Design of joints in RC structures with particular reference to seismic conditions

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The failure of reinforced concrete structures in recent earthquakes in several countries has caused concern about the performance of beam-column joints. IS codes do not include recommendations on beam-column joints explicitly. The reversal of forces in beam-column joints during earthquakes may cause distress and often failure, when not designed and detailed properly. The fifth revision of IS 1893 has brought more than 50 percent of the country under moderate and severe seismic zones. Under these circumstances, the detailing of joints assumes more importance. The behaviour and design of two-, three- and four-member beam – column joints in framed structures are discussed; obtuse and acute angle joints are included. Detailing of the joints based on experimental investigations is also explained. The specifications of American, New Zealand and Indian codes of practice are appraised. An equation for calculating the area of joint transverse reinforcement has been proposed for the Indian code, based on recent research.

The performance of framed structures depends not only upon the individual structural elements but also upon the integrity of the joints. In most of the cases, joints of framed structures are subjected to the most critical loading under seismic conditions. The failure of several reinforced concrete (RC) structures during the recent earthquakes in India as well as other countries causes concern about the performance of the beam-column joints [1, 2].

However, despite the significance of the joints in sustaining large deformations and forces during earthquakes, specific guidelines are not explicitly included in Indian codes of practice (IS 456 : 2000 and IS 13920 : 1993). While considerable attention is devoted to the design of individual elements (slabs, beams and columns), no conscious efforts are made to design joints in the absence of suitable guidelines. It appears that the integrity and strength of such joints are assumed to be satisfied by lapping the reinforcement. The design and detailing of joints play a crucial role in providing ductility and strength required to sustain large deformations and reversed stresses during earthquakes.

One of the basic assumptions of the frame analysis is that the joints are strong enough to sustain the forces (moments, axial and shear forces) generated by the loading, and to transfer the forces from one structural member to another (beams to columns, in most of the cases). It is also assumed that all the joints are rigid, and the members meeting at a joint deform (rotate) by the same angle. Hence, it is clear that unless the joints are designed to sustain these forces and deformations, the performance of structures will not be satisfactory under all the loading conditions, especially under seismic conditions. Post-earthquake analyses of structures, accidental loading or laboratory tests show that the distress in the joint region is the most frequent cause of failure, rather than the failure of the connected elements[1-8].

Considerable research and test results are reported on the strength and behaviour of beam-column joints[3-13]. Recommendations on the design and detailing are also available, based on these results[14 - 25]. The behaviour of the beam-column joints normally encountered in residential and commercial multistorey buildings is discussed in this paper. Acute and obtuse angled joints, and retaining walls are also included. The design and detailing procedures are summarised to provide guidelines for satisfactory

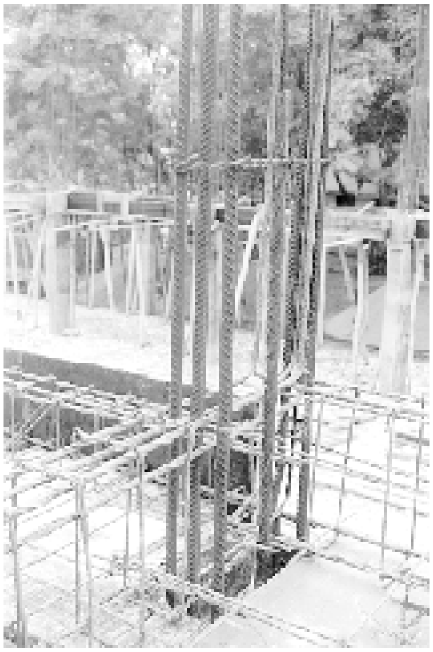

Fig 1 Typical beam-column joint with ample length of anchorage for beam reinforcement in the column but with incorrect placement

Fig 3 Poor quality concrete at the critical region of the joint, obviously due to poor quality formwork coupled with inadequate compaction

Fig 4 Poor awareness about joint detailing; main reinforcement in the column is bent to allow for change in the cross section

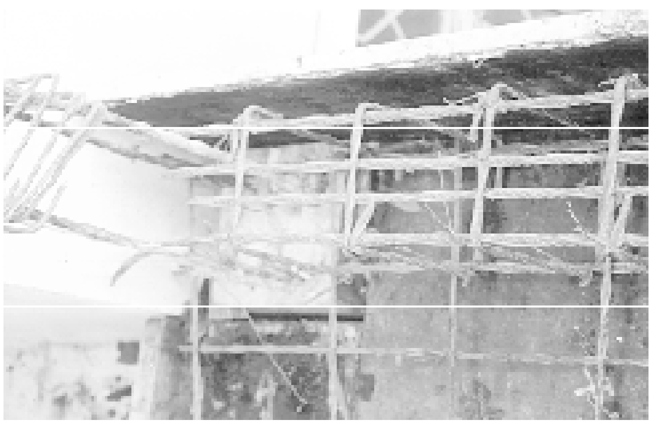

Fig 2 The beam reinforcement is not anchored properly in the structure

performance of joints under critical loading conditions.

The approaches of the ACI and New Zealand differ in designing the joints[14, 15, 17, 18]. The former does not consider the joint stirrups while estimating the joint shear capacity, while the capacity of joint stirrups is accounted in the latter approach.

Some of the design and construction practices peculiar to India are also discussed. The revised IS 1893: 2002 has enhanced the lateral forces on structures considerably compared to the previous version, which makes the design of joints imperative[26]. As per this code, more than 50 percent area of India falls under moderate and severe earthquake, thus signifying the importance of earthquake detailing of joints.

Requirements of beam-column joints

The essential requirements for the satisfactory performance of a joint in an RC structure can be summarised as follows[6, 14].

(*i*) A joint should exhibit a service load performance equal to or greater than that of the members it joins that is the failure should not occur within the joints. If at all, failure due to overloading should occur in beams through large flexural cracking and plastic hinge formation, and not in columns.

(*ii*) A joint should possess a strength that corresponds at least to the most adverse load combinations that the adjoining members could possibly sustain repeatedly several times, if possible.

(*iii*) The strength of the joint should not normally govern the strength of the structure, and its behaviour should not hinder the development of the full strength of the adjoining members.

(*iv*) Ease of fabrication and good access for placing and compacting concrete are the other significant parameters of joint design.

Design and detailing of joints

The problems of detailing and construction of beam-column joints are often not appreciated by designers. Because of the restricted space available in the joint block, the detailing of reinforcement assumes more significance than anywhere else. Indeed, the conflict between the small size bar requirement for good performance, and large size bars required for ease of placement and concreting is more obvious at the joints than anywhere else. This is particularly true at internal joints, where the beams intersect in both the horizontal directions, or where large moments are to be sustained by the connections. In the absence of specifications from the designers, site engineers often adopt expedient procedures for detailing, which are not always conducive for satisfactory structural performance.

Some of the incorrect detailing practices adopted by site engineers in India are illustrated in *Figs* 1 to 4. *Fig* 1 shows a typical beam-column joint with ample length of anchorage for beam reinforcement in the column but with incorrect placement. The beam bars are bent upwards instead of downwards; such disposition of reinforcement may cause diagonal cracking, leading to shear failure of the joint[6,7]. The beam reinforcement is not anchored properly in the structure shown in *Fig* 2; while the negative steel in the beam may be adequate to sustain the forces under the applied loads, inadequate anchorage may render the bars ineffective under critical loading conditions (seismic loading, for instance). *Fig* 3 shows poor quality concrete at the critical region of the joint, obviously due to poor quality formwork coupled with inadequate compaction. Possibly, the site engineer was not aware of the significance of the joint, while allowing column bars to be bent in the structure shown in *Fig* 4. The bending of bars has damaged concrete at the joint, and destroyed the bond between steel bars and concrete. The crooked bars at the column base may cause excessive stresses in the column, and lead to early distress, especially under lateral forces induced by earthquakes.

In all these cases, the joints do not appear to have been designed or checked for their performance; it is unlikely that they will behave satisfactorily under the critical forces induced by earthquakes. It may also be noted that shear reinforcement is not usually provided in these joints; the column ties are also not to the codal specifications[22]. Further, some of the beam reinforcement bars lie outside the column bars, *Figs* 1 and 2 which is not a correct practice; the beam reinforcement should lie within the column reinforcement.

Because the joint block area is small relative to the member sizes, it is essential to consider localised stress distribution within the joints. A simplified force system may be adopted in designing beam to column connections. The quantity of steel required is calculated on the assumption that steel reaches the design yield stress, and the concrete its compressive stress. Where local bearing or bond failure is expected, the lower of the two capacities for flexural and local failure should be adopted based on experimental results. It is essential to prevent bond and anchorage failure within the joints, especially at the external joints, through proper design and detailing practice.

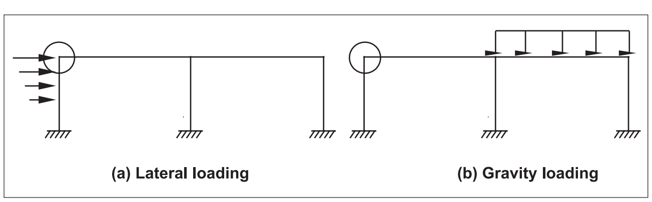

Fig 5 Opening joints in a frame

Corner joints

The external joints (corner joints) of a frame can be broadly classified into opening and closing corners. The corners that tend to open (increase the included angle) are shown circled in *Fig* 5, while those at the right ends tend to decrease the included angle, and are termed closing corners. The internal joints tend to open on one side, and close on the other depending upon the load applied.

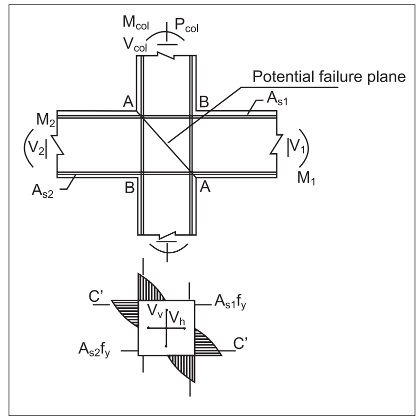

Fig 6 Forces on an interior beam - column joints

The corner joints will be subjected to alternate opening and closing forces under seismic loading. For closing corners, tests have shown that the usual detailing will be satisfactory. However, for an opening joint with the same details, the flexural efficiency may be only about 25 percent of the strength of the members meeting at the joint[11, 16]. The efficiency of connections and joints is usually defined as the ratio of failure moment of the connection or joint to the capacity of adjoining members[12, 13].

Typical corner joints in plane frames comprise two members, *Fig* 5, whilst in a three-dimensional frame, additional members, inclined to the vertical plane, may be present.

Internal joints

For a four-member connection, *Fig* 6, if the two beam moments are in equilibrium with one another then no additional reinforcement is required. *Fig 6* (a) indicates the force distribution in a frame with lateral loading (wind or earthquake loads) with two opening corners at locations B, and two closing corners at locations A. Diagonal cracks are liable to form along A-A unless stirrups are provided along B-B, or the principal tensile stress is restricted. The stresses induced on an internal beam-column joint are shown in *Fig 6 (b)*. The principal mechanisms of failure of such a joint are:

- shear within the joint
- anchorage failure of bars, if anchored within the joint, and
- bond failure of beam or column bars passing through the joint.

The joint is designed based on the fundamental concept that failure should not occur within the joint; that is, it is strong enough to withstand the yielding of connecting beams (usually) or columns.

Seismic shear resistance

Horizontal shear force of resistance within the joint, *Fig* 6, can be calculated by[17, 18]

$$V_h = (A_{s1} + A_{s2})\, f_{sy}^{*} + C' - V_{col} \qquad (1)$$

where,

A_{s1} = area of tension steel in beam 1

A_{s2} = area of compression steel in beam 2

C′ = compressive force in concrete = $k\, f_{ck}\, b\, x$

f_{sy}^{*} = factored yield strength which allows for over strength (the value of f_{sy}^{*} may be taken to be1.25 times f_{sy}, the characteristic strength of steel)

b = breadth of the beam

k, x = stress block parameters

f_{ck} = characteristic strength of concrete

V_{col} = net column shear force

This shear V_h is resisted by compressive strut action in the concrete and the horizontal stirrups as shown in *Fig* 7. Conservatively, the area of horizontal stirrups can be calculated for steel of design strength f_y (= f_{sy}/γ ; being the partial safety factor of steel) as

$$= V_h/f_y \tag{2}$$

ACI 318 – 1999 recommends that the cross-sectional area of horizontal stirrups should not be less than either

$$A_{sh} = 0.3\,(s\, h_c f'_c / f_{yh})\,[(A_g / A_c) - 1] \tag{3}$$

or

$$A_{sh} = 0.09\, s h_c f'_c / f_{yh} \tag{3a}$$

In the case of circular columns,

$$A_{sh} \geq 0.12 s h_c f_c / f_{yh} \tag{3b}$$

where,

A_{sh} = area of stirrups and cross-ties,

s = stirrup spacing,

h_c = dimension of the concrete core perpendicular to transverse reinforcement under consideration (centre-to-centre of perimeter reinforcement),

A_g = gross area of the concrete section,

A_c = area of the concrete core (to the outside of the stirrups),

f'_c = concrete cylinder compressive strength,

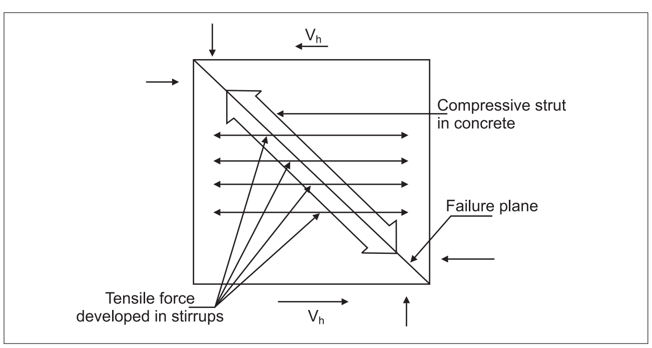

Fig 7 Horizontal shear resistance in a beam-column joint[17]

f_{yh} = yield strength of the stirrups[14].

The American code also stipulates that the spacing of these transverse reinforcement should not exceed the least of one quarter of the minimum column dimension, six times the diameter of the longitudinal bars to be restrained or S_x given by

$$S_x = 100+[(350-h_x)/3] \tag{4}$$

where,

h_x = maximum horizontal spacing of cross ties or hoops

Recently Saaticioglu and Razvi modified this equation to take into account the allowable storey drift ratio of 2.5 percent as[19]

$$\rho_c = \frac{0.35 f'_c}{f_{yh}}\left[\frac{Ag}{Ac} - 1\right]\frac{P_u}{\sqrt{k_2 \phi P_o}} \tag{5}$$

where,

r_c = [A_{sh} / ($h_c\, s$)], the area ratio of transverse confinement reinforcement

f = Capacity reduction factor (= 0.90)

P_o = Nominal concentric compressive capacity of column

P_u = Maximum axial load on column during earthquake

k_2 = Confinement efficiency parameter $\left[= 0.15\sqrt{b_c b_c}/(s s_1)\right]$

b_c = Core dimension, centre to centre of perimeter ties

s = spacing of transverse reinforcement along the column height

s_1 = spacing of longitudinal reinforcement, laterally supported by corner of hoop or hook of cross tie.

The amount of transverse steel required by equations (3) and (4) is linearly related to the compressive strength of concrete, f_{ck}. This may result in very large amount of transverse steel when high strength concrete (for example 100 N / mm^2) is used[20].

Stirrups must cross the failure plane shown in *Fig* 7, and be anchored at a distance that is not less than one-third of the appropriate column dimension on each side of it[17]. The maximum spacing of stirrups should not exceed that appropriate to the adjacent column.

Often the reinforcement in a beam-column joint is severely congested due to too many bars converging within a limited space of the joint (especially when the joint is confined on all the four sides by connecting beams of equal size). The beam and column sizes must be adopted carefully. In case the sizes of beam and column are the same, it will be very difficult to place the reinforcement within the joint. Usually the bars are cranked in plan as shown in *Fig* 8 or in

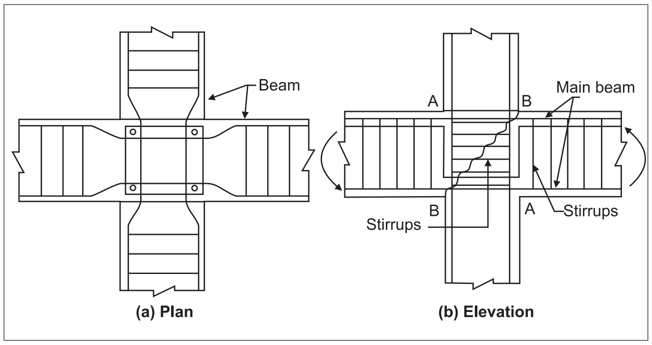

Fig 8 Undesirable details at beam-column joint

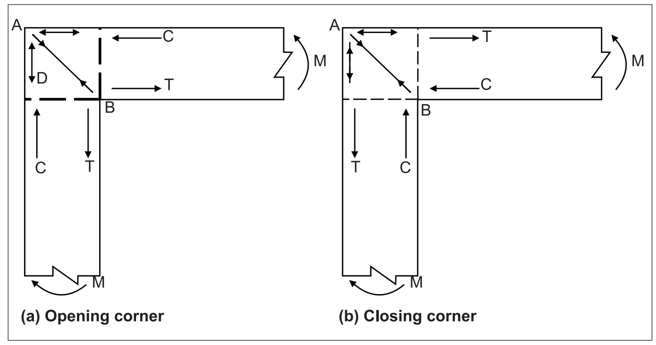

Fig 9 Forces in a two member connection I-joints[8]

elevation, or in both in plan and elevation, which is not a desirable arrangement. Another undesirable practice is to place the beam reinforcement outside the column bars *Figs* 1 and 2. It is possible to avoid congestion of steel and the above arrangements by selecting a little larger section for the column, thereby reducing reinforcement fraction.

Bond stresses

High bond stresses may develop at interior beam-column joints, where bars may have a high tensile stress at one face and a high compressive stress on the other face. Further, the penetration of yield zone from the plastic hinge in the beam may reduce the effective bond. The loss of bond at the column face results in high bearing stresses in concrete, which can be sustained only if the concrete within the joint region is well compacted. The New Zealand code limits the bar diameter to 1 / 25 of the column depth for grade 275 steel (f_{sy} = 275 N/mm^2) or 1 / 35 for grade 380 steel (f_{sy} = 380 N/mm^2) to avoid bond failure within the joint[18].

Shear stresses

ACI 318-99 restricts the shear stresses (in MPa) for joints in order to avoid the possibility of failure in the compression struts to the values indicated below [14]:

(i) joints confined on all the four faces : $1.7\, A_j \sqrt{f'_c}$ (6)

(ii) joints confined on three faces or on two opposite faces: $1.25\, A_j \sqrt{f'_c}$ (7)

(iii) others : $1.0\, A_j \sqrt{f'_c}$ (8)

where,

A_j = Effective cross–sectional area of the joint, in a plane parallel to plane of reinforcement generating shear in the joint in m^2. The joint depth is the overall depth of the column.

Where a beam frames into a support of larger width, the effective width of the joint should not exceed the smaller of:

(i) beam width plus the joint depth,

(ii) twice the smaller perpendicular distance from the longitudinal axis of the beam to the column side.

A joint is regarded as confined if members frame into each face of the joint, each member covering at least three-quarters of the joint face area[14].

External beam-column joints

Two-member beam to column joints, which occur in portal frames and at the top of building frames, are probably the most difficult joints to design, particularly when applied moments tend to 'open' the joint. Opening joints also occur in corners of tank walls and in wing wall-to-abutment junctions in bridges.

The simplified force system for an opening corner joint, shown in *Fig* 9 (a), illustrates how both compressive (C) and tensile forces must change direction by 90^o as they 'go round the corner'. Consequently, and in the absence of diagonal tie member (D), the tensile forces (T) will tend to straighten the reinforcement bar, which will then pull out of the corner at B. There will also be a tendency for the compressive forces (C) to "push off" the concrete at corner A.

The directions of all the forces are reversed for a closing corner as indicated in *Fig* 9 (b), and consequently the diagonal member D will be in compression. The resulting diagonal compressive force can be resisted by the concrete without reinforcement, and hence the joint strength is more readily achieved than with an opening corner.

A number of reinforcement layouts have been suggested for the two-member beam – column joints. The efficiencies of four such arrangements of reinforcement, shown in *Fig* 10 for opening corners are shown graphically in *Fig* 11 [8,10]. The efficiencies of all the joints are not satisfactory when acting as opening corners, though they proved to be satisfactory for

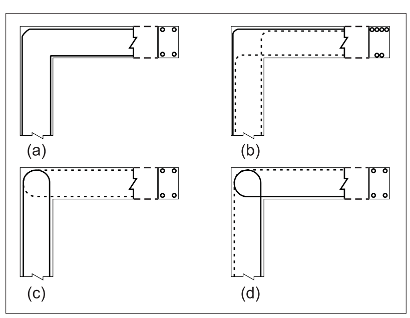

Fig 10 Basic reinforcement details for two-member connections

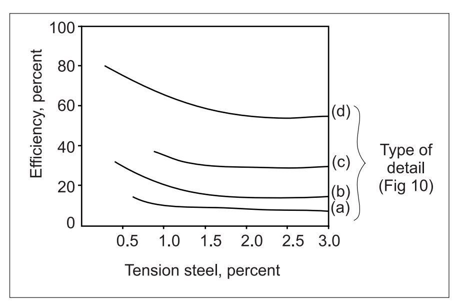

Fig 11 Efficiency of opening corners[8,11]

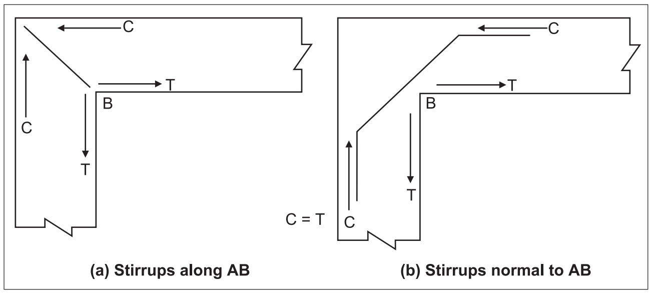

Fig 12 Diagonal stirrups to be used with basic reinforcement details[4,1]

closing corners[8]. However, it has been found that the joint efficiency can be enhanced by including diagonal stirrups along AB or perpendicular to AB as shown in *Fig* 12 [4, 10]. The diagonal stirrup along AB as shown in *Fig* 12 (a) has the greatest effect on joint strength. Although the stirrup perpendicular to AB, *Fig* 12 (b) has less effect on joint strength, its omission may lead to unacceptably wide cracks [8].

Half the area of main reinforcement should be provided as inclined bars, for a main steel ratio up to 1.0 percent and equal to the main steel for 1.0 to 1.5 percent [11]. If the main reinforcement is less than 0.4 percent, the inclined bars may be omitted altogether. The diagonal stirrups may be included in a corner chamfer, if provided. It is also desirable to restrict the main reinforcement percentage to about 0.8 to 1.2 percent in order to avoid failure in the corner [7,10].

The area of radial hoops required is approximately given by

$$a_{sj} = \left[\left(f_y / f_{yj}\right)\sqrt{1+\left(h_1/h_2\right)^2}\right]\left(A_{s1}/n\right) \qquad (9)$$

where,

f_{yj} = yield strength of the radial hoops with 'n' legs,

f_y = yield strength of the main steel,

h_1 and h_2 = beam depth and column width respectively, and

A_{s1} = the area of tension steel in the beam.

It is also suggested that the radial hoops are to be provided only when the flexural steel content exceeds 0.5 percent.

It may be noted that the problem is not so severe in obtuse angled corners, since the internal forces are not turned through such a large angle.

Desayi and Kumar concluded from the tests on six types of L-joints shown in *Fig* 13 that details B, C and E developed the full strength of the members (efficiency > 100 percent)[24]. They found that details D and F displayed about 62 percent average efficiency (η), while detail A was only about 40 percent efficient. They also observed that detail C developed the minimum crack width (about 0.2 mm) at working loads.

Desai and Kumar proposed the following equation for predicting the ultimate moment capacity of the joint, based on the area of steel resolved normal to the direction of the potential crack, A_{sj} (A_{sj} may be calculated as shown in *Fig* 14)

$$M_{uc} = 0.607\, f_{sp}\, bd^2 + 0.566\, f_y\, A_{sj}\, d \qquad (10)$$

where

f_{sp} = split cylinder tensile strength of concrete,

b = breath of beam at the joint

d = effective depth at the joint

A_{sj} = area of steel across the crack in the joint

f_y = yield stress of the joint steel

The above equation was found to give reasonably accurate values when compared with their experimental results[24].

T-joints

T-joints in cast-in-situ reinforced concrete may be classified into two main groups:

(i) connections between members of small width, for example between a column and a beam,

(ii) connection between walls and slabs (between bridge slab and the supporting pier), where the joints have

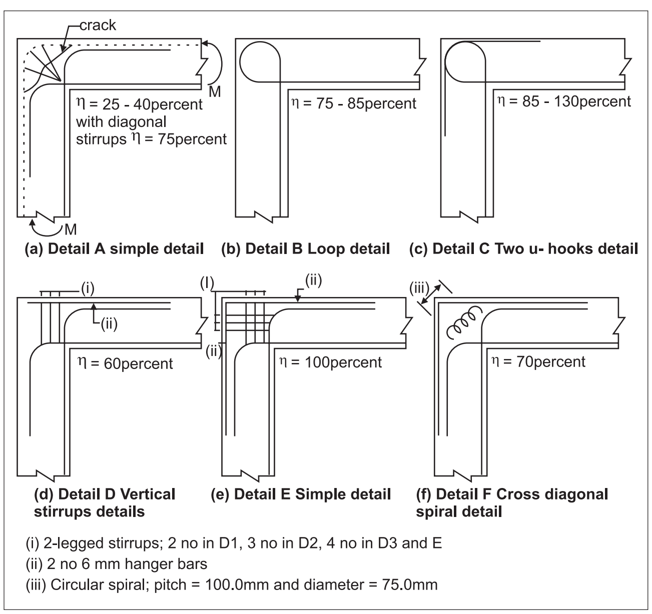

Fig 13 Six type of L-joints considered in Reference 24

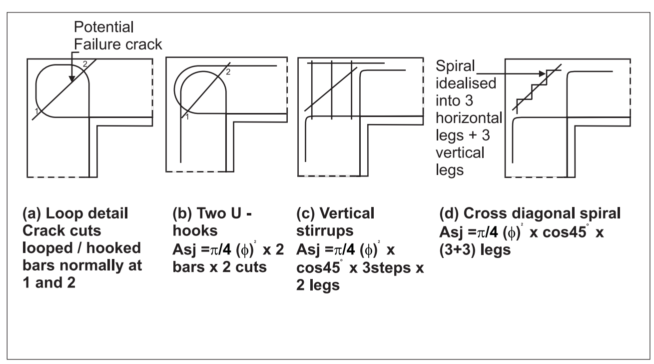

Fig 14 Typical calculations for A_{sj}[24]

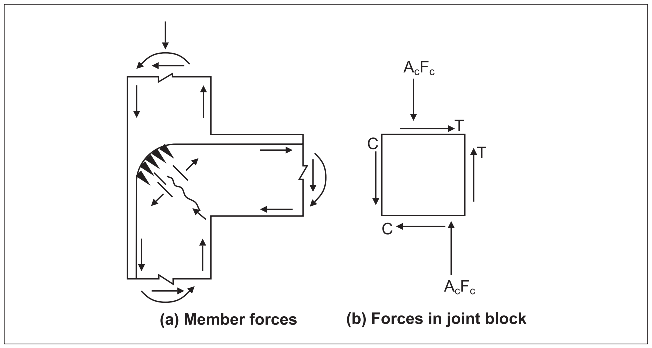

Fig 15 Forces in a three member T-joint[6]

large longitudinal dimensions.

In the latter case, due to constructional difficulties, stirrups should be avoided.

Such joints comprise closing and opening corners on either side of the joint. Similar to two member joints, here also the closing corner is stronger than opening corner. The forces acting on a T-joint are shown in *Fig* 15. The bent tensile bar induces contact pressure under the bend, which is balanced by a diagonal compression strut with the compressive force F_c. Diagonal tensile stress F_t in the joint may cause a diagonal crack as shown in *Fig* 15 (b).

Two details, one comprising L - bars from the beam to lower column, and the other with U-bars, are shown in *Fig* 16 The L- and U - bars should have sufficient overlap with main steel to develop full anchorage force. If the depth of the beam is greater than 600 mm, SP 34 recommends the use of U - bars as shown in *Fig* 16 (b)[23]. Experimental evidence indicates that such details develop the ultimate moment for beam reinforcement percentage up to 1.2 percent, unless diagonal ties in the form of stirrups (as shown in *Fig* 12) are provided [4, 25]. Inclusion of such diagonal ties will result in congestion of reinforcement. When they are not included and where beam reinforcement percentages exceed 1.2 percent, the following design procedure is recommended in order to avoid unacceptably wide diagonal cracks in the joint[8]. It has also been found that the geometry of the joint has a profound effect on its behaviour[6]. For example, deeper beams framing into shallow columns do not perform satisfactorily.

The Maximum shear stress on the joint block is taken as [8]

$$q_{max} = 1.5\, T / A_c \qquad (11)$$

where,

T = tensile force in the beam reinforcement,

A_c = area of the joint block.

The shear stress (Equation 11) should not exceed the maximum permissible shear stress given by

$$q_{all} = (f^2_t + f_t f_c) \qquad (12)$$

where

f_t = tensile strength of concrete,

f_c = compressive stress in the column.

The values corresponding to the design procedure are adopted in the above equation (limit state or working stress). This limitation is in effect, a limitation on the beam moment, which can be safely transmitted by the joint block. Alternatively, the principal tensile stress as given below could also be checked [4]

$$\text{Principle tensile stress} = f_c/2\sqrt{\left(f^2{}_c/4 + q^2{}_{max}\right)} \qquad (13)$$

The value given above should not exceed the design tensile strength of the concrete f_t at the serviceability limit state. Inclined reinforcement should be provided at the upper corner in order to reduce cracking at the junction. Only the ties that are situated in the outer two-thirds length of potential diagonal failure crack, which runs from corner to corner of the joint, should be considered to be effective, *Fig*16 (b). Thus, if V_s is the joint shear to be carried by the ties,

$$A_s = [0.5\, V_s\, s / (d\, f_y)] \qquad (14)$$

where,

A_s = total area of the tie legs in a set making up one layer of shear reinforcement, and

d = effective depth of the beam.

Anchorage of bars at joints

Frequently, joints are the weak links in a frame due to lack of adequate anchorage for bars extending into the joints from the columns and beams. Unless special measures are taken to remove the plastic hinge region away from the face of the column, the onset of yielding in the beam will penetrate the column area. For this reason, Key[17] suggests that the anchorage length of beam bars, anchored within the column area in external joints, should be reduced by the lesser of half the column depth or 10 times the bar diameter as illustrated in *Fig* 17. Park and Paulay suggest that for earthquake resistant structures the development length of the beam reinforcement should be computed from the beginning of 90^o bend, rather than from the face of the column[6]. In deep columns and whenever straight beam bars are preferred, mechanical anchors could be used[6] *Fig* 17(b). The top bars in a beam, passing through holes in a bearing plate may be welded to a steel plate[6]. One solution to the difficult problem of anchoring beam bars in external joints (especially for

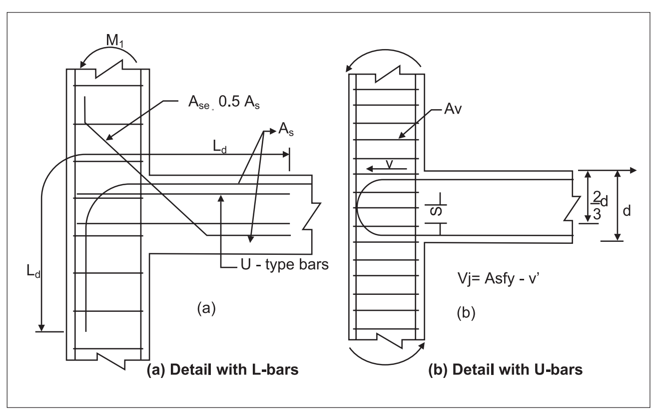

Fig 16 Reinforcement for three member connections

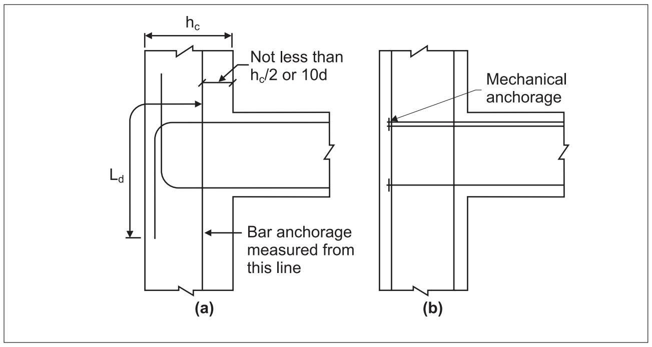

Fig 17 Anchorage of beam bars at an external joint[17]

shallow columns) is the use of projecting beam stubs as shown in *Fig* 18. This type of detailing has been adopted in Christchurch, New Zealand[17, 6]. However, architects may object to this solution on aesthetic grounds.

Indian code provisons

The Indian code on plain and reinforced concrete, IS 456:2000, does not contain any provision for the design of beam-column joints[27]. However, the Indian Code for earthquake resistant design and construction of buildings suggests that the transverse reinforcement as required at the end of column shall be provided through out the connection[21]. The area of reinforcement at the end of rectangular column is given by [21,23]

$$A_{sh} = 0.16 s h_c \frac{f_c}{f_{yh}} \left(\frac{A_g}{A_c} - 1\right) \tag{15}$$

For circular column this equation is given by

$$A_{sh} = 0.08 s D_k \frac{f_c}{f_{yh}} \left(\frac{A_g}{A_c} - 1\right) \tag{16}$$

where

f_c = 28-day compressive strength of concrete

D_k = diameter of core measured to the outside of the spiral.

It is to be noted that these equations are based on the ACI Code provisions given in Equations (3) and (4). In fact, the ACI Code correctly assumes that the spiral hoops in circular concrete columns provide better confinement than the rectangular ties of rectangular columns and gives a coefficient of 0.45 for circular columns as compared to the value 0.3 given in Equation 3. But the Indian code wrongly gives a lesser value for the coefficient for circular column as compared to the rectangular column and gives arbitrary values for these coefficients.

It is of interest to note that IS 13920 gives these coefficients as 0.09 and 0.18 for circular and rectangular column respectively[23]. It also specifies that the spacing of hoops shall not exceed one fourth of minimum member dimension but need not be less than 75 mm nor more than 100 mm. The code also suggests that if the connection is confined by beams from all four sides and if each beam width is at least 75 percent of column width, the amount of transverse reinforcement may be reduced by 50 percent[21,22]. In this case, the spacing of hoops shall not exceed 150 mm. In addition IS 13920 stipulates that the beam reinforcement in an external joint should be well anchored (with an anchorage length, beyond the inner face of the column, equal to the development length in tension plus 10 times the bar diameter minus the allowance for 90^{o} bend(s). In an internal joint both face bars of the beam should be taken continuously though the column.

SP 34 recommends that if the area of reinforcement exceeds 1.0 percent, diagonal stirrups as shown in *Fig* 12 (a) as well as splay steel as shown in *Fig* 8 (b) should be provided along with a corner haunch[23]. These recommendations are in conformity with international practices.

Bearing stresses inside bends

IS 456 : 2000 recommends that the high bearing stresses induced in the concrete on the inside of bends can be estimated as[27]

$$\sigma_b = F_{bt} / (r\,\phi) \tag{17}$$

where

F_{bt} = tensile force in the bar at ultimate loads,

r = internal radius of the bend,

f = bar diameter.

The Indian code also states that this bearing stress should not exceed $1.5 f_{ck} / [1 + (2\phi / a_b)]$ at the limit state [27].

Where a_b = centre-to-centre distance between the bars perpendicular to the plane of the bend; for a bar adjacent to the face of a member, a_b should be taken as the cover plus bar diameter ϕ.

High strength concrete columns

Recent research on high-strength concrete column indicates that the strength gain due to confinement is independent of concrete strength and percentage of strength gain is lower for high strength concretes[19]. Hence high strength concrete columns may require proportionally more confinement to attain prescribed deformabilities. It has also been found that high strength reinforcement (with yield strength of 600 MPa) confines high strength concrete columns effectively[19].

Deficiency of ACI and IS code formulae

It has been shown recently that the formula suggested by the American code Equation (3) and (4) do not provide good correlation with experimental data, producing over-conservative quantities of transverse reinforcement for spirally reinforced circular columns[19]. Similarly for rectangular and square columns

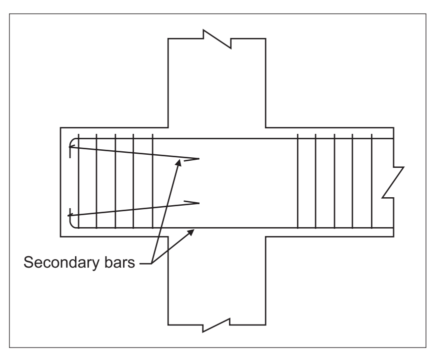

Fig 18 Reinforcement detail at an external joint with stub beam[17]

also the correlation with experimental results was poor[19]. The columns with the same amount and spacing of confinement reinforcement showed significantly different strength and deformability when confined by different arrangements of transverse reinforcement[19]. While a square column with four corner bars and tied with perimeter hoops showed the worst behaviour, columns confined with well distributed longitudinal reinforcement, laterally supported by cross-ties, overlapping hoops, or both, showed significantly improved performance.

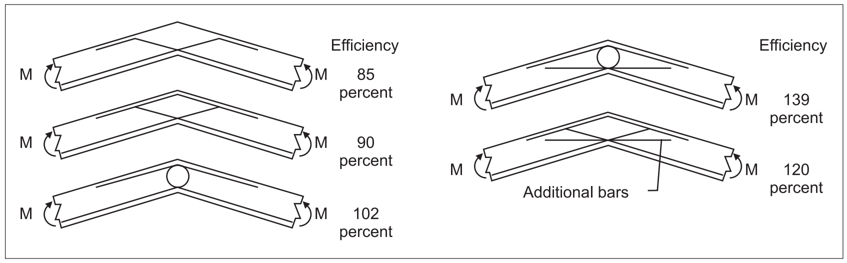

Fig 19 Reinforcement details tested by Wahab and Ali[12]

Based on these observation, Saatchioglu and Razvi[19] proposed the following equation which takes into account the tie spacings as well as the spacing of cross-reinforcement in the cross-section plane, s_l

$$A_{sh} = 0.16 s h_c f'_c\,(f_{yh} k_2)\left(\frac{A_g}{A_c} - 1\right) \tag{18}$$

Where,

$$k_2 = 0.15\sqrt{b_c^2/(s\,s_l)} \le 1 \tag{18a}$$

with $(A_g / A_c - 1) \ge 0.3$

where,

b_c = core dimension, centre-centre of perimeter ties

s = centre to centre spacing of transverse reinforcement along column height

s_l = centre-to centre spacing of longitudinal reinforcement, laterally supported by corner of hoop or hook of cross-tie

The above formula is found to correlate with test results of both high strength and normal strength concrete circular column as well as square columns. Hence the authors propose this equation for the Indian Code.

Note that for a displacement based design, the effect of axial force should also be considered as given in Equation 5.

Obtuse angled and acute angled corners

Corners with obtuse and acute angles occur in bridge abutments between the wing walls and the front wall and in folded plate roof. Tests on V shaped beams with 135^o to 145^o corners have been conducted by Nilsson and Losberg[11] and Wahab and Ali[12] for various reinforcement details as shown in *Fig* 19. Based on these investigations they concluded the following.

(i) The efficiency of the joint detail is improved when inclined bars are added to take up the tensile force at the inner corner. Loops with inclined bars *Fig* 19 (d) are preferable for continuous corners between lightly reinforced slabs (since their efficiency is of the order of 80 to 130 percent).

(ii) The efficiency of corners improved significantly when the thickness of the adjoining members were different. (The efficiency increased to 197 percent when the thickness of one leg was increased from 100 to 300 mm). Further, the mode of failure changed from diagonal tensile failure to flexural failure.

(iii) The efficiency of the corner increased by about 32 percent when the length ratio of the two legs was changed from 1 to 2.

Similar conclusions are reported[3]. The main reinforcement should be restricted to about 0.65 to 1.0 percent of the section in order to avoid brittle failure of the corner[10]. If the area of inclined reinforcement, A_{se} is the same as that of main reinforcement, the main reinforcement percentage may be increased to about 0.8 to 1.2 percent. If the reinforcement percentage is higher than this, the corner must be provided with a reinforced haunch and stirrups[11].

Nilsson and Losberg also tested 60^o acute angle specimens and observed that the failure had the same characteristics as in the tests with 90^o and 135^o corners[11]. Based on their tests, they suggested a reinforcement layout shown in *Fig* 20. The inclined reinforcement in acute angled corner is laid in a haunch. The length of this haunch is at least one half the thickness of the wing wall and the reinforcement is less than 0.5 to 0.75 percent, and at least equal to the thickness of wing wall when it is about 0.8 to 1.2 percent. The bars must not be spliced in the corner region. Further, recesses or openings should not be made at or in the immediate vicinity of corners or joints, since they reduce the strength and stiffness of the connection considerably.

Summary and conclusions

Structural joints in a rigid frame should be capable of sustaining forces higher than those of the connecting members.

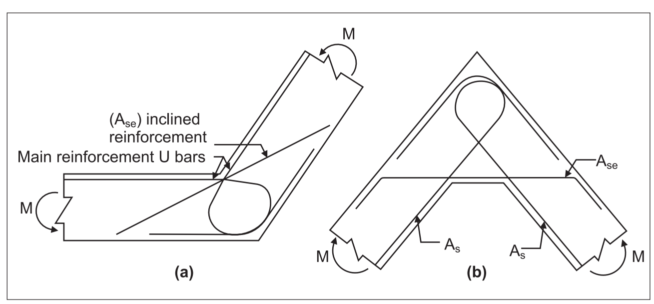

Fig 20 Reinforcement details for obtuse and acute corners[11]

However, while beams and columns are designed and detailed with considerable care, the same cannot be said about the joints in RC rigid frames. The structural behaviour will be different from that assumed in the analysis and design, if the joints are incapable of sustaining the forces and deformations induced due to the transfer of forces among the members meeting at the joint.

Some of the common incorrect design and construction practices of beam-column joints are explained along with possible remedies. Various types of joints, their behaviour under lateral forces are discussed; experimental results are also appraised. Structural joints in frames and slabs are included.

Especially, opening of joints has to be considered properly since it will result in diagonal cracking of the joint. Such opening of joints occurs in multi-storeyed structures due to lateral loads such as wind or earthquake. The discussions presented pertain to seismic forces, but are of general nature and can be applied to structures subjected to lateral forces. The behaviour of various types of joints is discussed in this paper, along with design and detailing aspects. An equation for calculating the area of transverse reinforcement in the joint has been proposed based on recent research. This equation is applicable to both normal strength and high strength concretes. A comparison of the codal equations show that the values found by Indian and ACI codes are unsafe and there is an urgent need to modify the Indian code provisions.

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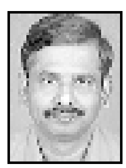

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