Proposed codal provisions for design and detailing of beam-column joints in seismic regions

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Beam-column joint is an important part of a reinforced concrete moment resisting frame subjected to earthquake loading. Design and detailing provisions on beam-column joints in IS 13920 : 1993 do not adequately address prevention of anchorage and shear failure in this region during severe earthquake shaking. In view of these limitations, this paper proposes new provisions for inclusion in IS 13920 : 1993. The paper also gives a clause-by-clause commentary on these recommended provisions and includes one solved example to illustrate the same.

Keywords: *Beam-column joints, wide beam, strong-column weak-beam, shear design.*

Beam-column joint is an important component of a reinforced concrete moment resisting frame and should be designed and detailed properly, especially when the frame is subjected to earthquake loading. Failure of beam-column joints during earthquakes is governed by bond and shear failure mechanism which are brittle in nature[1]. Therefore, current international codes give high importance to provide adequate anchorage to longitudinal bars and confinement of core concrete in resisting shear[2]. A review of the behaviour and design of different types of beam-column joints in reinforced concrete moment resisting frame under seismic loading illustrates that design and detailing provisions for the joints in the current Indian seismic code, IS 13920 : 1993 are not adequate to ensure prevention of such brittle failure[3,4,5]. Since joints are subjected to large shear force during earthquake, shear strength in this region should be adequate to carry this large amount of shear force. Therefore, the current code needs to be upgraded to incorporate shear design provisions of beam-column joints. Moreover, under cyclic lateral loading, longitudinal beam bars are subjected to pull out force and must be provided with sufficient anchorage length within the joint region. For an interior joint this anchorage length can only be provided through adequate column width and depth. Therefore, the code must have a provision for minimum dimension of column. The current code should also include confinement provisions on connection between columns and wide-beams, which are often found in one-way concrete joist systems and in buildings where floor-to-ceiling heights are restricted. This paper presents suggested provisions on beam-column joints for inclusion in IS 13920 : 1993[3]. These has been developed in line with ACI 318M[6]. The application of the proposed provisions has been illustrated by a solved example for design of an interior joint.

Proposed provisions for beam-column joints

Minimum column size

Clause 1.0

The minimum dimension of column shall not be less than (a) 15 times the largest beam bar diameter of the longitudinal reinforcement in the beam passing through or anchoring into the column joint, and (b) 300 mm.

Commentary 1.0

A small column width may lead to following two problems : *(a)* the moment capacity of column section is very low since the lever arm between the compression steel and tension steel is very small, and *(b)* beam bars do not get enough anchorage in the column (both at exterior and interior joints).

Hence, many seismic codes recommend that the dimension of an interior column should not be less than 20 times the diameter of largest beam bar running parallel to

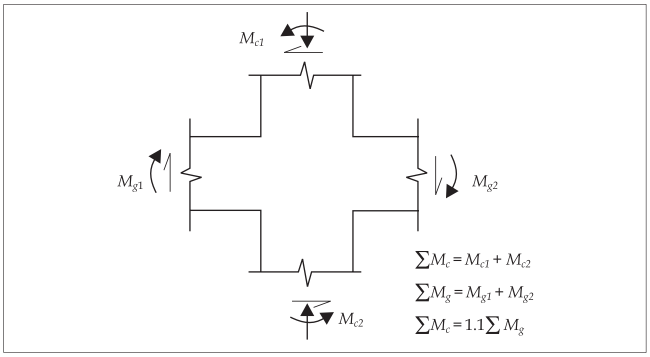

Fig 1 Strong-column-weak-beam concept

that column dimension that is, if beams use 20 mm diameter bars, minimum column width should be 400 mm. The proposed provision for minimum column size has been kept lower than the current international codes keeping in mind the practice in India where much smaller column sections are currently being used than what is common in other seismic countries like USA and New Zealand.

The existing clause no. 7.1.2 of IS 13920 : 1993 specifies the minimum dimension of columns as, "The minimum dimension of the member shall not be less than 200 mm. However, in frames which have beams with centre to centre span exceeding 5 m or columns of unsupported length exceeding 4 m, the shortest dimension of the column shall not be less that 300 mm". It is proposed to revise this clause as per clause 1.0 of this paper .

Longitudinal reinforcement

Clause 1.1

At a joint in a frame, resisting earthquake forces, the sum of the moment of resistance of the columns shall be at least 1.1 times the sum of the moment of resistance of the beams along each principal plane of the joint as shown in *Fig* 1. The moment of resistance of the column shall be calculated considering the factored axial forces on the column and it should be summed such that the column moments oppose the beam moments. This requirement shall satisfy for beam moments acting in both directions in the principal plane of the joint considered. Columns not satisfying this requirement shall have special confining reinforcement over their full height instead of the critical end regions only.

Commentary 1.1

This clause is based on strong-column-weak-beam theory. It is meant to make the building fail in beam-hinge mechanism (beams yield before the columns do) and not in the storey mechanism (columns yield before the beams). Storey mechanism must be avoided as it causes greater damage to the building. Therefore, column should be stronger than the beams meeting at a joint. ACI 318M requires the sum of the moment of resistance of the columns to be at least 20 percent more than the sum of the moment of resistance of the beams[6]. NZS 3101 : 1995 recommends that the sum of the design flexural strength of columns is at least 40 percent in excess of the overstrength of adjacent beams meeting at the joint[7].

Transverse reinforcement

Clause 1.2.1

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 clause 1.2.3.

Commentary 1.2.1

Quite often joints are not provided with stirrups because of construction difficulties. Similarly, in traditional constructions the bottom beam bars are often not continuous through the joint. Both these practices are not acceptable when the building has to carry lateral loads.

Following are the main concerns regarding joints:

- Serviceability – Diagonal tension cracks should not occur due to joint shear.
- Strength – Should be more than that in the adjacent members.
- Ductility – Not needed for gravity loads, but needed for seismic loads.
- Anchorage – Joint should be able to provide proper anchorage to the longitudinal bars of the beams.
- Ease of construction – Joint should not be congested.

Clause 1.2.2

For a joint, which is confined by structural members as specified by clause 1.2.3, transverse reinforcement equal to at least half the special confining reinforcement required at the end of the column shall be provided within the depth of the shallowest framing member. The spacing of the hoops shall not exceed 150 mm.

Commentary 1.2.2

Transverse reinforcement can be reduced as per 1.2.2 if structural members frame into all four sides of the joints.

Clause 1.2.3

A member that frames into a face is considered to provide confinement to the joint if at least three-quarters of the face of the joint is covered by the framing member. A joint is considered to be confined if such confining members frame into all faces of the joint.

Commentary 1.2.3

A joint can be confined by the beams/slabs around the joint, longitudinal bars (from beams and columns, passing though the joint), and transverse reinforcement.

Wide beam

Clause 1.2.4

If the width of beam exceeds corresponding column dimension, transverse reinforcement as required by clause nos. 7.4.7 and 7.4.8 of IS 13920 : 1993 shall be provided through the joint to provide confinement for longitudinal beam reinforcement outside the column core if such confinement

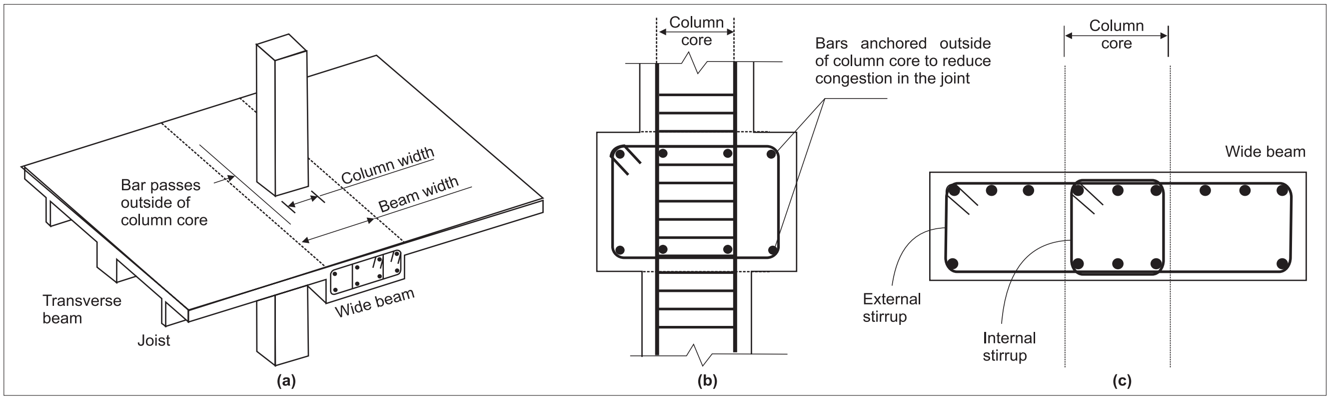

Fig 2 Connection between wide beam and column[8]

is not provided by a beam framing into the joint. In such a case, the value of width of beam b_b should be less than the values of $3b_c$ and $b_c + 1.5h_c$, where b_c and h_c are the column width and depth, respectively.

Commentary 1.2.4

This clause refers to the wide beam, that is, the width of the beam exceeds the corresponding column dimension as shown in *Fig* 2. In that case, the beam reinforcement not confined by the column reinforcement should be provided lateral support either by a girder framing into the same joint or by transverse reinforcement. The limit of maximum width of wide beam is specified to ensure the formation of beam plastic hinge. The maximum beam width recommended here is based on some experiments on joints between wide beam and column[9,10,11]. The limit recognises that the effective width of wide beam is closely related to the depth of column than it is to the depth of the wide beam.

Hook

Clause 1.2.5

In the exterior and corner joints, all the 135° hook of the cross-ties should be along the outer face of the column.

Shear design

Shear strength

Clause 1.3.1

The nominal shear strength of the joint shall not be taken greater than $1.5A_{ej}\sqrt{f_{ck}}$ for joints confined on all four faces, $1.2A_{ej}\sqrt{f_{ck}}$ for joints confined on three faces or two opposite faces, and $1.0A_{ej}\sqrt{f_{ck}}$ for others, where, A_{ej} = effective shear area of the joint ($b_j h_j$), b_j = effective width of joint as per clause 1.3.2, h_j = effective depth of joint as per clause 1.3.3, and f_{ck} = characteristic compressive strength of concrete cube in MPa.

Commentary 1.3.1

The concept and values of nominal shear strength specified are in line with ACI 318M- provisions[6]. The nominal shear strength value specified includes the shear carried by the concrete as well as the joint (shear) reinforcement.

Effective width of joint

Clause 1.3.2

The effective width of joint, *bj* (*Fig* 3) shall be obtained based on the following equations:

$$\begin{cases} Min\ [b_c; b_b + 0.5h_c] \text{ if } b_c > b_b,\ Fig\ 3(a) \\ Min\ [b_b; b_c + 0.5h_c] \text{ if } b_c < b_b,\ Fig\ 3(b) \end{cases}$$

where,

b_b = width of beam

b_c = width of column

h_c = depth of column in the considered direction of shear.

Effective depth of joint

Clause 1.3.3

The effective depth of joint h_j can be taken as depth of the column, h_c as shown in *Fig* 3.

Shear Force

Clause 1.3.4

Shear force in the joint shall be calculated assuming that the stress in flexural tensile reinforcement is $1.25f_y$, where f_y = yield stress of steel.

Commentary 1.3.4

Shear force in the joint due to earthquake load can be calculated as shown in *Fig* 4. The larger the tension force in the steel, the greater will be the shear in the joint. Hence, the tensile force in the reinforcement is conservatively taken as $1.25f_yA_{st}$, where f_y is the specified yield strength of steel bars

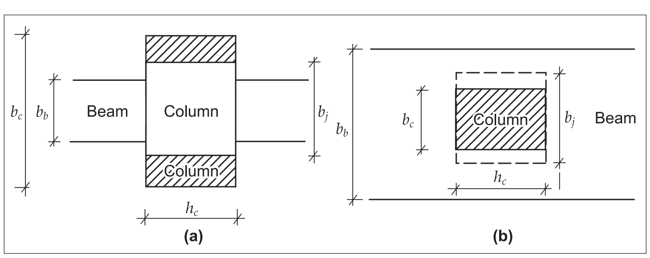

Fig 3 Effective width of joint (plan view)

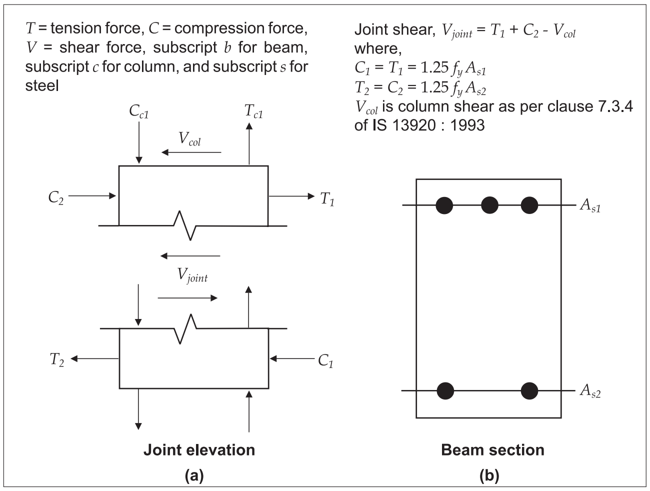

Fig 4 Evaluation of horizontal joint shear[12]

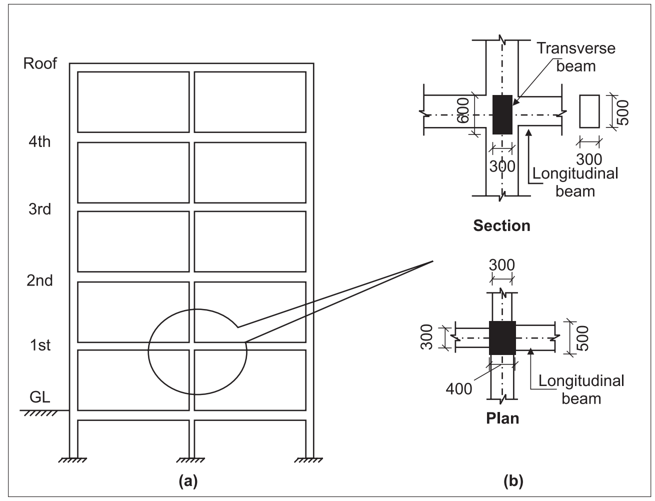

Fig 6 *(a)* Joint location in elevation of the building, *(b)* Details of the connected members in the joint

and A_{st} is cross sectional area of steel bars, for computation of joint shear to account for *(a)* the actual yield strength of the steel normally being greater than the specified yield strength f_y, and *(b)* the effect of strain hardening at high strain. Some experiments conducted in the structural engineering laboratory at IIT Kanpur on Indian high yield strength deformed (HYSD) (Fe415) steel bars found the actual yield strength (≈ 440 MPa) to be higher than the specified yield strength (415 MPa) and the ultimate stress of HYSD bars was found as ≈$1.27f_y$.[(13)]

Solved example

The detailed design of an interior joint in an intermediate RC moment resisting frame is explained here as per the above mentioned provisions. The structure is a ground plus four storey office building situated in Zone V. Examples on other different types of joints are available on the website, http://www.iitk.ac.in/nicee/IITK-GSDMA/EQ22.pdf.

Design data

The joint of column marked in *Fig* 5 is considered for design. The plan and sectional elevation of the building are shown in *Figs* 5 and 6. The details of the column and beam reinforcement meeting at the joint are shown in *Fig* 7. The transverse beam of size 300 mm × 600 mm is reinforced with 5ϕ20 + 4 ϕ16 (2374 mm^2, that is, 1.44 percent) at top and 5ϕ16 + 1ϕ20 (1320 mm^2, that is., 0.80 percent) at bottom. The hogging and sagging moment capacities of the transverse beams are evaluated as 377 kN-m and 246 kN-m, respectively. The longitudinal beam of size 300 mm × 500 mm is reinforced with 4ϕ20 + 5ϕ16 (2260 mm^2, that is, 1.67 percent) at top and 3ϕ20 + 4ϕ16 (1746 mm^2, that is, 1.29 percent) at bottom. The hogging and sagging moment capacities of the longitudinal beams are evaluated as 288 kN-m and 221 kN-m, respectively.

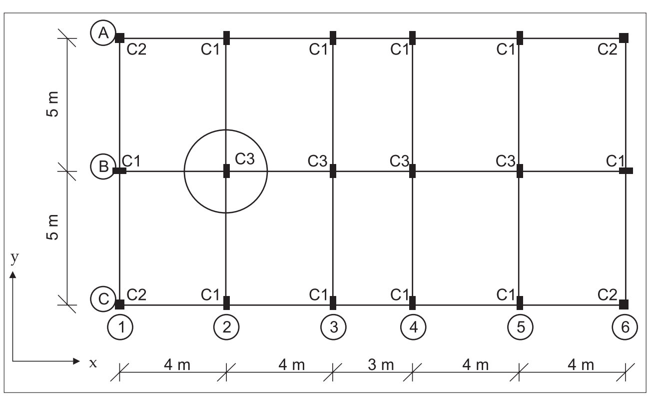

Fig 5 Plan of a ground plus four storey reinforced concrete office building

Minimum column size

Minimum size of column

$$= \text{maximum of} \begin{cases} 15 \times 20 \text{ mm} \\ 300 \text{ mm} \end{cases}$$

= 300 mm < width of column = 400 mm.

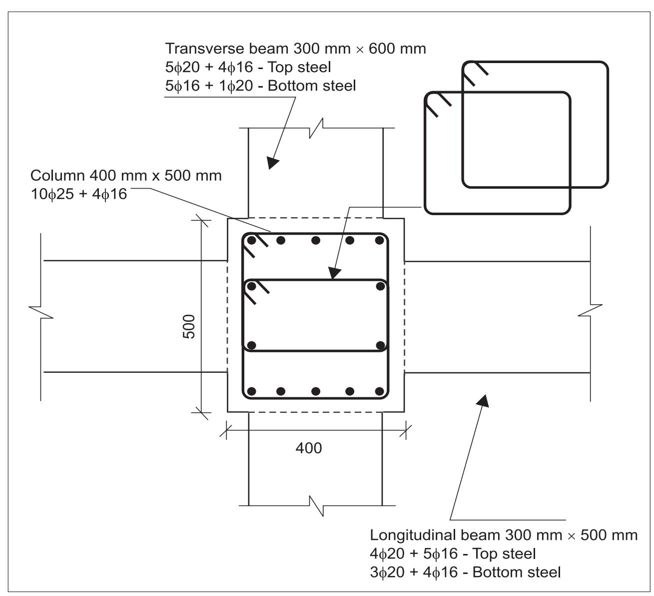

Fig 7 Reinforcement details for beams and column

Hence, the values are acceptable as per clause 1.0

Check for earthquake in y direction

Column shear

The column shears for sway to right and left is shown in *Fig* 8. For both the cases,

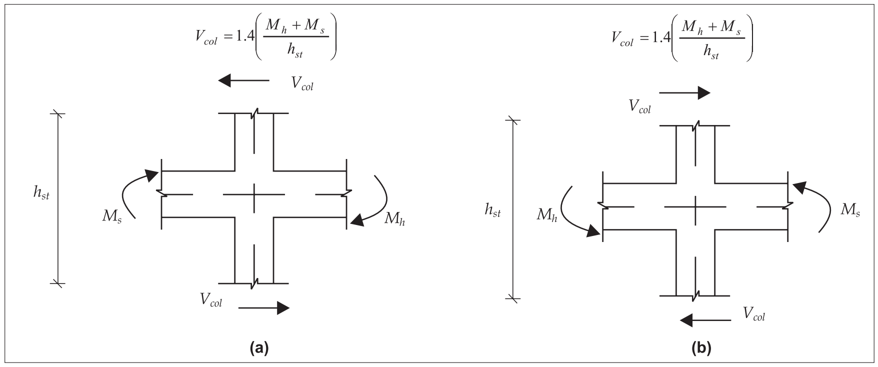

Fig 8 Column shear ***(a)*** **Sway to right** ***(b)*** **Sway to left**

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

$$= 1.4\left(\frac{377 + 246}{3}\right)$$

$$= 291 \text{ kN}$$

Joint shear

The development of forces in the joint for sway to right and left is shown in *Fig* 9.

Force developed in the top bars

$$T_1 = 1.25\, f_y A_{st} = 1.25 \times 415 \times 2374 \;/\; 1000 = 1232 \text{ kN} = C_1$$

The factor 1.25 is to account for the actual ultimate strength being higher than the actual yield strength as per clause 1.3.4

Force developed in the bottom bars

$$T_2 = 1.25\, f_y\, A_{st} = 1.25 \times 415 \times 1320 \;/\; 1000 = 685 \text{ kN} = C_2$$

Referring to clause 1.3.4

Joint Shear, $V_{Joint} = T_1 + C_2 - V_{col} = 1232 + 685 - 291 = 1626$ kN

Maximum value of T_1 and minimum value of V_{col} are used in the above equation.

Check for joint shear strength

The effective width provisions for joints are shown in *Fig* 3. As per clause 1.3.2 the effective width of the joint is lesser of the following two values:

(i) $b_j = b_b + 0.5 \times h_c$

(ii) $b_j = b_c$

$b_j = b_b + 0.5 \times h_c = 300 + 0.5 \times 500 = 550$ mm, or

$b_j = b_c = 400$ mm

Therefore, effective width of joint, $b_j = 400$ mm.

h_j = depth of the column = 500 mm

Effective shear area of the joint = $A_{ej} = b_j h_j$

The joint is confined on two opposite faces as per clause 1.2.3,

$$\text{Shear strength} = 1.2 A_{ej}\sqrt{f_{ck}}$$
$$= 1.2 \times (400 \times 500 \;/1000) \times \sqrt{20}$$
$$= 1070 \text{ kN} < 1626 \text{ kN.}$$

Hence, the values are unsafe as per clause 1.3.1

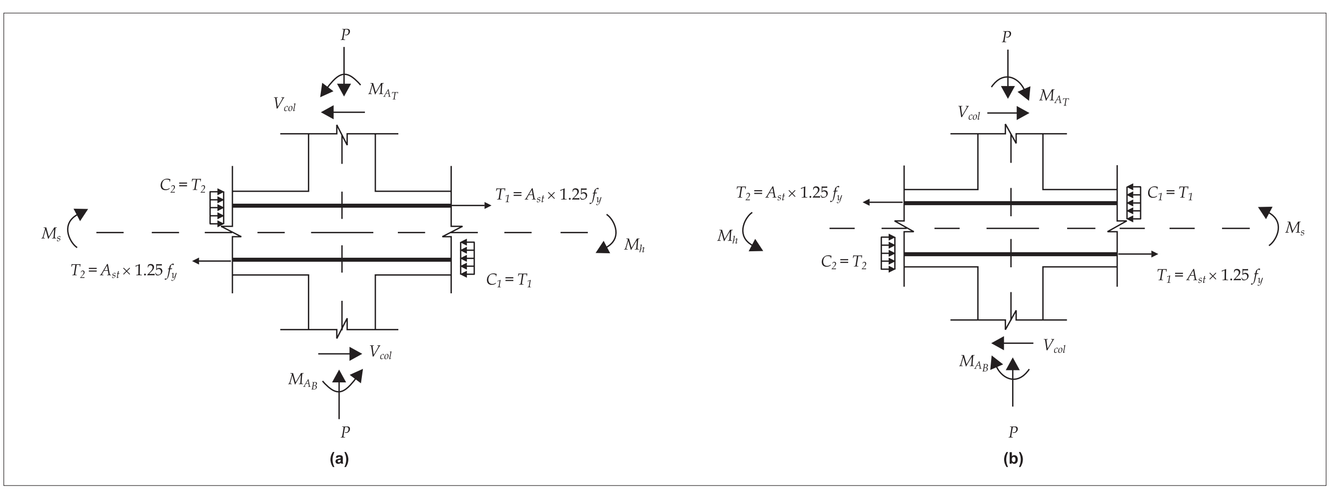

Fig 9 Free body diagram of the joint ***(a)*** **Sway to right,** ***(b)*** **Sway to left**

Check for flexural strength ratio

The hogging and sagging moment capacities of the transverse beams are 377 kN-m and 246 kN-m, respectively.

The column *is* reinforced with 10ϕ25 + 4ϕ16 bars (5714 mm^2, that is, 2.85 percent). Hence, p/f_{ck} = 2.85/20 = 0.14

It is conservative here to calculate the moment capacity of column with zero axial loads for lower values of $\frac{P_u}{f_{ck}bD}$. In actual practice, it is desirable to take minimum $\frac{M_u}{f_{ck}bD^2}$ corresponding to actual $\frac{P_u}{f_{ck}bD}$ obtained from different load combinations. It may be noted that for higher values of $\frac{P_u}{f_{ck}bD}$ the corresponding values of $\frac{M_u}{f_{ck}bD^2}$ will be less and hence the value corresponding to $\frac{P_u}{f_{ck}bD}$ = 0 .00. is to be considered. As per chart 44 of SP 16 : 1980, corresponding to $\frac{P_u}{f_{ck}bD}$ = 0 .00 for p/f_{ck} = 0.14 and d'/D = (40 + 25 /2) / 500 = 0.11, we get[14],

$$\frac{M_u}{f_{ck}bD^2} = 0.19$$

$$M_u = (0.19 \times 20 \times 400 \times 500^2) / 10^6 = 380 \text{ kN-m}$$

The joint is checked for strong-column-weak-beam as per clause 1.1.

$$\sum M_c = 380 + 380 = 760 \text{ kN-m}$$

$$\sum M_g = 377 + 246 = 623 \text{ kN-m}$$

The ratio of $\frac{\sum M_c}{\sum M_g}$ = 760 /623 = 1.2 > 1.1

Hence, requirement of strong-column-weak-beam condition is satisfied as per clause 1.1.

Check for earthquake in x direction

Column shear

The column shears for sway to right and left are shown in *Fig* 8. For both the cases,

$$V_{col} = 1.4\left(\frac{M_s + M_h}{h_{st}}\right) = 1.4\left(\frac{288 + 221}{3}\right) = 238 \text{ kN}$$

Joint shear

The development of forces in the joint for sway to right and left is shown in *Fig* 9.

Force developed in the top bars

$$T_1 = 1.25 f_y A_{st} = 1.25 \times 415 \times 2260 / 1000 = 1170 \text{ kN} = C_1$$

Force developed in the bottom bars

$$T_2 = 1.25 f_y A_{st} = 1.25 \times 415 \times 1746 / 1000 = 905 \text{ kN} = C_2$$

The joint shear is evaluated as per clause 1.3.4 considering maximum T_1 and minimum V_{col}.

$$V_{Joint} = T_1 + C_2 - V_{col} = 1170 + 905 - 238 = 1837 \text{ kN}$$

Check for joint shear strength

The effective width provisions for joints are shown in *Fig* 3. As per clause 1.3.2, the effective width of the joint is lesser of the following two values:

(i) $b_j = b_b + 0.5 \times h_c$ = 300 + 0.5 × 400 = 500 mm, or

(ii) $b_j = b_c$ = 500 mm

Adopt lesser of the two values,

b_j = 500 mm

h_j = depth of the column = 400 mm

Effective shear area of the joint = $A_{ej} = b_j h_j$

The joint is not confined as per clause 1.2.3

Shear strength $= 1.0 A_{ej}\sqrt{f_{ck}}$

$= 1.0 \times (500 \times 400 /1000) \times \sqrt{20}$

= 894 kN < 1837 kN

Hence, the values are unsafe as per clause 1.3.1

Check for flexural strength ratio

The limiting hogging and sagging moment capacities of the longitudinal beam are 288 kN-m and 221 kN-m, respectively. It is conservative here to calculate moment capacity of column with zero axial loads for lower values of $\frac{P_u}{f_{ck}bD}$. In actual practice, it is desirable to take minimum $\frac{M_u}{f_{ck}bD^2}$ corresponding to actual $\frac{P_u}{f_{ck}bD}$ obtained from different load combinations. It may be noted that for higher values of $\frac{P_u}{f_{ck}bD}$ the corresponding values of $\frac{M_u}{f_{ck}bD^2}$ will be less and hence the value corresponding to $\frac{P_u}{f_{ck}bD}$ = 0.00 is to be considered . As per chart 44 of SP 16 : 1980, corresponding to $\frac{P_u}{f_{ck}bD}$ = 0.00, for p/f_{ck}= *0.14* and d'/D = (40 + 25 / 2) / 400 = 0.13, we get[14],

$$\frac{M_u}{f_{ck}bD^2} = 0.178$$

$$M_u = (0.178 \times 20 \times 500 \times 400^2) / 10^6 = 284 \text{ kN-m}$$

The joint is checked for strong-columnweak-beam as per clause 1.1

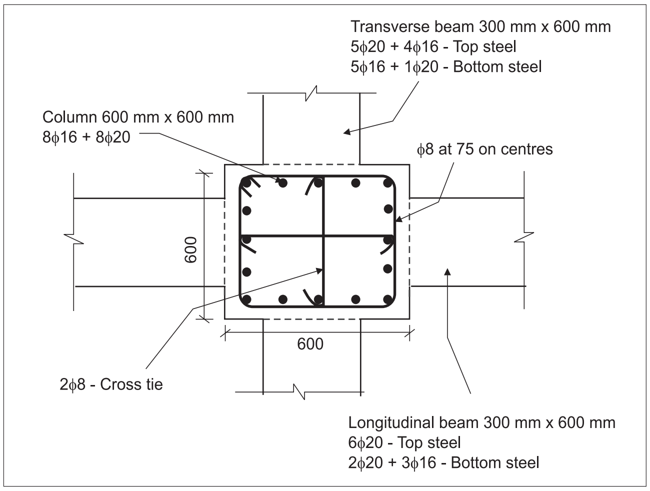

Fig 10 Revised reinforcement details for column and beams

$\sum M_c$ = 284 + 284 = 568 kN-m

$\sum M_g$ = 288 + 221 = 509 kN-m

The ratio of $\dfrac{\sum M_c}{\sum M_g}$ = 568/509 = 1.12 > 1.1.

Hence, requirement of strong-column-weak-beam condition is satisfied as per clause 1.1

Revision

As can be seen from the checks in the above section, the joint is not safe in shear. In such cases, the following three alternatives can be tried.

(i) **Increase of column section**

This option will not only increase the area of joint but also reduce the requirement of main longitudinal steel bars in the column owing to larger column size.

(ii) **Increase of 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 of grade of concrete**

This option will increase the shear strength of joint and also reduce the steel required in columns.

It is proposed to increase column size from 400 mm × 500 mm to 600 mm × 600 mm and longitudinal beam size from 300 mm × 500 mm to 300 mm × 600 mm. Member forces are taken as calculated earlier without reanalysis of the structure. In practice, the structure may be reanalysed.

The redesigned longitudinal beam of size 300 mm × 600 mm is reinforced with 6ϕ20 (1884 mm^2, that is, 1.14 percent) at top and 2ϕ20 + 3ϕ16 (1230 mm^2, that is, 0.74 percent) at bottom. The hogging and sagging moment capacities are evaluated as 293 kN-m and 229 kN-m, respectively.

The $\sum M_c$ required in transverse direction is 623 × 1.1 = 685 kN-m and 522 × 1.1 = 574 kN-m in longitudinal direction.

Hence, required moment capacity for column is M_c = 685/2 = 343 kN-m in y direction and 574 / 2 = 287 kN-m in x direction as per clause 1.1.

It is found that 1.1 percent steel is required to satisfy the above moment capacity of column (SP 16 : 1980[14]). Hence, change of the main longitudinal steel bars to 8ϕ20 + 8ϕ16 (4120 mm^2, that is 1.14 percent steel) is to be used. The revised reinforcement details are shown in *Fig* 10. This column section will satisfy the flexural strength check.

While redesigning the column, some of the load combinations may give an axial stress less than 0.1 f_{ck}. The section needs to be checked for flexure for these load combinations.

Minimum column size

Minimum size of column

$$= \text{maximum of} \begin{cases} 15 \times 25 \text{ mm} \\ 300 \text{ mm} \end{cases}$$

= 300 mm < width of column = 600 mm.

Hence, the values are acceptable as per clause 1.0

Check for earthquake in y direction

$b_j = b_b + 0.5 \times h_c = 300 + 0.5 \times 600 = 600$ mm

or, $b_j = b_c = 600$ mm

Adopt lesser of the two values,

b_j = 600 mm

h_j = depth of column = 600 mm

Shear strength $= 1.0 A_{ej}\sqrt{f_{ck}}$

$= 1.0 \times (600 \times 600 \,/1000) \times \sqrt{20}$

= 1610 kN < 1620 kN.

Here, shear strength value of 1610 kN is less than shear stress developed at the joint (1620 kN). Hence, one should re-design the joint dimensions because it is unsafe as per clause 1.3.1. But, since the difference is only 0.6 percent, this is ignored in the present case.

Check for earthquake in x direction

Referring to *Fig* 8, for both the cases, shear due to formation of plastic hinges in beams is,

$$V_{col} = 1.4\left(\frac{M_s + M_h}{h_{st}}\right) = 1.4\left(\frac{293 + 229}{3}\right) = 244 \text{ kN}$$

Referring to *Fig* 9, we get,

$$T_1 = 1.25\, f_y\, A_{st} = 1.25 \times 415 \times 1884 \;/\; 1000 = 978 \text{ kN} = C_1$$

$$T_2 = 1.25\, f_y\, A_{st} = 1.25 \times 415 \times 1230 \;/\; 1000 = 638 \text{ kN} = C_2$$

The joint shear is evaluated considering maximum T_1 and minimum V_{col}.

$$V_{Joint} = T_1 + C_2 - V_{col} = 978 + 638 - 244 = 1370 \text{ kN}$$

$$b_j = b_b + 0.5 \times h_c = 300 + 0.5 \times 600 = 600 \text{ mm}$$

or, $\quad b_j = b_c = 600$ mm

Adopt lesser of the two values,

$$b_j = 600 \text{ mm}$$

$$h_j = \text{depth of column} = 600 \text{ mm}$$

$$\begin{aligned}\text{Shear strength} &= 1.0 A_{ej} \sqrt{f_{ck}} \\ &= 1.0 \times (600 \times 600 \;/1000) \times \sqrt{20} \\ &= 1610 \text{ kN} > 1370 \text{ kN}.\end{aligned}$$

Hence, the values are safe as per clause 1.3.1

Confining links

In this case with the column dimensions revised to 600 mm × 600 mm, the width of beam is 300 mm, which is less than 3/4 width of column, that is, 3/4 × 600 = 450 mm. Hence, full confining reinforcement is required in the joint as per clause 1.2.1.

The spacing of links for the confining zone shall not exceed:

(i) ¼ of minimum column dimension, that is , 600 / 4 = 150 mm

(ii) But should not be less than 75 mm nor more than 100 mm (clause 7.4.6 of IS 13920 : 1993[3])

The area of cross section A_{sh} of the bar forming rectangular hoop to be used as special confining reinforcement shall not be less than

$$A_{sh} = \frac{0.18 \times S \times h \times f_{ck}}{f_y}\left(\frac{A_g}{A_k} - 1\right)$$ (clause 7.4.8 of IS 13920 : 1993[3])

Assuming, nominal cover of 40 mm to the longitudinal reinforcement, the area of concrete core, $A_k = (600 - 2 \times 40) \times (600 - 2 \times 40) = 27 \times 10^4 \text{ mm}^2$

A_g = gross area of the column cross section = $600 \times 600 = 36 \times 10^4 \text{ mm}^2$

h = longer dimension of the rectangular confining hoop measured at its outer face

= (600 - 40 × 2) = 520 mm > 300 mm

Hence, a single cross tie in both the directions will have to be provided. Thus,

$$h = 520/2 = 260 \text{ mm} < 300 \text{ mm}.$$

Assuming, rectangular hoops of diameter 8 mm, $A_{sh} = 50 \text{ mm}^2$,

$$50 = \frac{0.18 \times S \times 260 \times 20}{415}\left(\frac{36 \times 10^4}{27 \times 10^4} - 1\right)$$

$$S = \text{spacing of hoops} = 65 \text{ mm} < 75 \text{ mm}$$

Provide ϕ8 mm confining links at 75 mm on centres in the joint.

Conclusion

Beam-column joints in moment resisting frames have traditionally been neglected in design process while the individual connected elements, that is , beams and columns, have received considerable attention in design. Research on beam-column joints of reinforcement concrete moment resisting frame was started only in the 1970s. The 1993 version of IS 13920 : 1993 incorporated some provisions on the design of beam-column joints[3]. However, these provisions are inadequate to prevent shear and bond failure of beam-column joints in severe seismic shaking. Therefore, these provisions need to be upgraded substantially with inclusion of explicit provisions on shear design and anchorage requirements. This article proposes provisions for shear design of beam-column joint and anchorage requirements of tension beam bars in the joint area. It also suggests provisions for the confinement of wide beam and column connections. A solved design example has been provided to illustrate these provisions for an interior beam-column joint. In the solved example it was seen that the joint fails in shear for design earthquake shaking in both x and y directions. The joint can be redesigned by increasing size of column, size of beam, or grade of concrete. In this example, however, the increase of size of column and depth of beam are sufficient to satisfy the shear strength requirements.

Acknowledgement

This work has been supported through a project entitled, 'Review of Building Codes and Preparation of Commentary and Handbooks,' awarded to IIT Kanpur by Gujarat State Disaster Management Authority (GSDMA), Gandhinagar through World Bank funding. The views and opinions expressed here are those of the authors and do not necessarily reflect those of GSDMA or the World Bank. The authors are grateful to Dr S.R. Uma of University of Canterbury and Dr Bhupinder Singh of IIT Roorkee for critical review of the solved examples.

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