Discussion Forum

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

Mr. Indrajit Barua writes

This has reference to the paper titled 'Proposed codal provisions for design and detailing of beam-column joints in seismic regions' by Sudhir K. Jain, R.K. Ingle and Goutam Mondal published in the August 2006 issue of your journal. The above paper is timely and pertinent. I have done some calculations with the data given in the authors' examples.

However, for computing the 'flexural strength ratio', I feel that taking $P_u = 0$ for computing $M_u/(f_{ck}bD^2)$ for the column is overly conservative. We may perhaps use the minimum 'P_u' for the purpose, which could be the least 'P_u', without any LL, resulting from frame analysis, factored by 0.8 for further safety.

Secondly, I feel that the computation for tensile force in the rebars is also rather conservative. Recall that the area of steel required/provided is computed on the principle that the maximum stress in the rebars at limit state of collapse cannot exceed $0.87f_y$, granted that in laboratory conditions the ultimate strength of HYSD bars may have been found to be $1.27f_y$. Moreover, when the stress in the steel is around $1.25f_y$, the corresponding stress in the concrete could be very much higher than 'f_{ck}' and the concrete itself may fail much before the steel does and this mode of failure is not desirable. Why, then use a factor of 1.25 ? We may perhaps therefore take a factor of 1.0 instead of 1.25 or is the factor of 1.25 taken to cover the fact that the codal provisions for computing seismic forces are low and we need to provide for forces higher than those anticipated by our codes ? In that case, something has to be done about preventing premature failure of concrete before the steel yields.

In almost all multistoreyed buildings designed by my firm, shear walls or diagonal bracings are provided, and the demand on the MR frames, as well as the steel in the beams and columns, are vastly reduced thereby. Even so, it will be good to check the beam-column joints on the lines proposed by the author.

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Mr. D.S. Joshi writes

This has a reference to the above paper. The seismic region should mean the areas where the expected intensity of earthquake ground acceleration is much greater than $0.1g\ cm/s^2$ and not artificially upgraded zones merely due to panic or other reasons (that is, Zone IV and V). For other areas less stringent provisions can be made.

It is true, that design and detailing provisions in beam-column joints in IS 13920 : 1993 do not adequately address prevention of anchorage and shear failure during severe earthquake shaking.

Clause 1

It is not understood as to after giving a few important reasons out of many, for increasing the sizes of the columns and importance of joint design and also their provisions in ACI and New Zealand codes. Why is it proposed that the minimum dimension of column to be not less than 15 times the largest beam bar diameter of the longitudinal reinforcement in the beam passing through or anchoring into the column joint, on non conservative side without giving any valid technical reasons, when it is clear that the values given in ACI code are the critical values as the development length of the longitudinal bars in compression govern the size of the column along the length of the bar. Research has shown that, straight beam bars may slip within the beam column joint during a series of large moment reversals. The bond stresses on these straight bars may be very large. To substantially reduce slip during the formation of adjacent beam hinging, it would be necessary to have a ratio of column dimension to bar diameter of approximately 32, which would result in very large joints. On reviewing the available test data, the limit of 20 was chosen by ACI by accepting inevitable slip.

Clause 1.1

The sum of the moments of resistance of the columns are proposed to be at least 1.1 times the sum of the moment of resistance of the beams along each principal plane of the joint as against 1.2 proposed by ACI.

It is well known that, because of the disproportionate distribution of the moments around column beam joint during the higher modes of response of a multi storeyed frame, the bending moments at the critical sections, considerably larger than those derived from static analysis could result.

The intent of this clause is to reduce the likelihood of yielding in columns that are considered as part of the lateral force resisting system. If columns are not stronger than beams framing into a joint, there is likelihood of inelastic action. In the worst case of weak columns flexural yielding can occur at both ends of all columns, in a given story, resulting in a column failure sway mechanism of the structure.

We design our structures for only a fraction of the actual seismic force to which they are really expected to be subjected to, as we take in to account the benefits of over-strength, redundancies and ductility while determining the design forces.

The proposal of bigger dimension of the column requires lesser steel for resisting same combination of forces.

This provision of 20 percent higher moments is a good provision from the point of strong column weak beam design concept to achieve formation of hinges at the joints in the beam portion only, which is not suggested in IS 13920 : 1993.

This clause helps in increasing the stiffness of the frame which ultimately helps in reducing the drift.

Clause 1.3.1

It is apparent that, diagonal tension and compression stresses are induced in the panel zone of the joint. The diagonal tension may be high, when the ultimate capacity of the adjoining members is developed, and this can lead to, extensive diagonal cracking. The severity of diagonal tension is influenced by flexural steel content and the magnitude of the axial compression load on the column.

The strength of the diagonal strut controls the joint strength, when the joint shear forces are large and diagonal cracking occurs in the joint core.

There are in fact several struts, separated from one another by diagonal cracks. Not only are they subjected to indeterminable eccentricities, they are also exposed to transverse tensile strains. In this biaxial state of stress, a considerable reduction of strength occurs. Cyclic loading in cross-cracked concrete causes a repeated opening and closure of cracks. Because of the dominance of the shearing action across the joint, movements parallel to open cracks also occur in the joint core.

The joint reinforcement does not play a major role in such situation. It is therefore necessary to limit the magnitude of horizontal joint shear stress to protect the joint against diagonal crushing.

When the cracks become large, the transverse reinforcement yields and the process of grinding and progressive splitting due to uneven concrete bearing begins. The transverse reinforcement can resist shears only at this stage.

However, the hysteretic response of the joint core shows severe pinching. A complete disintegration of the concrete within the body of the joint can result. This is associated with drastic volumetric increase of the core unless confinement is provided. This is how the confinement reinforcement is required to be provided in joint portion also.

When the frame has deep and strong columns, the joint shear stresses become small and as a result the problem of diagonal cracking in the joint core is completely eliminated. Thus, the physical size or volume of the joint becomes the most important parameter because, it not only directly controls the level of stress in compressed diagonal concrete but also dictates how much transverse steel in each direction can be provided.

Further, the stiffness of the joint, which determines the contribution of the joint deformation to overall frame deformation, is also proportional to the volume of the joint.

The nominal shear strength of the joint as proposed by ACI, is not be

Table 3: Mechanical properties of high strength deformed bars and wires+

No.	*Property*	*Fe 415*	*Fe 415 D*	*Fe 500*	*Fe 500D*	*Fe 550*	*Fe 550 D*	*Fe 600*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
i)	0.2 percent proof stress / yield stress, Min N/mm^2	415.0	415.0	500.0	500.0	550.0	550.0	600.0
ii)	Elongation, percent, Min, on guage length 5.65√A where A is the cross-sectional area of the test piece	14.5	14.5	12	14.5	8	14.5	11
iii)	Tensile strength, Min	10 percent more than the actual 0.2 percent proof stress but not less than 485.0 N/mm^2	12 percent more than the actual 0.2 percent proof stress but not less than 485.0 N/mm^2	8 percent more than the actual 0.2 percent proof stress but not less than 545.0 N/mm^2	10 percent more than the actual 0.2 percent proof stress but not less than 565.0 N/mm^2	6 percent more than the actual 0.2 percent proof stress but not less than 585.0 N/mm^2	8 percent more than the actual 0.2 percent proof stress but not less than 600.0 N/mm^2	8 percent more than the actual 0.2 percent proof stress but not less than 660.0 N/mm^2
iv)	Uniform elongation, percent, Min, on guage length 5.65 √A where A is the cross-sectional area of the test piece*	-	5.0 percent	-	5.0 percent	-	5.0 percent	-

+Table from revised IS 1786

*Uniform elongation, for the enhanced ductility category, must be measured and reported in the test certificate but shall not be a criterion for rejection

greater than 1.5 $A_j\sqrt{f_{ck}}$ for joints confined on all faces, 1.125 $A_j\sqrt{f_{ck}}$ for joints confined on three faces or two opposite faces and 0.9 $A_j\sqrt{f_{ck}}$ for others, where A_j is effective area of the joint for resisting shear and f_{ck} is the characteristic compressive strength of concrete cube in MPa at 28 days, whereas the authors have proposed, 1.5 $A_j\sqrt{f_{ck}}$, 1.2 $A_j\sqrt{f_{ck}}$ and 1 $A_j\sqrt{f_{ck}}$ respectively.

It is not understood as to why the authors have proposed higher stresses on non conservative side than those permitted by ACI.

Clause 1.3.2

The effective width of the joint, as proposed by ACI, is smaller of ,

> Min [b_c ; b_b + h_c] if b_c > b_b for concentric and coaxial arrangement of configuration for beam and column, and Min [b_c ; b_b + 2x] if $b_c > b_b$ where x is the smaller distance from the edge of the column in case of non concentric arrangement of configuration for beam.
>
> A_j is given by product of effective joint width b_c and joint depth h_c

The significant reason, as to why the dimension h_c is replaced by 0.5 h_c for the purpose of calculating the effective shear area of the joint is not understood.

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 the steel.

In earthquake resistant reinforced concrete structure, it is important to use good quality concrete and steel for their proper performance.

Draft Code Doc: CED 54(7303), March 2005 published by Bureau Of Indian Standards Draft Specification for High Strength Deformed Steel Bars and Wires for concrete Reinforcement, fourth revision of IS 1786 in its *Table* 3 (shown above) shows mechanical properties of high strength deformed bars and wires.

In the interest of safety of earthquake resistant reinforced concrete structures this needs to be corrected as follows.

1. Upper limit on variation in yield stress – Max 20 percent.

   Fe 415 – 498 MPa

   Fe 500 – 600 MPa
2. Ratio of ultimate stress to yield stress should be at least 1.25.
3. Minimum total percentage elongation - 18 percent and minimum uniform elongation percentage - 10 percent.
4. Fe 550 and Fe 600 Grade Steel need not be used.

One of the reasons for collapse of buildings during Gujarat earthquake 2001 was the use of substandard steel in construction of reinforced concrete structures.

Solved example

Looking to Figure 5, only the columns marked C3 on grid B-2 and grid B-5, are subjected to uniaxial moments, where as all other columns will be subjected to biaxial moments, irrespective of the direction of the action of earthquake force. It was therefore necessary as a general case, to solve the columns for biaxial bending considering minimum eccentricity and slenderness as required by IS 456 : 2000 and also considering the effect of factored axial force including thrust and 20 percent increased moments (the flexural strength of beams framing into the joint in the direction considered) as the lowest flexural strength of the columns meeting at a joint should be 20 percent higher than the flexural strength of the beams framing in the joint in the respective direction of earthquake considered. It is known that the presence of moment in one direction tends to reduce the flexural strength of the column in the other direction. The axial force can become negative in certain circumstances.

Figure 7

Column dimension is 500 mm x 400 mm. The provision of links is correct as per ACI 318, but will require to provide at least one central link as it narrowly escapes provision of three vertical links as per IS 456 : 2000 clause 26.5.3.2.(b) and *Fig* 8. Also, as per more lenient IRC 21 : 2000, clause 306.3.1, one vertical link will be required to tie central bar leaving adjacent bars free. It is not understood as to which code we should follow here in India.

Confining Links

The spacing of links for the confining zone shall not be less than 75 mm nor more than 100 mm (clause 7.4.6 of IS 13920 : 1993). In the calculations of confinement reinforcement for the arrangement of reinforcement of Fig. 10, using rectangular hoops of 8 mm diameter, the spacing of hoops is worked out as 65 mm, which is less than 75 mm.

Here, 8 mm diameter confining links at 75 mm centres in the joint is proposed to be provided. Possibly, this is not the meaning of above clause. This reinforcement either could have been 10 mm diameter at 75 mm centres or 10 mm diameters at 100 mm centres to account for the area to be provided as required by the calculations.

'Design of reinforced concrete structures for earthquake resistance' authored by D.S. Joshi *et al* and published by Indian Society of Structural Engineers, Mumbai (2001), contains complete explanation of all the clauses of IS 13920 : 1993 discussed above and also suggests the required changes to be made in IS provisions taking into consideration the provisions in various international codes available throughout the world and gives a step by step procedure to design earthquake resistant dual system frame which includes the calculations for columns subjected to biaxial bending and design of internal and external joints.

This discussion is made to understand the views of the authors of the article on the subject.

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The author replies:

We would like to thank Mr. Barua and Mr. Joshi for their interest on design and detailing of beam-column joint provisions proposed in this paper.

Reply to Mr. Barua's queries:

We agree with Mr. Barua's comment that for calculating the flexural strength ratio, it is conservative to calculate the moment capacity of column corresponding to zero axial load ($P_u = 0$). We have in fact mentioned in the paper, "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." Since, we have not calculated different load combinations for the example problem in this article, we have conservatively taken the value of $\frac{M_u}{f_{ck}bD^2}$ corresponding to $\frac{P_u}{f_{ck}bD} = 0.0$ for the purpose of this example.

In earthquake resistant design of structures ductile failure of member is desirable rather than brittle failure. Failure of steel is ductile and that of concrete is brittle. Brittle failure of beam-column joint is not desirable. Therefore, during severe earthquake shaking, stress in steel should reach tensile strength before compressive stress in concrete reaches the compressive strength value (f_{ck}). Hence, the tensile stress in the reinforcement is conservatively taken as $1.25f_y$ 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. When the tensile stress of HYSD bar reaches $1.25f_y$, corresponding compressive stress in concrete should be smaller than the compressive strength of concrete (f_{ck}). Hence, section size and concrete grade should be such that the maximum stress in concrete corresponding to ultimate strength of steel ($1.25 f_y$) is less than f_{ck}.

We indeed appreciate Mr Barua very much for providing shear walls or diagonal bracings in his projects in north-east India for long years. What he has done in Guwahati needs to be emulated in other cities. It is not easy to provide ductility to a moment resisting frame and enormous effort in design and construction is required for the same. It is much easier to make a safe building using shear walls. We hope many more engineers will go by the example of his professional practice.

Response to Mr. Joshi's queries:

Seismic provisions on beam-column joints tend to vary widely from code to code. An overview of beam-column joint provisions in some codes of different countries is available elsewhere[1]. Considering the large variation in codal provisions in different seismic countries, we need not follow the ACI exactly. In fact, the proposed effective width of joint (clause 1.3.2) is in line with New Zealand seismic code (NZS 3101: 1995) rather than with ACI provisions.[2]

We agree with the concern of Mr. Joshi on high strength deformed steel bars and therefore, we have proposed some changes in IS :13920 (clause 5.3)[3]. Moreover, these issues have also been discussed thoroughly in an e-conference on 'Steel Reinforcement' by the Structural Engineering Forum of India (http://

www.sefindia.org/)[4]. Since these issues are beyond the scope of the present paper, we are not discussing them here.

The example problem is meant to explain the proposed clauses on beam-column joint. Hence, a simple beam-column joint C-3 in *Fig* 5 in the example building has been chosen to illustrate the same. One could have chosen another joint.

There is a variation in the provisions of lateral ties in column sections in different codes, for example, IS 456 : 2000, IS 13920 : 1993 and IRC 21 : 1987[5,6,7]. These need to be reconciled by the different code committees. It may be noted that clause 5.1 of IS 13920 : 1993 states "The design and construction of reinforced concrete buildings shall be governed by the provisions of IS 456 : 1978, except as modified by the provisions of this code." Hence, the ties in columns for RC buildings should be governed by IS 13920 rather than by IS 456. On the other hand, IRC 21 : 1987 deals with concrete road bridges and is yet to evolve its own ductile provisions for RC bridges and hence IS 13920 may be used until IRC develops its own provisions.

Column section in *Fig* 7 was a trial section and its size did not meet the shear requirement. Therefore, we have not detailed the confining links of this trial section and revise the section size to that of *Fig* 10. In the revised section *Fig* 10, the confining links are designed and detailed as per IS 13920 : 1993.

We agree with Mr. Joshi that we should use 10 mm diameter hoops with 100 mm centre to centre to meet the requirement of clause 7.4.6 of IS 13920 : 1993.

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