinges. Beam-column joint behaviour is controlled by bond and shear failure mechanisms, which are weak sources for energy dissipation. The performance criteria for joints under seismic actions may be summarized as follows:

1. The joint should have sufficient strength to enable the maximum capacities to be mobilized in the adjoining flexural members.
2. The degradation of joints should be so limited such that the capacity of the column is not affected in carrying its design loads.
3. The joint deformation should not result in increased storey drift.

3.4 JOINT MECHANISM

In the strong column-weak beam design, beams are expected to form plastic hinges at their ends and develop flexural overstrength beyond the design strength. The high internal forces developed at plastic hinges cause critical bond conditions in the longitudinal reinforcing bars passing through the joint and also impose high shear demand in the joint core. The joint behavior exhibits a complex interaction between bond and shear. The bond performance of the bars anchored in a joint affects the shear resisting mechanism to a significant extent.

3.5 BOND REQUIREMENTS

The flexural forces from the beams and columns cause tension or compression forces in the longitudinal reinforcements passing through the joint. During plastic hinge formation, relatively large tensile forces are transferred through bond. When the longitudinal bars at the joint face are stressed beyond yield splitting cracks are initiated along the bar at the joint face which is referred to as 'yield penetration'. Adequate development length for the longitudinal bar is to be ensured within the joint taking yield penetration into consideration. Therefore, the bond requirement has a direct implication on the sizes of the beams and columns framing into the joint.

3.6 EXTERIOR JOINT

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 . Hooks are helpful in providing adequate anchorage when furnished with sufficient horizontal development length and a tail extension. Because of the likelihood of yield penetration into the joint core, the development length is to be considered effective from the critical section beyond the zone of yield penetration. Thus, the size of the member should accommodate the development length considering the possibility of yield penetration.

When the reinforcement is subjected to compression, the tail end of hooks is not generally helpful to cater to the requirements of development length in compression. However, the horizontal ties in the form of transverse reinforcement in the joint provide effective restraints against the hook when the beam bar is in compression .

3.7 FACTORS AFFECTING BOND STRENGTH

The significant parameters that influence the bond performance of the reinforcing bar are confinement, clear distance between the bars and nature of the surface of the bar.Confinement of the embedded bar is very essential to improving the bond performance in order to transfer the tensile forces. The relevant confinement is obtained from axial compression due to the column and with reinforcement that helps in arresting the splitting cracks. Joint horizontal shear reinforcement improves anchorage of beam bars . But, there is an upper bound to the beneficial effects of confinement. At this limit, maximum bond strength is attained beyond which the crushing of concrete in front of the rib portion of the deformed bar occurs. Research indicates better bond performance when the clear distance between the longitudinal bars is less than 5 times the diameter of the bar . As expected, the deformed bars give better performance in bond. The behavior of the reinforcing bar in bond also depends on the quality of concrete around the bar.

3.8 SHEAR REQUIREMENTS OF JOINT

The external forces acting on the face of the joint develop high shear stresses within the joint. The shear stresses give rise to diagonal stresses causing diagonal cracks when tensile stresses exceed the tensile strength of concrete. Extensive cracking occur within the joint under load reversals, affecting its strength and stiffness and hence the joint becomes flexible enough to undergo substantial shear deformation (distortion). Before discussing the shear beahviour in detail, it is imperative to arrive at the shear force demand on joints.

The determination of shear force in the vertical and horizontal direction is usually essential. However, since well established code procedures aim at the beam hinging mechanism, it is generally sufficient to discuss the shear force demand in the horizontal direction only.

SHEAR FORCE IN AN EXTERIOR JOINT

The features of an exterior beam column joint where one beam frames into the column. Based on equilibrium principles, the column shear and the horizontal shear force in the joint can be calculated as follows.

The column shear force is

$$V_{col} = \frac{T_b Z_b + V_b (h_c/2)}{2}$$

and the horizontal shear across the joint can be expressed as

$$V_{jh} = V_{col} (l_c/z_b - 1) - V_b (h_c/2z_b)$$

3.9 JOINT SHEAR AREA

The relative severity of joint shear forces may be conveniently expressed in terms of shear stresses. The cross sectional area over which the shear forces can be transferred cannot be defined uniquely. Since the joint is the zone of intersection of beams and columns, the shear area of the joint is to be specified based on the dimensions of the beams and columns. The effective shear area *Aj* is defined by the width of the joint, *bj*, and the depth of the joint *hj*. The area effective in resisting joint shear may not be as large as the column's entire cross section area since the width of the beam, *bw* and the column, *bc* may differ from each other. The codes recommend effective joint shear area based largely on engineering approximations. The depth of the joint *hj* is taken as the depth of the column, *hc*.

3.10 SHEAR RESISTING MECHANISM

In general, the joint region is idealized as a two dimensional plane subjected to internal forces from the beam and the column acting on the joint face. The forces primarily consist of compressive, tensile and shear forces. The shear forces in the joint region develop diagonal compressive and tensile forces within the joint core, resulting in the formation of a diagonal failure plane. The essential components of the shear resisting mechanism are discussed with respect to joint.

. 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 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. The strength of the strut mechanism depends on the compressive strength of concrete and that of the truss mechanism on the tensile yield strength of the lateral reinforcement crossing the failure plane.

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.

JOINT SHEAR STRENGTH

The joint shear strength is affected by the parameters influencing the two principal shear resisting mechanisms. The total strength contributed by each mechanism can be considered as the shear strength of the joint in the horizontal direction and is given as

$$Vjh = Vch + Vsh$$

in which Vch is the contribution from the concrete strut and Vsh is the contribution from the truss mechanism. The contribution of each mechanism is affected significantly by the prevailing bond conditions as discussed in the previous sections and also by the contributions from the slab .

CONTRIBUTION FROM STRUT AND TRUSS MECHANISM

The shear force in the joint is considered to be resisted by two principal mechanisms viz. the strut and the truss mechanisms. In the previous section, the role of the strut and the truss mechanism in resisting joint shear with respect to prevailing bond conditions has been discussed. To recapitulate a few points, the truss mechanism is supported by good bond transfer and well distributed vertical and horizontal reinforcement in the joint core. This mechanism tends to diminish in case of bond deterioration and the lateral reinforcements can no longer be utilised for taking up joint shear. The compressive strength of the diagonal concrete strut is the reliable source for resisting joint shear. The strength of the diagonal concrete strut in turn is affected by the tensile strain (or tensile stresses) in the core concrete. At this stage, the lateral reinforcement provides confinement to improve the efficiency of the concrete in the strut mechanism.

Based on the above observations, formulations have been suggested for the design of joints for shear. The recommendations focus on the following two major aspects:

1. Determination of the nominal shear stress in terms of a function of the compressive stress of core concrete cf', or in terms of the tensile stress of core concrete expressed as function of cf'.
2. Provision of lateral reinforcement with specifications for the spacing and the area of the ties for confinement effect and to support the truss mechanism.

At an exterior joint only one beam frames into a column and hence the joint shear will be generally less than that encountered in an interior joint. The shear transfer mechanism within the joint core will be similar to that postulated for interior joints and will comprise of a concrete strut and a truss sustaining the diagonal compression field. The diagonal strut will be developed between the bend of the hooked top tension bar and the diagonally opposite corner of the joint where compression forces in both the vertical and horizontal directions are introduced. If adequate anchorage of the beam flexural tension reinforcement in the form of a standard hook is provided, the contribution from strut mechanism will be taken care of. The truss mechanism shall be sustained by the longitudinal bars and the confining stirrups. The amount of stirrups needed for this may be obtained by considering the total tensile force contributed by the steel reinforcement, *As1,* including the effective flange width.

DESIGN OF SHEAR REINFORCEMENT

Presence of horizontal and vertical shear reinforcement within joint can develop truss mechanism in resisting shear. The design of shear reinforcement is governed by the minimum reinforcement area needed to support the truss mechanism and the maximum permissible area based on the limit stress corresponding to diagonal compression failure. As a minimum requirement, horizontal hoop reinforcement has to be designed for 40% of the total horizontal shear force.

IS :13920:1993 has been revised and draft recommendations with supporting design examples are available in open publications 17.After arriving at design horizontal shear, the vertical shear can be approximated when the columns do not form plastic hinges as:

$$Vjv=Vjh(hb/hc)$$

In general, intermediate column longitudinal bars are expected to contribute to vertical shear and if they amount to 1/3 of the total longitudinal column reinforcement, no additional vertical shear reinforcement is found to be necessary. The bond force in the column bars extending into the joint core forms a part of the truss mechanism. Vertical transverse reinforcements are usually provided by the intermediate column bars. This necessitates every column to have at least one intermediate bar on each face of the column.The required horizontal shear reinforcement to resist *Vsh* is to be provided in the form of closed stirrups, cross ties or overlapping hoops. The stirrups and ties are preferred to be bent with 135 degree hooks having an extension of 6d, where d is the diameter of the stirrup. The arrangement of stirrups with regard to the orientation of the 90 deg and 135 deg hooks should be such that effective core confinement is available in the joint. The spacing requirement of horizontal stirrups is also governed by the buckling criteria of column bars passing through the joint. The lateral spacing of the stirrups is to be restricted so as to effectively transfer the bond force in the column bar into the joint core such that it forms a part of the truss mechanism.On the research front, various detailing schemes have been tried so as to study the seismic performance of exterior joints . The use of continuous beam bars bent in the form of a U placed horizontally and distributed through the depth of the beam provided a simple detail that worked effectively.

CYCLIC BEHAVIOUR OF CONCRETE AND REINFORCEMENT

CHAPTER – 4

CYCLIC BEHAVIOUR OF CONCRETE AND REINFORCEMENT

4.1 PLAIN CONCRETE

Plain concrete is a brittle material. During the first cycle the stress-strain curve is the same as that obtained from static tests. If the specimen is unloaded and reloaded in compression, stress-strain curves are similar . It can be seen that slope of the stress-strain curves as well as the maximum attainable stress decrease with number of cycles. Thus the stress-strain relationship for plain concrete subjected to repeated compressive loads is cycle dependant. The decrease in stiffness and strength of plain concrete is due to the formation of cracks. The compressive strength of concrete depends on the rate of loading. As the rate of loading increase, the compressive strength of concrete increase but the strain at the maximum stress decreases. Plain concrete cannot be subjected to repeated tensile loads since its tensile strength is practically zero.

4.2 REINFORCEMENT

Reinforcing steel has much more ductility than plain concrete. The ultimate strain in mild steel is of the order of 25% whereas, in concrete it is of the order of 0.3%. In the first cycle the reinforcing steel shows stress-strain curves similar to that obtained in the static test. After the specimen has reached its yield level and direction of load is reversed, that is, unloading begins, it can be seen that the unloading curve is not straight but curvilinear. This curve in the unloading segment of stress-strain curve is referred to as the Bauschinger effect after the discoverer of the phenomenon.

4.3 REINFORCED CONCRETE

Plain concrete can be subjected only to repeated compressive loading cycles and not to repeated tensile loading cycles due to its poor tensile strength. However, reinforcing steel can be subjected to repeated reversible tensile and compressive loading cycles and exhibits stable hysteresis loops. Thus the cyclic behavior of reinforced concrete members is significantly improved due to the presence of reinforcing steel.

4.4 DUCTILITY

Member or structural ductility is defined as the ratio of absolute maximum deformation to corresponding yield deformation. The ductility μ is defined by the equation:

$\mu = \Delta_u / \Delta_y$ with respect to displacement

$= \partial_u / \partial_y$ with respect to curvature

$= \acute{\emptyset}_u / \acute{\emptyset}_y$ with respect to rotation

4.5 SIGNIFICANCE OF DUCTILITY

When a ductile structure is subjected to overloading it will tend to deform inelatically and in doing so, will redistribute the excess load to elastic parts of the structure. This concept can be used in several ways.

(i) If a structure is ductile, it can be expected to adapt to unexpected overloads, load reversals, impact and structural movements due to foundation settlement and volume changes. These items are generally ignored in the analysis and design but are assumed to have been taken care of by presence of some ductility in the structure.

(ii) If a structure is ductile, its occupants will have sufficient warning of the impending failure thus reducing the probability of loss of life in the event of collapse.

(iii) The limit state design procedure assumes that all the critical sections in the structure will reach their maximum capacities at design load for the structure.

For this to occur, al joints and splices must be able to withstand forces and deformations corresponding to yielding of the reinforcement.

4.6 VARIABLES AFFECTING THE DUCTILITY

1. TENSION STEEL RATIO *P*

The ductility of a beam cross-section increases as the steel ratio p or ($p-p_c$) decreases. If excessive reinforcement is provided, the concrete will crush before the steel yields, leading to a brittle failure corresponding to $\mu = 1.0$. In other wards, a beam should be designed as under reinforced.

The ductility increases with the increase the characteristic strength of concrete, and decreases with the increase in characteristic strength of steel. In fact, ductility is inversely proportional to the square of σy. It suggests that Fe 250 grade mild steel is more desirable from the ductility point of view as compares with the Fe 415 grade or Fe 500 grade high strength steels.

2. COMPRESSION STEEL RATIO P

It suggests that ($p-p_c$) is an important parameter defining the ductility ratio. The ductility increases with the decrease in ($p-p_c$) value, that is, ductility increases with the increase in compression steel.

3. SHAPE OF CROSS SECTION

The presences of an enlarged compression flange in a T- beam reduce the depth of the compression zone at collapse and thus increase the ductility.

4. LATERAL REINFORCEMENT

Lateral reinforcement tends to improve ductility by preventing premature shear failure, restraining the compression reinforcement against buckling and by confining the compression zone, thus increasing deformation capacity of a reinforced concrete beam

4.7 DESIGN FOR DUCTILITY

Selection of cross section that will have adequate strength is rather easy. But it is much more difficult to achieve the desired strength as well as ductility. To ensure sufficient ductility, the designer should pay attention to detailing of reinforcement. Bar cut-offs, splicing and joint details. Sufficient amount of ductility can be ensured by following certain simple design details such as;

(i)The structural layout should be simple and regular avoiding offsets of beams to columns, or offsets of columns from floor to floor. Changes in stiffness should be gradual from floor to floor.

(ii) The amount of tensile reinforcement in beams should be restricted and more compression reinforcement should be provided. The latter be enclosed by stirrups to prevent it from buckling.

(iii) Beams and columns in a reinforced concrete frame should be designer in such a manner that inelasticity is confined to beams only and the columns should remain elastic. To ensure this, sum of the moment capacities of the columns for the design axial loads at a beam- column joint should be greater than the moment capacities of the beams along each principle plane.

$$\sum M_c = 1.2\ M_b$$

Thus the flexural resistances are summed such that the column moments oppose the beam moments.

(iv) The shear reinforcement should be adequate to ensure that the strength in shear exceeds the strength in flexure and thus, prevent a non- ductile shear failure before the fully reversible flexural strength of a member has been developed.

(v) Closed stirrups or spirals should be used to confine the concrete at sections of maximum moment to increase the ductility of members. Such sections include upper and lower ends of columns and within beam-column joints which do not have beams on all sides. If an axial load exceeds 0.4 times the balanced axial load, a spiral column is preferred.

(vi) Splices and bar anchorages must be adequate to prevent bond failures.

(vii) The reversal of stresses in beams and columns due to reversal of direction of earthquake forces must be taken into account in the design by appropriate reinforcement.

(viii) Beam-column connections should be made monolithic.

CHAPTER-5

ANALYTICAL INVESTIGATION

The design of six storey reinforced concrete building frame can be analysed. From that , first storey exterior beam column joint of the concrete frame is designed and analysed using ANSYS software. The live load and dead load calculated as per IS :875 (part-2)-1987 table-1 and IS :875 (part-1)-1987 table-1. Wind load is not considered in the design. While designing earthquake resistant structure the following assumption shall be made.

5.1 ASSUMPTION

a) Earthquake causes impulsive ground motions, which are complex and irregular in character, changing in period and amplitude each lasting for a small duration. Therefore, resonance of the type as visualized under steady- state sinusoidal excitations will not occur as it would need time to build up such amplitudes.
b) Earthquake is not likely to occur simultaneously with wind or maximum flood or maximum sea waves.
c) The values of elastic modulus of materials, wherever required, may be taken as for static analysis unless a more definite value is available for use in such condition

5.2 LOAD COMBINATION

In the limit state design of reinforced concrete structure, the following load combinations shall be accounted for:

1) LL
2) DL

3) 1.5(DL+LL)
4) 1.5(DL-LL)
5) 1.2(DL+LL+EL)
6) 1.2(DL+LL-EL)
7) 1.5(DL+EL)
8) 1.5(DL-EL)
9) 0.9DL+1.5EL
10) 0.9DL-1.5EL

DESIGN OF HORIZONTAL EARTHQUAKE LOAD

When the lateral load resisting elements are not oriented along the orthogonal horizontal direction, the structure shall be designed for the effects due to full design earthquake load in one horizontal direction plus 30 percent of the design earthquake load in the other direction.

DESIGN OF VERTICAL EARTHQUAKE LOAD

When effects due to vertical earthquake load are to be considered, the design vertical force shall be calculated in accordance with 6.4.5 of IS1893(part-I) :2002

COMBINATION FOR THREE COMPONENTS MOTION

Assuming three earthquake components are to considered for the design, the response (EL) due to the combined effects of the three components can be obtained on the basis of 'Square root of the square (SRSS)' that is

$$EL = \sqrt{EL_X^{\,2} + EL_y^{\,2} + EL_Z^{\,2}}$$

INCREASE IN PERMISSIBLE STRESSES

When earthquake forces are considered along with other normal design forces, the permissible stresses of material, in the elastic method of design, may be increased by one-third. However, for steels having a definite yield stress, the stress is limited to the yield stress; for steels will be limited to 80 percent of the ultimate strength or 0.2 percent proof stress, whichever is smaller.

DESIGN SPECTRUM

The design horizontal seismic coefficient A_h for a structure shall be determined by the following expression

$$A_h = Z\, I\, S_a / 2\, R\, g$$

Provided that for any structure with $T \leq 0.1$ s, the value of A_h will not be taken less than Z/2 whatever be the value of I/R

For medium soil sites

$$S_a/g = 1+15T,\ 00 \leq T \leq 0.10$$
$$2.5,\quad 00 \leq T \leq 0.10$$
$$1.36T,\ 00 \leq T \leq 0.10$$

The importance factor and response factor for design purpose taken from IS 1893 (part-I) table -6 & 7

DESIGN LIVE LOADS FOR EARTHQUAKES FORCE CALCULATION

For various loading classes as specified in IS:875 (part-2), the earthquake force shall be calculated for the full dead load plus the percentage of imposed load as given in table 8. for calculating the design seismic forces of the structure, the imposed load on roof need not be considered.

SEISMIC WEIGHT

The seismic weight of each floor is its full dead load plus appropriate amount of imposed load, as specified in above. While computing the seismic weight of each floor, the weight of columns and walls in any storey shall be equally distributed to the floors above and below the storey. The seismic weight of the whole building is the sum of the seismic weight of all the floors.

DESIGN OF SEISMIC BASE SHEAR

The design lateral force along any principal direction shall be determined by the following expression.

$$V_B = A_h W$$

FUNDAMENTAL NATURAL PERIOD

The approximate fundamental natural period of vibration (T_a) in seconds of all other buildings, including moment-resisting frame building with brick infill panel, may be estimated by the empirical formula

$$T_a = .009h / \sqrt{d}$$

DISTRIBUTION BASE SHEAR

The design base shear is distributed along the height of the building as per the following expression.

$$Qi = V_B \; W_i h_i^2 / \sum^{n}_{j=1} W_j h_j^2$$

5.3 ANALYSIS OF JOINT

The beam-column joint model is created in the ANSYS Software.The section properties, loads, safety factor, load combination, yield stress of steel, young's modulus of concrete & steel and other necessary data were given in the input. The software performed the analysis.

5.4 DESIGN AND DETAILING OF JOINT

FLEXURAL MEMBER

While design and detailing of reinforced concrete buildings, the members shall satisfy the following requirements.

1. The factored axial stress on the member under earthquake loading shall not. exceed 0.1 f_{ck}
2. The member shall preferably have a width to depth ratio of more than 0.3.
3. The width of the member shall not be less than 200 mm
4. The depth of the member shall preferably be not more than ¼ of the member length.

LONGITUDINAL REINFORCEMENT

(a)The top and bottom of longitudinal reinforcement shall consist of at least two bars through the member length.

(b)The tension steel ratio on any face at any section, shall not be less than ρ_{min} =0.24 $\sqrt{f_{ck}}/f_y$ where f_{ck} and fy are in N/mm^2

(c)The maximum steel ratio on any face at any section, shall not be exceed ρ_{max}=0.025.

I. The positive steel at a joint face must be at least equal to half the negative steel at the\at face.

II. The steel provided at each of the top and bottom face of the member at any section along its length shall be at least equal to one- fourth of the maximum negative moment steel provided at the face of the either joint.

III. In an external joint, both the top and bottom bars of the beam shall be provided with anchorage length, beyond inner face of the column, equal to the development length in tension plus 10 times the bar diameter minus the allowance for 90 degree bend.

IV. The longitudinal bars shall be spliced, only if hoops are provided over the entire

V. splice length, at a spacing not exceeding 150 mm. the lap length shall not be less than the bar development length in tension. Lap splice shall not be provided within a joint, within a distance of 2d from the face and within a quarte length of the member where flexural yielding may generally occur under effect of earthquake forces. Not more than 50% of the bars shall be spliced at one section.

WEB REINFORCEMENT

Web reinforcement shall consist of vertical hoops. A vertical hoop is a closed stirrup having a 135° hook with a 10-diameter extension (but not <75mm) at each end that is embedded in the confined core. In compelling circumstances, it may also be made up of two pieces of reinforcement;

The minimum diameter of the bar forming a hoop shall be 6mm. however in the beam with clear span exceeding 5m, the minimum bar diameter shall be 8mm.

The shear force to be resisted by the vertical hoops shall be the maximum of:

(a) Calculated factored shear force as per analysis, and

(b) Shear force due to formation of plastic hinges at both ends of the beam plus the factored gravity load on the span,

(i) for sway to right:

$$V_{u,a} = V_a^{D+L} - 1.40[M_u^{AS}{}_{lim} + M_u^{Bh}{}_{lim}] / L_{AB}$$

$$V_{u,b} = V_b^{D+L} + 1.40[M_u^{AS}{}_{lim} + M_u^{Bh}{}_{lim}] / L_{AB}$$, and

(ii) for sway to left

$$V_{u,a} = V_a^{D+L} + 1.40[M_u^{Ah}{}_{lim} + M_u^{Bs}{}_{lim}] / L_{AB}$$

$$V_{u,b} = V_b^{D+L} - 1.40[M_u^{Ah}{}_{lim} + M_u^{Bs}{}_{lim}] / L_{AB}$$

The spacing of hoops over a length of 2d at either end of a beam shall not exceed (a) d/4, and (b) 8 times the diameter of the smallest longitudinal bar; however it need not be less than 100 mm. The first hoop shall be at a distance not exceeding 50mm from the joint face. Vertical hoops at the same spacing as above shall also be provided over a length equal to 2d on either side of a section where flexural yielding may occur under the effect of earthquake forces, elsewhere, the beam shall be provided with vertical hoops at a spacing not exceeding d/2.

COLUMNS AND FRAME MEMBERS SUBJECTED TO BENDING AND AXIAL LOAD

These requirements apply to frame members, which have a factored axial stress in excess of 0.1 f_{ck} under the effect of earthquake forces.

The minimum dimension of the member shall not be less than 200mm. however, in frames which have beams with centre to centre span exceeding 5m or columns of unsupported length exceeding 4m, the shortest dimension of column shall not be less than 300mm.

The ratio of the shortest cross sectional dimension to the perpendicular dimension shall preferably not be less than 0.4.

LONGITUDINAL REINFORCEMENT

Lap splices shall be provided only in the central half of the member length, it should be proportioned as a tension splice. Hoops shall be provided over the entire splice length at spacing not exceeding 150mm centre to centre. Not more than 50 percent of the bars shall be spliced at one section.

TRANSVERSE REINFORCEMENT

Transverse reinforcement for rectangular columns is provided with closed stirrups, having a 135° hook with a 10-diameter extension (but not <75mm) at each end that is embedded in the confined core. The spacing of hoops shall not be exceeding half the least lateral dimension of the column except where special confinement is provided.

The design shear force for the columns shall be the following of:

(a) Calculated factored shear force as per analysis and

(b) A factored shear force given by

$$V_u = 1.40\ [M_u{}^{bL}{}_{lim} + M_u{}^{bR}{}_{lim}]\ /\ h_{st}$$

SPECIAL CONFINING REINFORCEMENT

Special confining reinforcement shall be provided over a length l_o from each joint face, towards mid span, and on either side of any section, where flexural yielding may occur under the effect of earthquake forces. The length l_o shall not be less than (a) larger lateral dimension of the member at the section where yielding occurs, (b) 1/6 of clear span of the member, and (c) 450mm

JOINTS OF FRAMES

The special confining reinforcement as required at the end of column shall be provided through the joint as well, unless the joint is confined as specified by the following:

A joint, which has beams framing into all vertical faces of it and where each beam width is at least ¾ of the column width, may be provided with half the special confining reinforcement required at the end of the column. The spacing of hoops shall not exceed 150mm.

5.5 FINITE ELEMENT ANALYSIS

The finite element method is a numerical analysis technique for obtaining approximate solutions to a wide variety of engineering problems.A finite element model of a problem gives a piecewise approximation to the governing equations.The basic premise of the finite element method is that a solution region can be analytically modelled or approximated by replacing it with an assemblage of discrete elements.Since these elements can be put together in a variety of ways,they can be used to represent exceedingly complex shapes.

5.6 STAGES IN FINITE ELEMENT SOLUTION

In general,a finite element solution may be broken in to the following three stages.

1.Preprocessing:defining the problem

- Define keypoints/lines/areas/volumes
- Define element type and material/geometric properties
- Mesh lines/areas/volumes as required

Different sections of this module are,

1. Defining the parameters-Element types,Real constants,Material properties
2. Geometry of the structure
3. Meshing

DEFINING THE PARAMETERS

Element Type

- In ANSYS program, the element used for analysis is **SOLID187-3 Dimensional 10 Noded Tetrahedral Structural Solid.**
- **SOLID187** is an higher order 3D,10 noded element.
- It has quadratic displacement and well suited to modelling irregular meshes.
- It has three degrees of freedom at each node along x,y,z.
- **Special features**-Plasticity,hyper elasticity,creep,stress stiffening,large deflection,and large strain capabilities.

Real Constants

The real contants of the element are the properties that are specific to a given element type such as the cross sectional properties.

Material Properties

Material properties which are frequently used are Youngs Modulus,Density,Poissons ratio,etc.

2. Geomentry of the structure

The ultimate purpose of the finite element analysis is to create an accurate mathematical model of a physical prototype.Model generation is the process of defining the geometric configuration of the model nodes and elements.The model used for my analysis is Solid Model.

3. Meshing

Meshing consists of three steps,

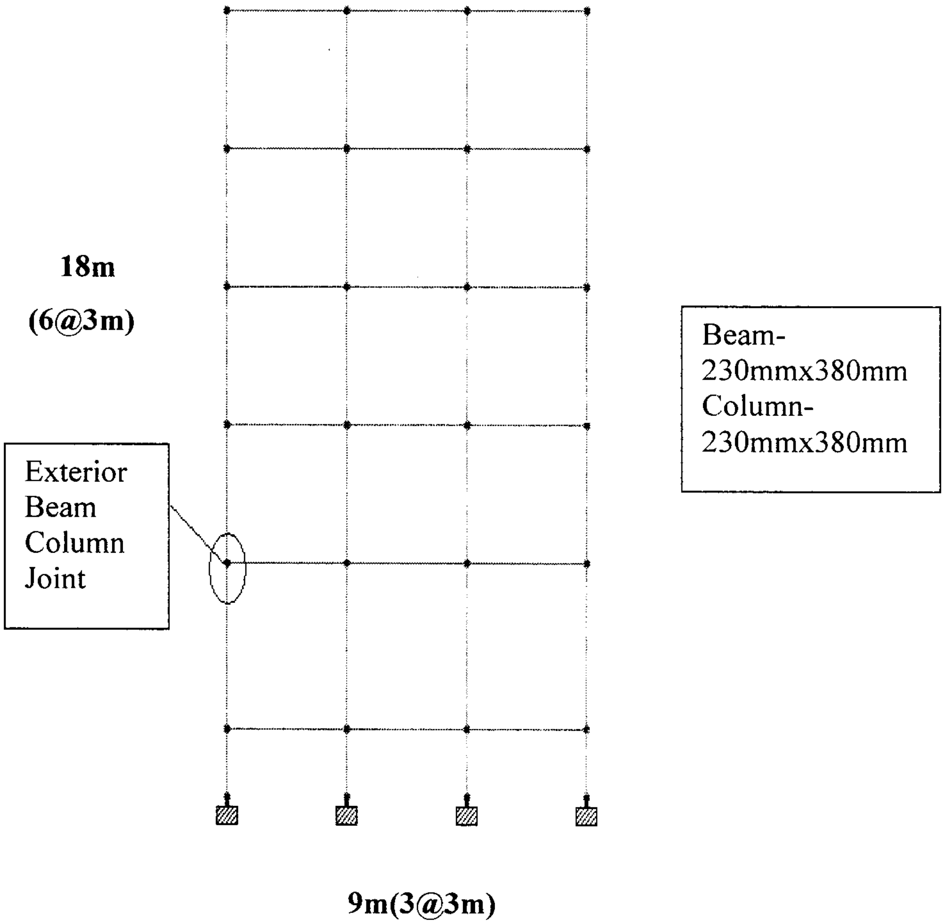
- Setting of element attributes
- Mesh control
- Shape

Free meshing is used for analysis of joint.

4. **Solutions-assigning loads,constraints and solving**

5. **Post-Processing:further processing and viewing of the results**

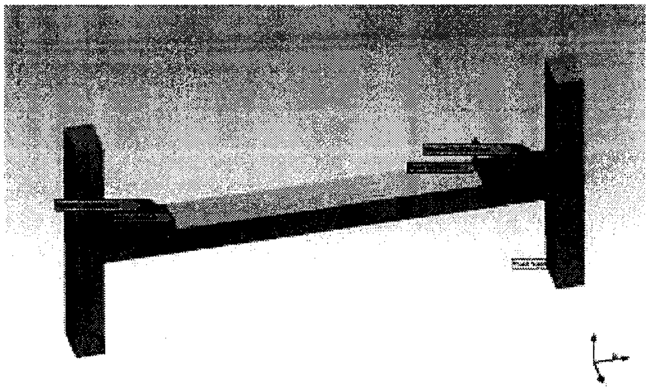
In this analysis ,six storied building is analysed and the exterior beam column joint in the first storey is designed as per IS:456-2000 and IS:13920.The elevation of the building is shown.The size of the beam is 230mmx380mm and the column is 230mmx380mm as per IS codes.As per IS:456-2000,the reinforcement details of beam is 5nos of 12mm dia bars with 3nos at bottom and 2nos at top with a stirrups spacing of 150mmc/c and the column consists of 6nos of 12mm dia bars with a stirrups spacing of 150mm c/c.



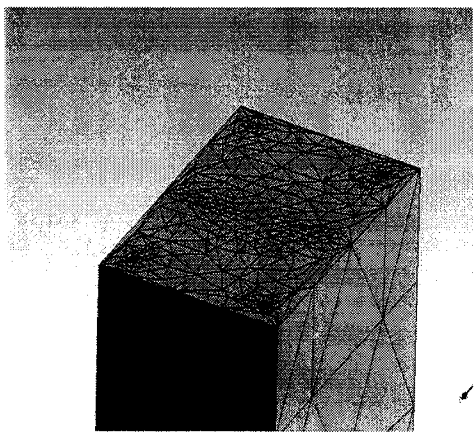
PLAN OF THE STRUCTURE

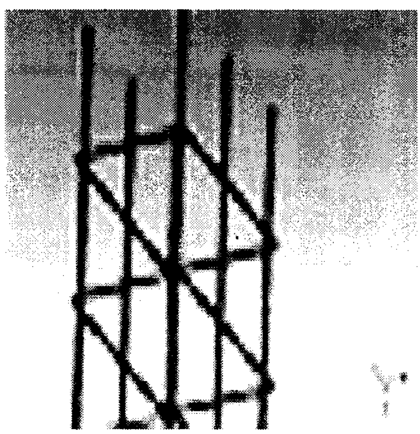
As per IS:13920,the reinforcement details of beam is 5nos of 12mm dia bars with 3nos at bottom and 2nos at top with a stirrups spacing of 100mmc/c and the column consists of 6nos of 12mm dia bars with a stirrups spacing of 100mm c/c.

In this three exterior beam column joint is considered for studying the behaviour of joint subjected to earthquake motion.One beam column joint is analysed and detailed as per IS:456-2000 and other two joint analysed as per IS:13920 for various detailing which can be shown below.

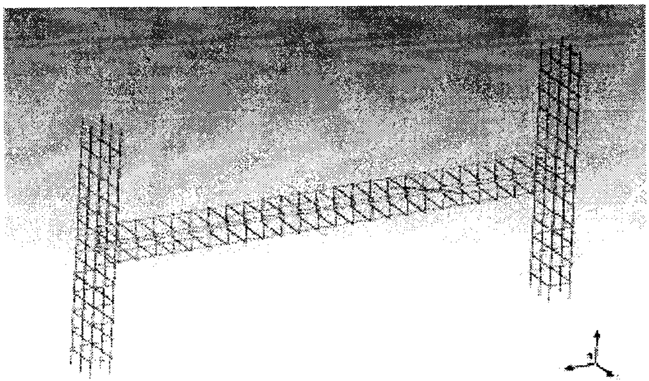


BOUNDARY CONDITION OF BEAM COLUMN JOINT SPECIMEN



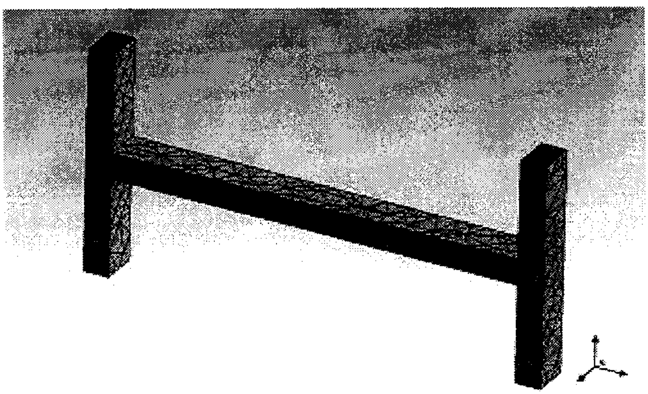


MESH GENERATION OF SPECIMEN & REINFORCEMENT

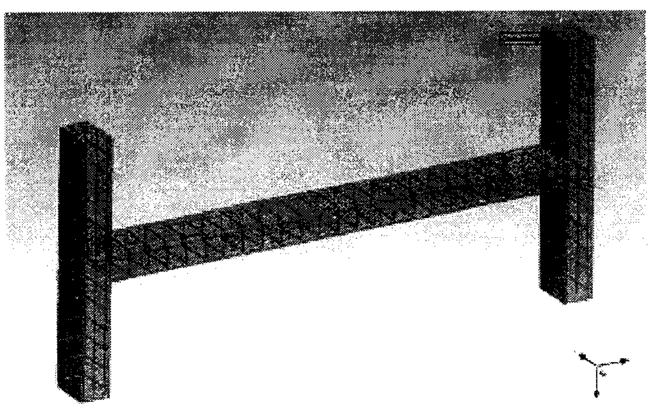


Beam-230mmx380mm
(Top-2#12mmbar &
Bottom-3#12mm bar)
Column-
230mmx380mm
(6#12mm bar)
Stirrupspacing

REINFORCEMENT DETAILS OF SPECIMEN-1(IS:456-2000)

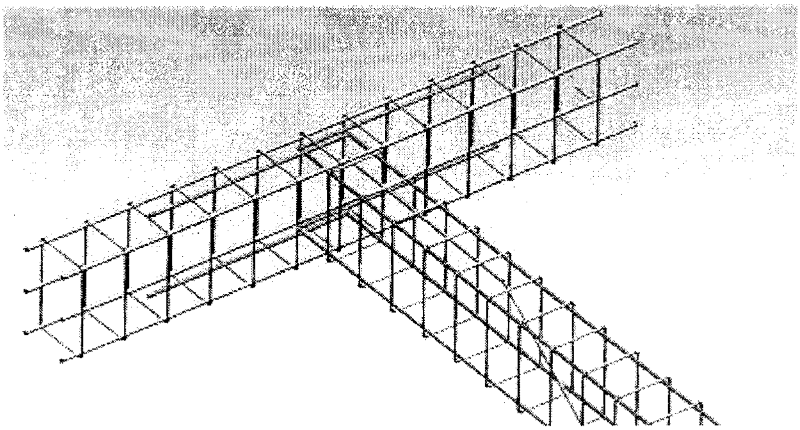


MESHING OF SOLID SPECIMEN-1



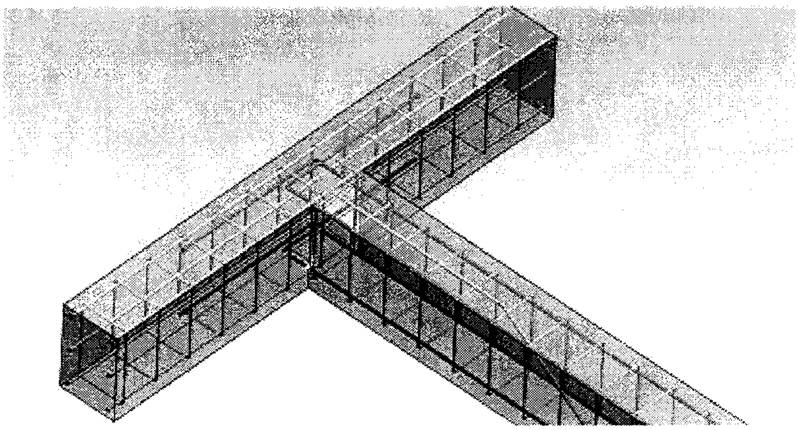
SOLID MODEL OF SPECIMEN-1

SPECIMEN DETAILS AS PER IS 456



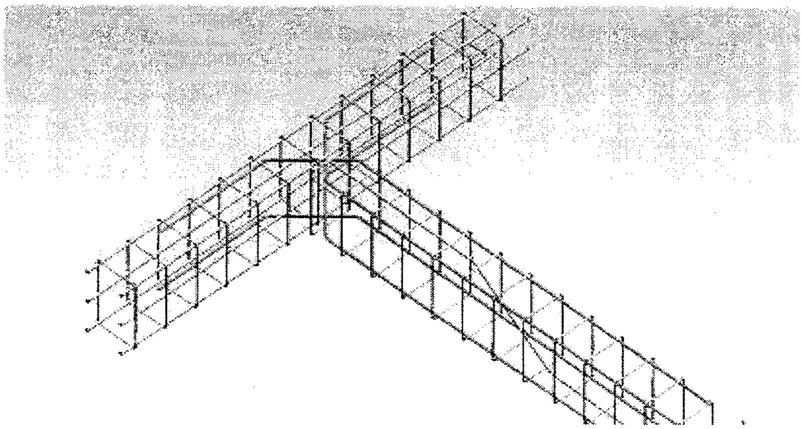
Beam-
230mmx380mm
(Top2#12mm bar &
Bottom3#12mmbar)
Column-
230mmx380mm
(6#12mmbar)
Stirrupspacing 100mn
c/c.

REINFORCEMENT DETAILS OF SPECIMEN -2(IS:13920)



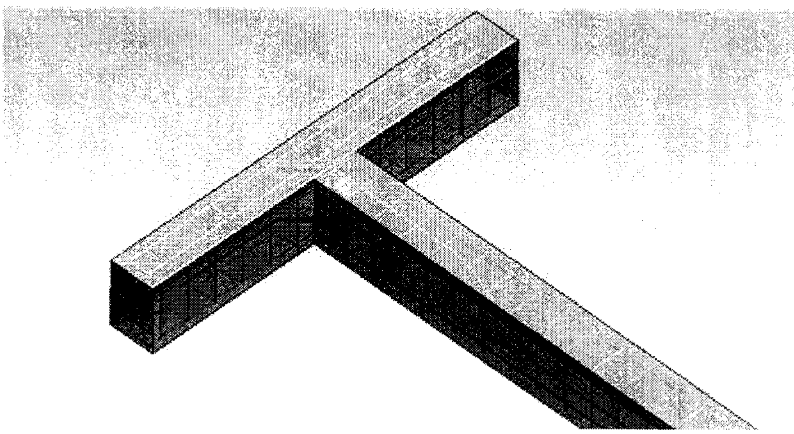
SOLID MODEL OF SPECIMEN -2

SPECIMEN DETAILS AS PER IS 13920



Beam-
230mmx380mm
(Top 2#12mm bar &
Bottom 3#12mm bar)
Column-
230mmx380mm
(6#12mmbar)
Stirrupspacing
100mmc/c.

REINFORCEMENT DETAILS OF SPECIMEN-3



SOLID MODEL OF SPECIMEN-3

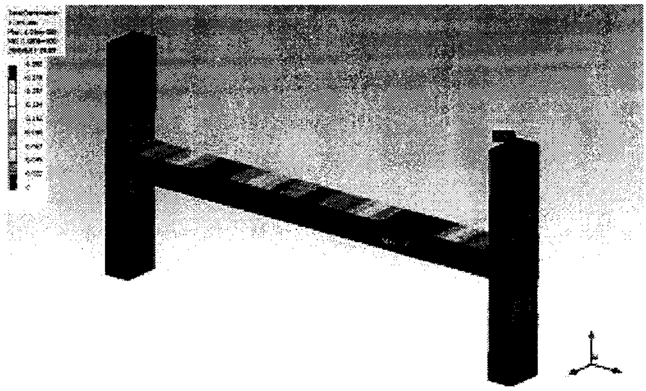
SPECIMEN DETAILS AS PER IS 13920

CHAPTER 6

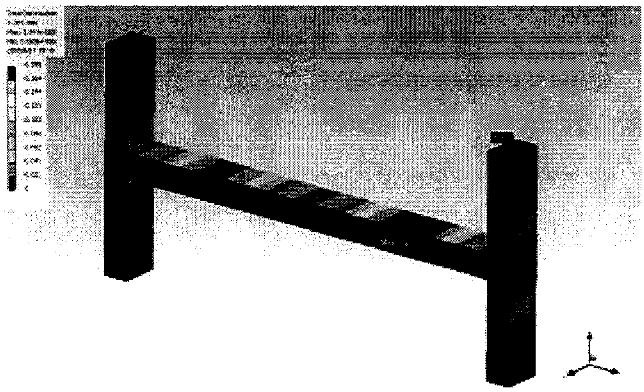
RESULTS

The results of the behaviour of exterior beam column joint as per IS:456-2000 and IS:13920 using ANSYS-10 software is shown.

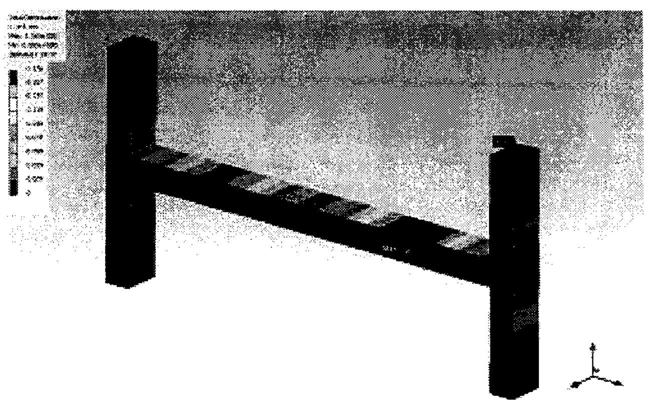
S.NO	SPECIMEN	DETAILS
1	As per IS:456-2000 (Stirrups should be placed equally)	Beam-230mmx380mm (Top-2#12mm & Bottom-3#12mm with stirrups spacing 150mm c/c) Column-230mmx380mm (6 nos of 12mm bar with stirrups spacing 150mm c/c)
2	As per IS:13920 (Stirrups should be placed closer in joint region over a distance of lo(600mm) and the anchorage length should be 50d(600mm),in beam the stirrups should be placed closer at a distance of lo(600mm)	Beam-230mmx380mm (Top-2#12mm & Bottom-3#12mm with stirrups spacing 100mm c/c) Column-230mmx380mm (6 nos of 12mm bar with stirrups spacing 100mm c/c)
3	As per IS:13920 (The bars in core region is inclined at 45 degree)	Beam-230mmx380mm (Top-2#12mm & Bottom-3#12mm with stirrups spacing 100mm c/c) Column-230mmx380mm (6 nos of 12mm bar with stirrups spacing 100mm c/c)



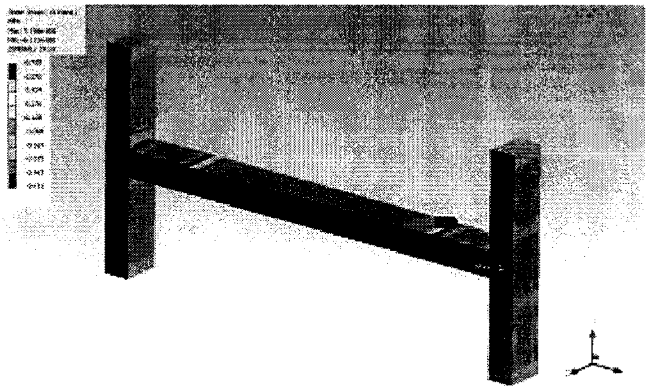
DEFORMATION OF SPECIMEN-1



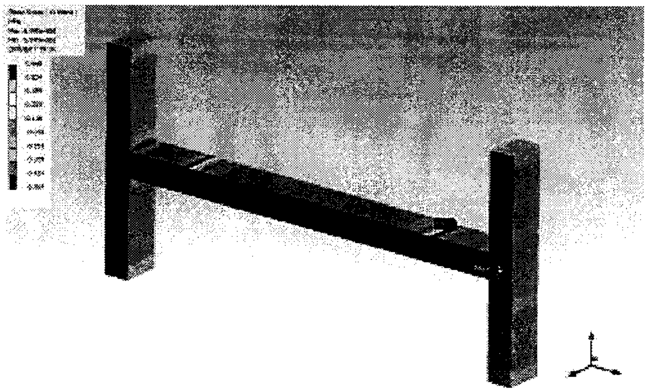
DEFORMATION OF SPECIMEN-2



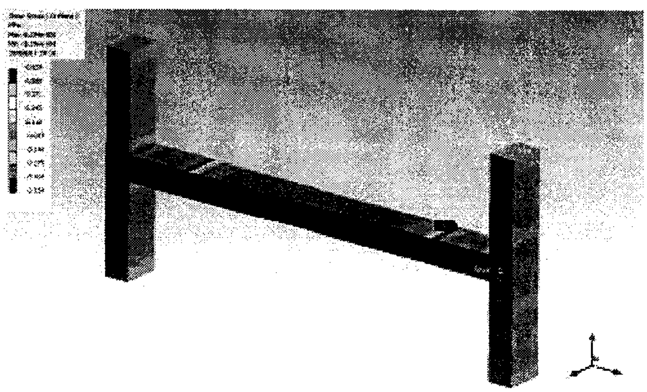
DEFORMATION OF SPECIMEN-3



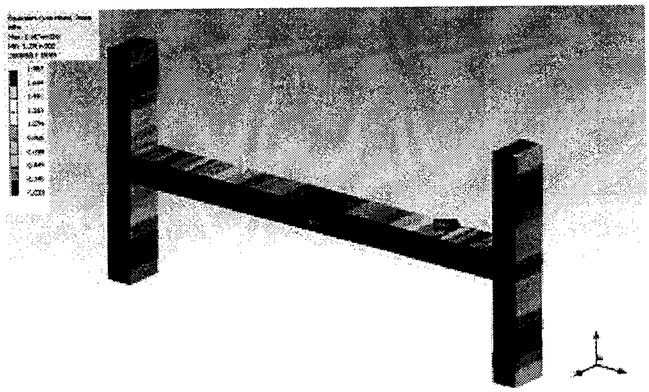
SHEAR STRESS OF SPECIMEN-1



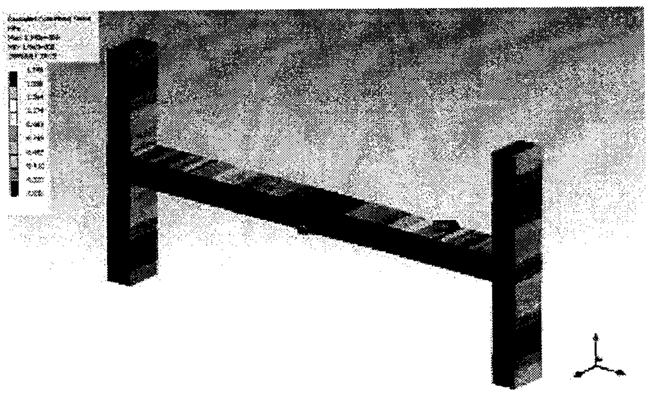
SHEAR STRESS OF SPECIMEN-2



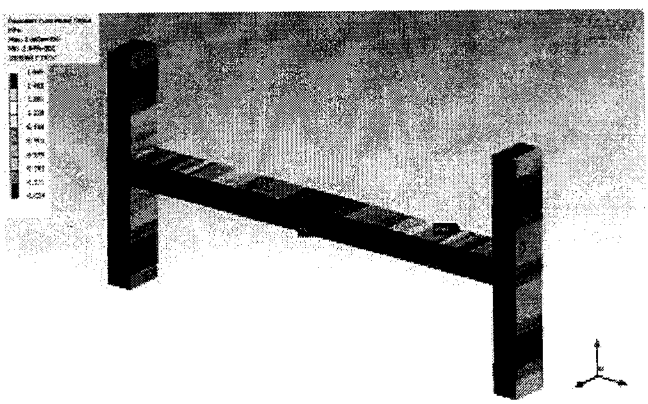
SHEAR STRESS OF SPECIMEN-3



STRESS CONTOURS OF SPECIMEN-1



STRESS CONTOURS OF SPECIMEN-2



STRESS CONTOURS OF SPECIMEN-3

COMPARISON OF RESULTS

S.NO	DEFORMATION In mm	SHEAR STRESS In N/mm^2	EQUIVALENT STRESS In N/mm^2
SPECIMEN-1	0.202	0.720	1.907
SPECIMEN-2	0.185	0.660	1.745
SPECIMEN-3	0.176	0.629	1.665

CHAPTER 7

CONCLUSION

Based on the analytical results of this thesis, the following conclusions are drawn for exterior beam- column joint connection subjected to reversed cyclic loading.

1. The joint shear stress appears to have a significant effect on strength and stiffness of subassembladge at lower ductility levels. The shear reinforcement in joint core portion can be increased some more percentage over the requirement as per IS code.
2. The shear reinforcement in beam – column interface should be provided for increasing the stiffness of the joint portion.
3. The shear reinforcement in flexural member can be increased up to 3d from the face of the column, so that plastic hinges formation can be shifted from the critical zone.
4. As per IS:456-2000, the shear stress in exterior beam column joint is maximum when compared to the beam column joint analysed as per IS:13920.If the reinforcement details can be altered as per IS:13920 in the joint core region,the shear stress would be reduced.
5. From that it is concluded that the spacing of stirrups very closer in the joint core region as per IS:13920 will reduce the shear stress,deformation.

SCOPE OF FUTURE WORK

As per IS:456-2000 and IS:13920-1993, the calculated shear stress is within the permissible limit. Based on the quantity takeoff further study can be made.

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CERTIFICATE

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has participated & presented a paper in the National Conference on "Innovative Materials & Methodologies for Disaster Resistant Structures (NCIMMDRS - 08)" sponsored by Defence Research & Development Organisation (DRDO), New Delhi and Coir Board (Govt. of India) on 10th & 11th April 2008.

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