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PARAMETRIC STUDY OF REINFORCED CONCRETE INTERIOR FRAMED JOINTS SUBJECTED TO SEISMIC LOADS

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ABSTRACT

The behaviour of reinforced concrete moment resisting frame structures in recent earthquakes all over the world has highlighted the consequences of poor performance of beam column joints. Beam column joints can be critical regions in reinforced concrete frames designed for inelastic response to severe seismic attack. As a consequence of seismic moments in columns of opposite signs immediately above and below the joint, the joint region is subjected to horizontal and vertical shear forces whose magnitude is typically many times higher than in the adjacent beams and columns. This paper presents design and detailing procedure of interior joints. A ground plus five story building is analyzed and designed using ETABS software. Analysis is also done by using kanis method for gravity loads and factor method for lateral loads for software validation. Design and detailing is carried out for different well established codes viz., Indian (IS13920 draft revision), American (ACI318-2002 and ACI352R-02), and Euro code. The parameters which are considered in this study are concrete compressive strength, steel yield strength, and column B/D ratios. Significant parameters influences the design of beam-column joints are identified and the effect of their variation on design is compared.

Introduction

Earthquakes are one of the most feared natural phenomena that are relatively unexpected. Their impact is sudden due to the almost instantaneous destruction that a major earthquake can produce. Beam column joints can be critical regions in reinforced concrete frames designed for an inelastic response to a severe seismic attack. As a consequence of seismic moments in columns of opposite signs immediately above and below the joint, the joint region is subjected to horizontal and vertical shear forces whose magnitude is typically many times higher than in the adjacent beams and columns. If the joint is not designed for these forces, it could result in a joint shear failure. The reversal in the moments across the joint also means that the beam reinforcement is required to be in compression on one side of the joint and at tensile yield on the other side of the joint. The high bond stress required to sustain this force gradient across the joint may cause a bond failure and the corresponding degradation of moment capacity accompanied by an excessive drift.

In the analysis of reinforced concrete moment resisting frames the joints are generally assumed as rigid.

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In Indian practice, the joint is usually neglected for specific design with attention being restricted to the 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 which occurred in 1999, shown in fig. 1 which has 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.

Figure 1. Total pancake collapse of a building because of inadequate design at the beam-column joints during the Kocaeli earthquake, Turkey, August 17, 1999, Magnitude 7.4.

Present Work

A Ground plus four storey building in zone V has been analysed with the aid of computer software ETABS version – 8.4.8 (2004). In zone-V the building is designed in three cases by varying the column B/D ratios and concrete compressive strength and the amount of joint shear in three different locations viz. interior, exterior, and corner are compared. This building form a representative group of medium rise buildings which of lately have been included to be designed as only SMRF. This building does not have any shear walls. Seismic forces in building have been obtained using the equivalent static method. Ductility provisions of IS: 13920-1993 and IS: 13920 (Draft) will mainly influence design of columns, beams and joints. Hence the quantity of reinforcement in slabs, foundations and other structural and non-structural elements has not been considered.

Modeling of building frame

A G+4 storey building having panel aspect ratio 1.25 for first two bays and 1.67 for middle bay is analysed and designed for seismic forces in Zone V as SMRF respectively using ETABS version-8.4.8 (2004). The plan and sectional elevation of the building is shown in the figure. The schedule of the member sizes of the frame is as shown in Table 2.

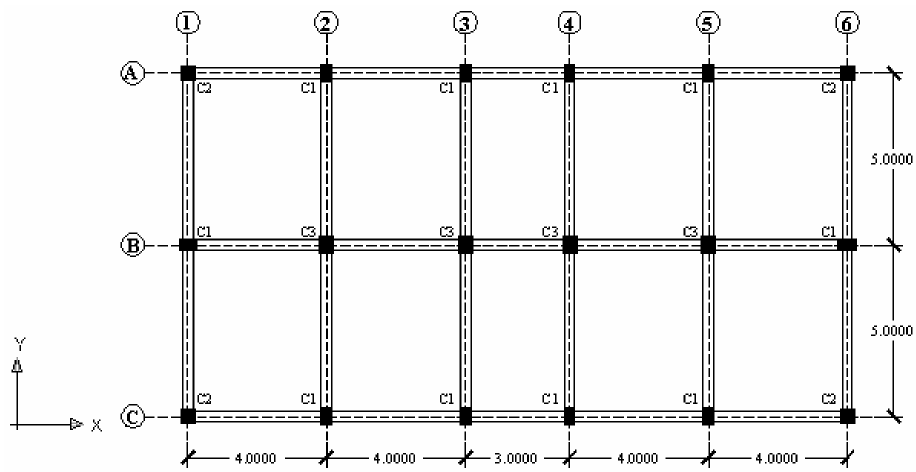


Figure 2. Plan of building (All dimensions are in meters).

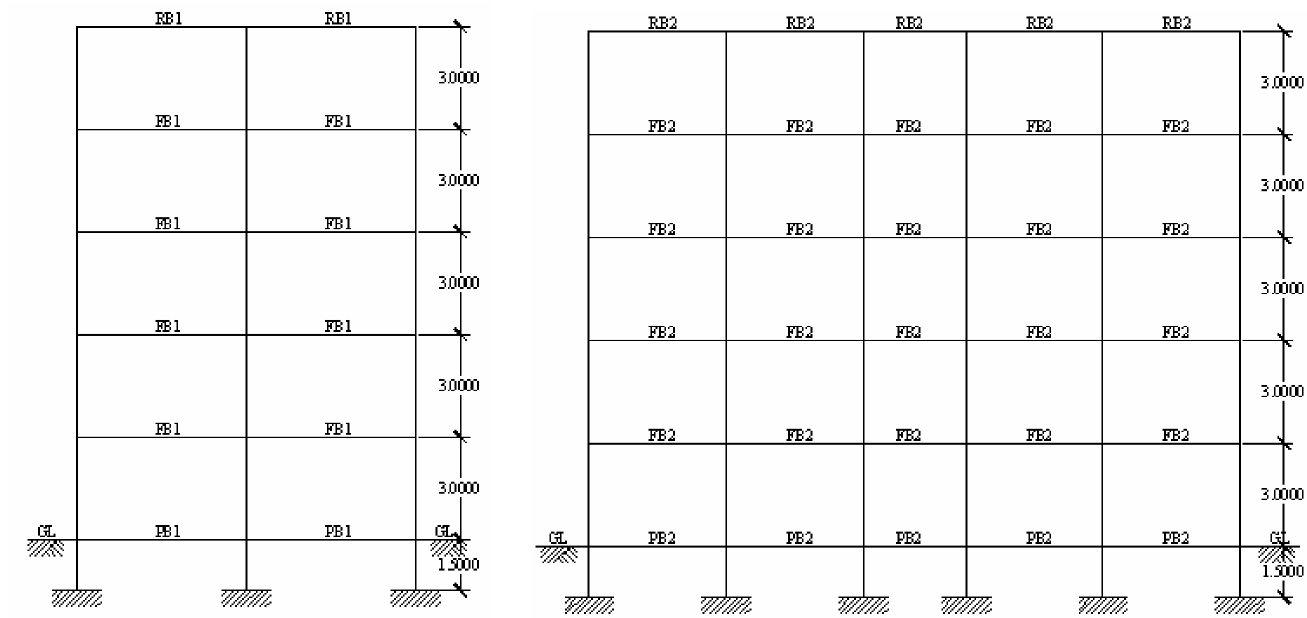


Figure. 3. Sectional elevation along X and Y-axis (All dimensions are in meters).

Table. 1. Schedule of member sizes (Case-1, Case-2 and Case-3).

Beam Dimensions		Column Dimensions			
RB1,FB1	300 X 600		Case 1	Case 2	Case 3
RB2,FB2	300 X 600	C1	300 X 500	500 X 500	500 X 500
PB1	300 X 400	C2	400 X 400	500 X 500	400 X 500
PB2	300 X 350	C3	400 X 500	500 X 600	600 X 600
Slab thickness : 125 mm		Note: All dimensions are in mm			

General data:

Grade of concrete : M20, M25, M30
Grade of steel : Fe 250, Fe 415
Live load on roof : 1.5 KN/m^2 (Nil for earthquake)
Live load on floors : 3.0 KN/m^2 (25 % for earthquake)

Roof finish	: 1.0 KN/m^2
Floor finish	: 1.0 KN/m^2
Brick wall on peripheral beams	: 230 mm thick
Brick wall on Internal beams	: 150 mm thick
Density of concrete	: 25 KN/m^3
Density of brick wall Including plaster	: 20 KN/m^3

Design of beams and columns has been done by limit state method taking analysis results from ETABS version – 8.4.8 (2004). Shear reinforcement in beams is obtained for design shear force at supports and at centre. For detailing main reinforcement in beams, the available diameters of steel ranging from 12 to 20 mm has been used and a set of arrangement is provided that is closest to theoretical area of steel required. If two members are on either side of a column and are continuous in alignment, the same reinforcement is provided on both sides of the column by picking up the higher steel area. The minimum and maximum reinforcement requirements are also checked.

In addition to detailing of beams as in above, in the design of beams in SMRF it is also ensured that the positive steel at support is at least half the negative steel provided at that support or joint. The steel provided at each of the top and bottom face of the member at any section along its length is more than one fourth of the maximum negative moment steel provided at the face of either joint. All other requirements are checked as per clauses given in IS: 13920-1993. In the design of columns, for design axial forces and biaxial moments generated using ETABS version – 8.4.8 (2004), and steel area obtained, a steel arrangement corresponding to the required steel area is provided equally distributed on all four sides of the sections. In addition to design and detailing of main reinforcement in columns, confinement steel as required by IS: 13920-1993 is calculated and provided the same over the required length at ends of column in SMRF. This enabled a fairly comprehensive and reliable estimation of reinforcement quantities. The design results are provided in a tabular format. The ductile design of joint are shown for illustration.

Interior joint design

The details of the column and beam reinforcement meeting at the joint are shown in Figure 5. The transverse beam of size 300 x 600 is reinforced with 6-20Ø + 3-16Ø (2489.13 mm^2, i.e., 1.5596%) at top and 3-20Ø + 4-16Ø (1747.429 mm^2, i.e., 1.0948%) at bottom. The hogging and sagging moment capacity is evaluated as 391.422 KN-m and 321.638 KN-m, respectively.

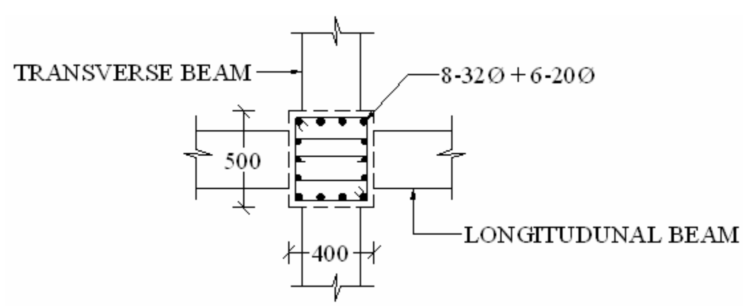


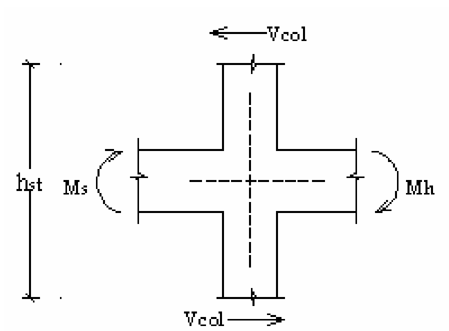
Figure 4. Reinforcement details for column and beams.

The longitudinal beam of size 300 x 600 mm is reinforced with 4-20Ø + 2-16Ø + 5-12Ø (2225 mm^2, i.e. 1.394%) at top and 3-20Ø + 3-16Ø + 1-12Ø (1659.429 mm2 i.e. 1.0397%) at bottom. The hogging and sagging moment capacity is evaluated as 348.119 KN-m and 305.138 KN-m, respectively.

Check for Earthquake in X-Direction

Column Shear

The column shear is as explained below for sway to right and left conditions respectively.



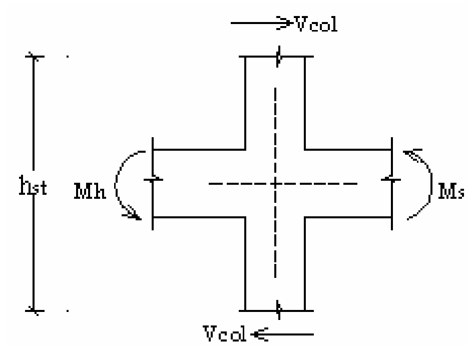


Figure 5a. Column with sway to right. Figure 5b. Column with sway to left.

For both the above case, the column shear is calculated from the given Eq. 1

$$V_{col} = 1.4\left(\frac{M_s + M_h}{h_{st}}\right) \tag{1}$$

The development of forces in the joint due to beam reinforcement, for sway to right and left, are calculated from the given Eq. 2 and 3.

Force developed in the top bars

$$T_1 = A_{st1} \times 1.25 \times f_y = C_1 \tag{2}$$

The factor 1.25 is to account for the actual ultimate strength being higher than the actual yield strength. [Draft revision of IS 13920]

Force developed in the bottom bars

$$T_2 = A_{st2} \times 1.25 \times f_y = C_2 \tag{3}$$

Joint Shear

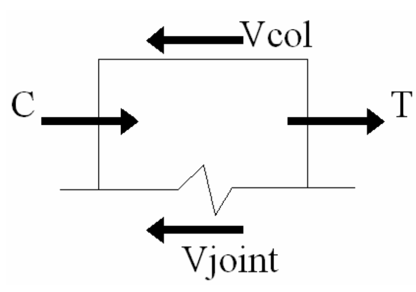


Figure 6. Joint shear.

The forces acting on the joint region are shown in the fig 6. By considering Equilibrium of forces acting on the joint. The joint shear is calculated from the Eq. 4

$$V_{Joint} = T + C - V_{col} \tag{4}$$

Where V_{Joint} is Joint shear, T is Tension in beam bar, C is Compression in beam bars, V_{col} is column shear. Maximum value of $T_1$and minimum value of V_{col} is used in the above equation.

Check for Joint Shear Strength

The effective width provisions for joints are shown in the following figure 7. The calculation of the effective width of the joint is shown in the following table 2.

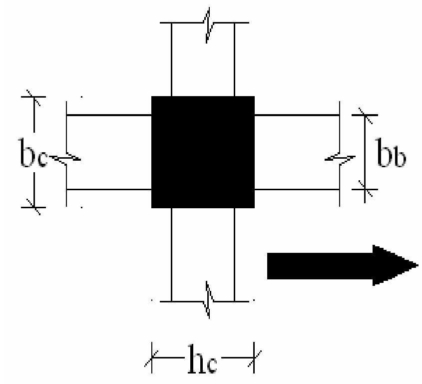


Figure. 7. Effective width for joint.

Table 2. Effective joint width b_j.

S.No	Category	IS 13920 Draft	ACI352R-02	Eurocode-8
1	$b_c > b_b$	minimum of i) ($b_j = b_c$) ii) ($b_b + 0.5\ h_c$)	minimum of i) ($b_b + b_c$)/2 ii) ($b_b + 0.5\ h_c$) iii)) ($b_j = b_c$)	minimum of i) ($b_j = b_c$) ii) ($b_b + 0.5\ h_c$)
2	$b_c < b_b$	minimum of i) ($b_j = b_b$) ii) ($b_c + 0.5\ h_c$)	$b_j = b_c$	minimum of i) ($b_j = b_b$) ii) ($b_c + 0.5\ h_c$)

The effective joint depth is taken as the total depth of the column in the direction of seismic force considered. Effective shear area of the joint is calculated from Eq. 5

$$A_{ej} = bj\ h_c \tag{5}$$

Nominal joint shear strength

As per IS 13920 Draft, nominal shear strength of the joint as a function of only concrete compressive strength, which in turn depends upon the degree of confinement, offered by the members and is given as, $1.5\sqrt{f_{ck}}A_{ej}$ if confined on four faces, $1.2\sqrt{f_{ck}}A_{ej}$ if confined on three faces and $1.0\sqrt{f_{ck}}A_{ej}$ for other cases. Apart from this, the code requires a minimum amount of transverse reinforcement in the joint as shear reinforcement and to provide for confinement of core concrete.

ACI 318M-02 sets the nominal shear strength of the joint as a function of only concrete strength, which in turn depends upon the degree of confinement, offered by the members and is given as, $1.7\sqrt{f_{ck}}A_{ej}$ if confined on four faces, $1.25\sqrt{f_{ck}}A_{ej}$ if confined on three faces and $1.0\sqrt{f_{ck}}A_{ej}$ for other cases. Apart from this, the code requires a minimum amount of transverse reinforcement in the joint as shear reinforcement and to provide for confinement of core concrete.

Euro code 8 also has limited the nominal shear within interior beam column joint to be less than the value given by the expression $20\tau_{Rd} A_{ej}$ and for exterior joints it is $15\tau_{Rd} A_{ej}$.

The joint shear force obtained by the beam bars, is checked against the nominal joint shear strength calculated according to the formula given in three different well established codes as discussed in the previous section. Finally the joint is checked against the strong column-weak beam condition, by calculating the hogging and sagging moment capacities of the beams meeting at the joint and the moment capacities of the columns meeting at the same joint. The flexural strength ratio can be calculated from the followig Eq. 6

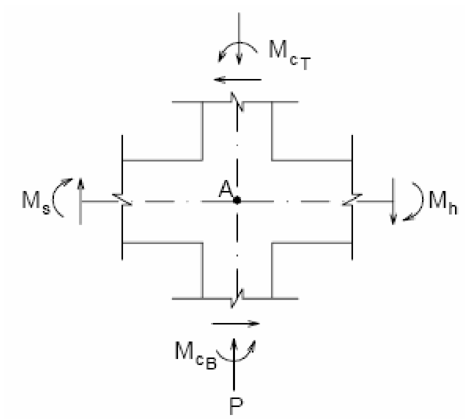


Figure. 8. Check for strong column – weak beam condition.

$$\text{Flexural strength ratio} = \frac{\sum M_c}{\sum M_b} \tag{6}$$

Results and Discussion

As can be seen from the checks the joint is not safe against shear. Joint region requires higher grades of concrete. Higher grades of concrete is undesirable, in such cases the following three alternatives can be tried.
i) Increase the column section so that the joint area is increased. This will also reduce the main longitudinal steel requirement in the column owing to larger column size.
ii) Increase the size of the beam section. If this option is adopted, it is advisable to increase the depth of the beam. This will reduce the steel required in the beam and hence will reduce the joint shear. In case of depth restriction in the beam, increase in beam width can be considered if the difference between the shear strength of joint and joint shear is small.
iii) Increase the grade of concrete. This option will increase the shear strength of joint and also reduce the steel required in columns.
From the above three options, column dimensions and concrete compressive strength are considered for the re-analysis. The reanalysis and design is performed for three sets of column dimensions for different concrete compressive strengths. The results obtained in the three cases are shown in the following tables and figures.

Conclusions

- For non-seismic loads column shear alone is acting as the joint shear, whereas in seismic conditions contribution of beam forces developed by seismic loads is also considered in the joint shear calculation.

- For **medium rise** buildings in **Zone-V**, joint shear is **18 to 20** times more for seismic loads than in non-seismic loads.
- The joint shear strength can be increased with increase in concrete compressive strength, and **column B/D ratio**. However, there is no increase in joint shear strength observed, while changing the **beam B/D ratio**.
- In seismic zones V, the minimum grade of concrete is M30. M25, can be used for beams and columns with rich mix concrete in joint regions.
- In all cases roof joints are safe against joint shear but the column reinforcement should be provided in the roof column to satisfy the strong column weak beam condition.
- In an exterior joint and a corner joint the depth of the column should be provided to satisfy the anchorage requirements of the beam longitudinal bar.
- Joint shear in floor joints is 3 to 5 times more than the roof joints.

Appendix: Nomenclature

SMRF	Special Moment Resisting Frame
ETABS	Extended Three Dimensional Analysis of Building Systems
M_s	Sagging Moment
M_h	Hogging moment
h_{st}	Height of the column
T_1	Tension in the top bars of the beam
T_2	Tension in the bottom bars of the beam
C_1	Compression in the top bars of the beam
C_2	Compression in the bottom bars of the beam
A_{st1}	Area of the top reinforcement on left side of the joint
A_{st2}	Area of the top reinforcement on right side of the joint
f_y	Yield strength of the steel reinforcement
b_b	Breadth of the beam
b_c	Breadth of the column
b_j	effective breadth of the joint
h_c	Depth of the column
f_{ck}	Compressive strength of the concrete
A_{ej}	Effective area of the joint
$\sum M_c$	Sum of the column moments meeting at a joint
$\sum M_b$	Sum of the beam moments meeting at a joint

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Table 3. Joint shear details at Roof level for case-1 (Zone-V).

fy	**415**	**N/mm^2**	Dimensions	B (mm)	D (mm)	Column	B (mm)	D (mm)
			BEAM X	300	600	Interior	400	500
	Zone-V		BEAM Y	300	600	Exterior	300	500
			CASE-1			Corner	400	400
Joint	**Concrete compressive strength N/mm^2**	**Location**	**Direction**	**Joint shear strength KN**	**Joint shear KN**	**Result**	**Strong column Weak beam condition**	**Confining links**
INTERIOR								
1	20	ROOF	Y	1073.31	492.70	SAFE	SATISFIED	10# 100 C/C
			X	894.43	440.95	SAFE	SATISFIED	10# 100 C/C
2	25	ROOF	Y	1200.00	465.31	SAFE	SATISFIED	10# 80 C/C
			X	1000.00	423.74	SAFE	SATISFIED	10# 80 C/C
3	30	ROOF	Y	1314.53	503.32	SAFE	SATISFIED	10# 75 C/C
			X	1095.44	462.45	SAFE	SATISFIED	10# 75 C/C
EXTERIOR								
4	20	ROOF	Y	670.82	283.65	SAFE	SATISFIED	10# 80 C/C
			X	603.74	468.93	SAFE	NOT SATISFIED	10# 80 C/C
5	25	ROOF	Y	750.00	280.80	SAFE	SATISFIED	10# 75 C/C
			X	675.00	467.72	SAFE	NOT SATISFIED	10# 75 C/C
6	30	ROOF	Y	821.58	222.11	SAFE	SATISFIED	10# 75 C/C
			X	739.42	467.72	SAFE	NOT SATISFIED	10# 75 C/C
CORNER								
7	20	ROOF	Y	715.54	198.37	SAFE	SATISFIED	10# 100 C/C
			X	715.54	185.76	SAFE	SATISFIED	10# 100 C/C
8	25	ROOF	Y	800.00	202.33	SAFE	SATISFIED	10# 85 C/C
			X	800.00	185.21	SAFE	SATISFIED	10# 85 C/C
9	30	ROOF	Y	876.35	234.11	SAFE	SATISFIED	10# 75 C/C
			X	876.35	230.15	SAFE	SATISFIED	10# 75 C/C

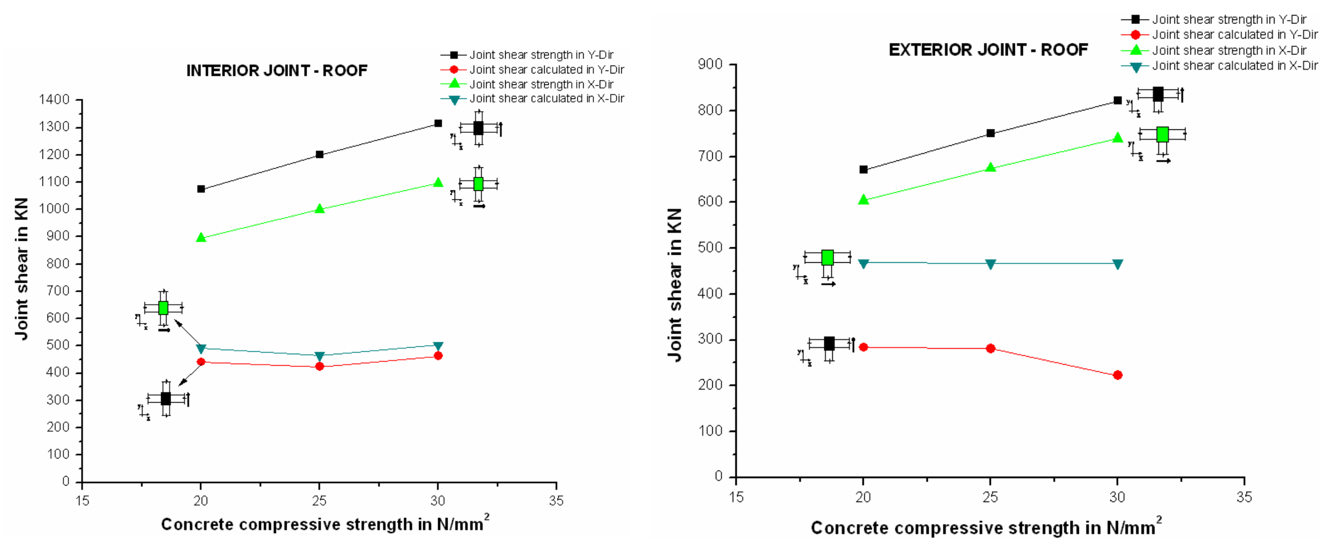


Figure 9. Joint shear due to seismic forces for Interior and Exterior joints - Roof (Zone-V, Case-1).

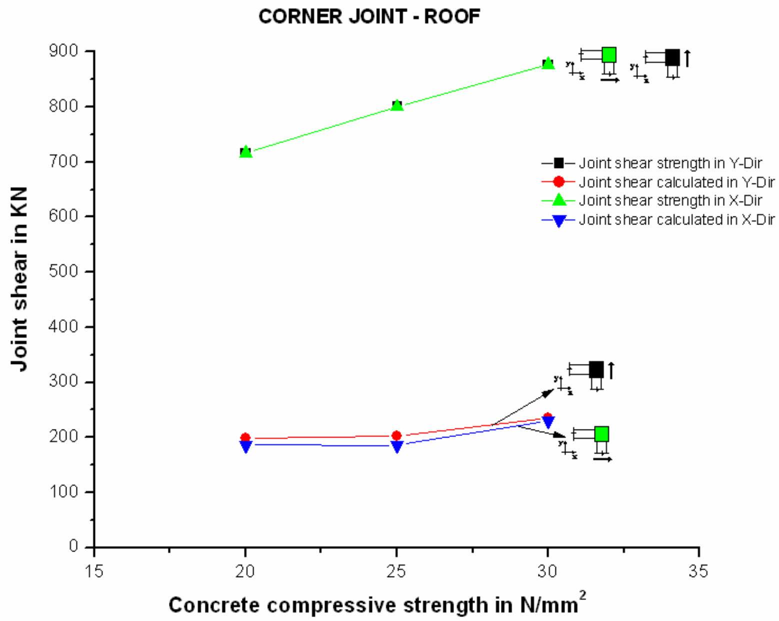
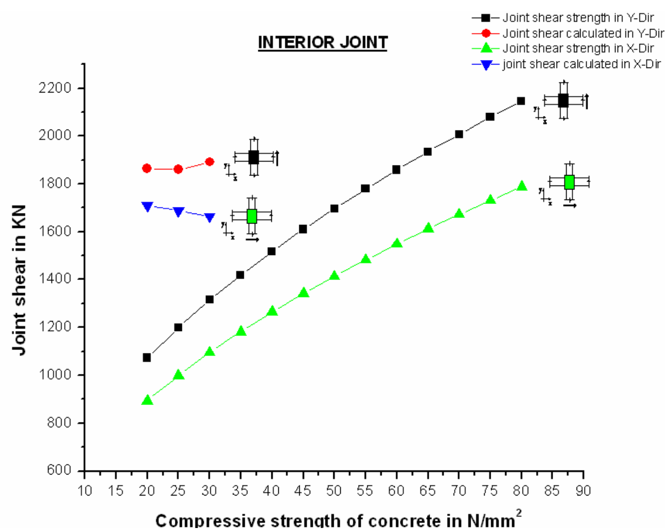


Figure 10. Joint shear due to seismic forces for Corner joint - Roof (Zone-V, Case-1).



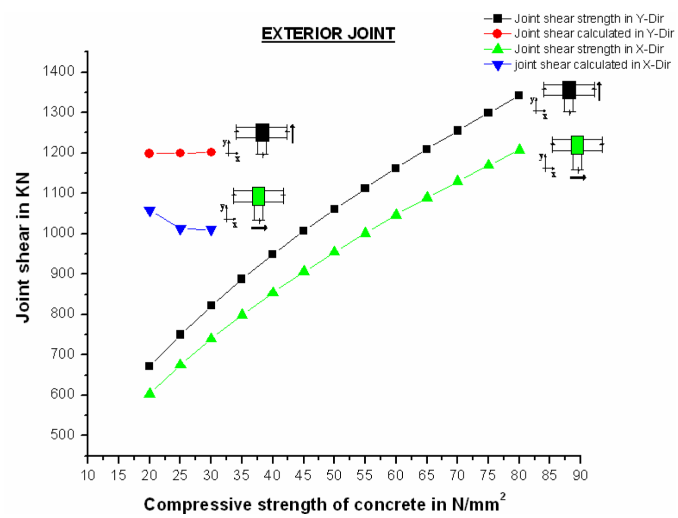


Figure 11. Joint shear due to seismic forces for Interior and Exterior joints (Zone-V, Case-1).

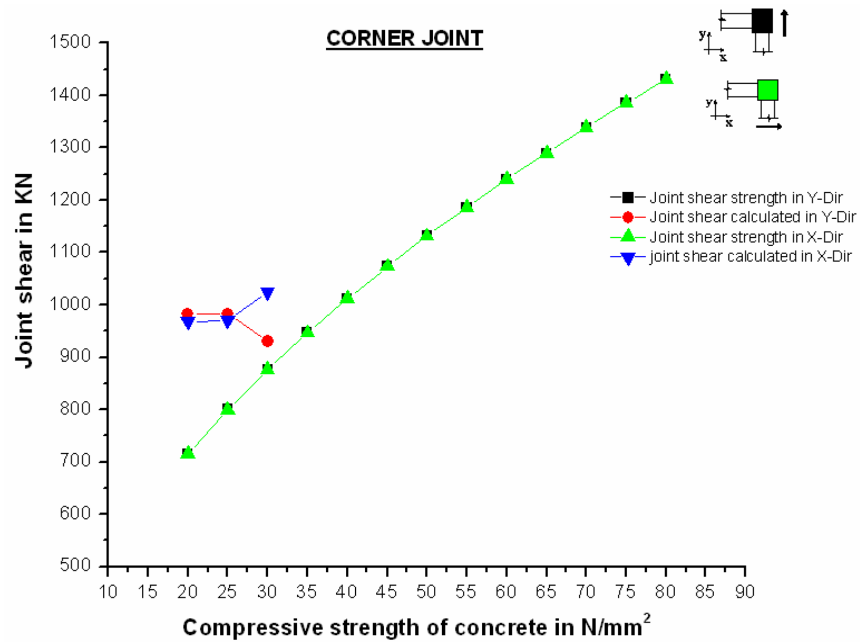


Figure 12. Joint shear due to seismic forces for Corner joint (Zone-V, Case-1).

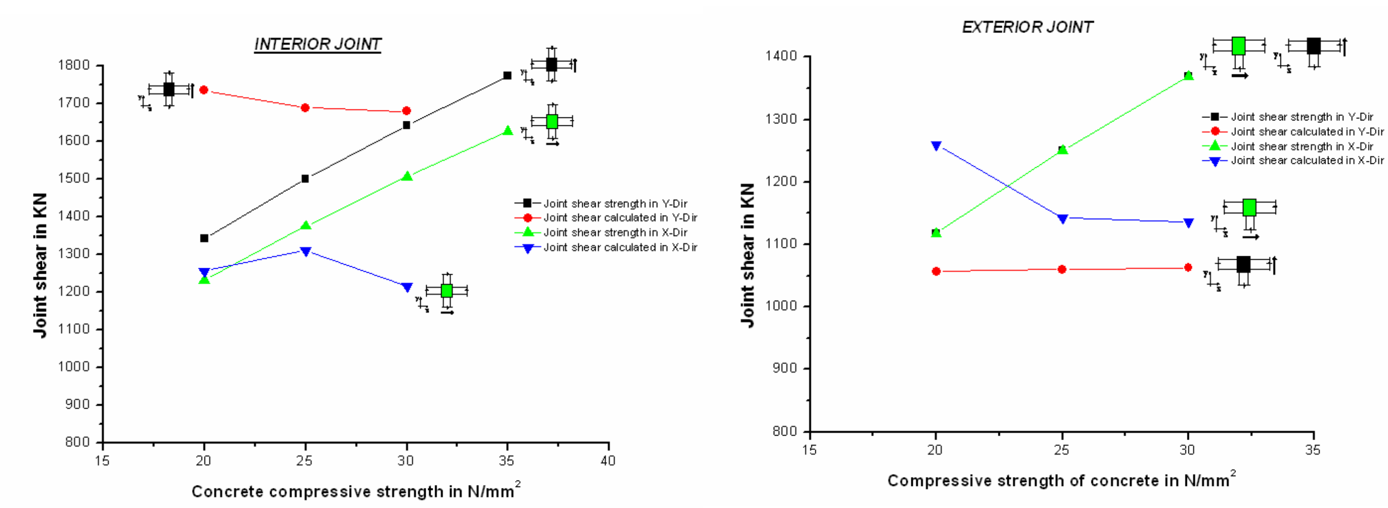


Figure. 13. Joint shear due to seismic forces for Interior and Exterior joints (Zone-V, Case-2).

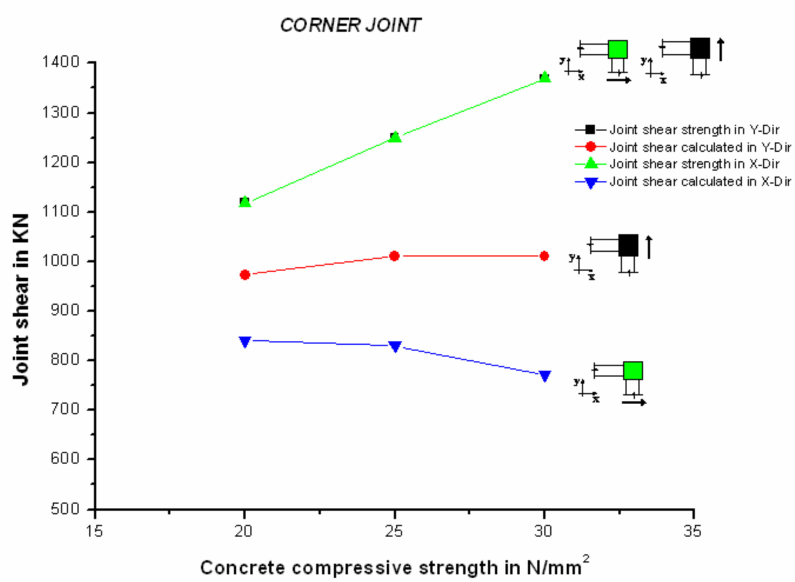


Figure 14. Joint shear due to seismic forces for Corner joint (Zone-V, Case-2).

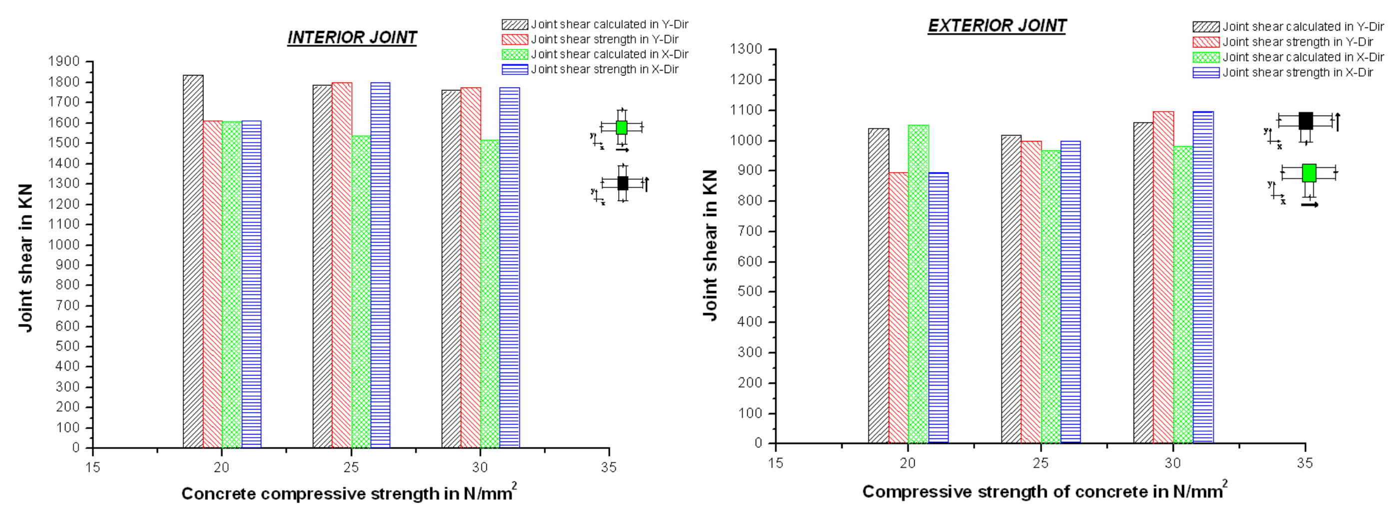


Figure 15. Joint shear due to seismic forces for Interior and Exterior joint (Zone-V, Case-3).

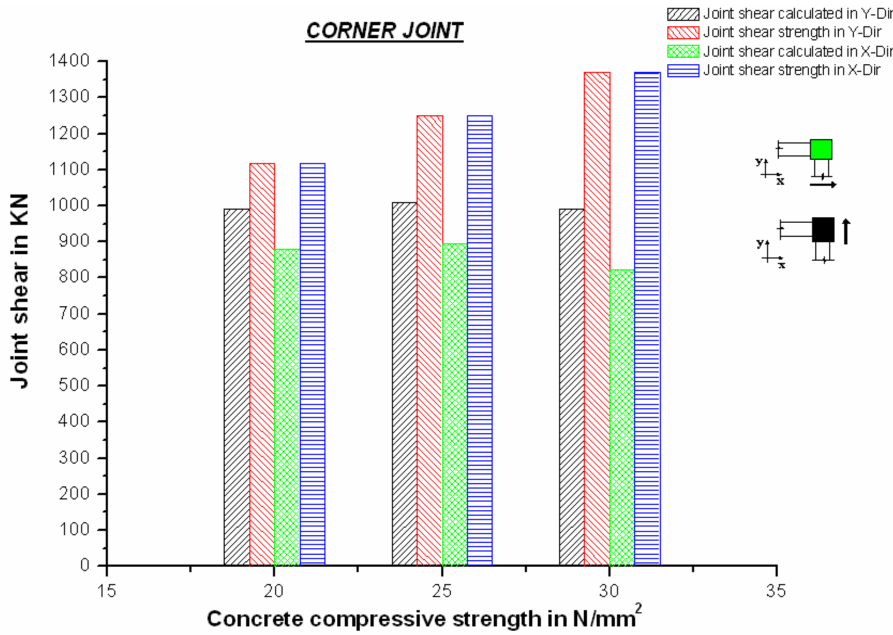


Figure 16. Joint shear due to seismic forces for Corner joint (Zone-V, Case-3).