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DUCTILITY REQUIREMENTS IN INDIAN CODES FOR ASEISMIC DESIGN OF R. C. FRAME STRUCTURES : A REVIEW

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ABSTRACT

In this paper ductility provisions of IS codes have been thoroughly reviewed and compared with those in American codes. The provisions needing revisions, inclusions or clarifications have been identified. Suggestions made include rationalization of performance factor, providing of ductility detailing as per zones rather that α_h, introduction of strong column—weak girder concept in design, revision of minimum reinforcement requirements, etc. A method has also been suggested for determination of plastic moment capacity for R.C. beams.

INTRODUCTION

Earthquake resistant design involves determination of expected seismic forces and designing the structural members to resist these forces. Bureau of Indian Standards has published two codes, IS: 1893-1984 (Ref. 1), which is primarily a load standard, specifying minimum seismic design loads for structures and, IS:4326-1976 (Ref. 2), which contains design standards, setting down requirements by which to proportion and detail members.

The seismic codes do not intend to ensure that no structure shall suffer damage during a large earthquake. For instance, IS:1893-1984 mentions that, "It has been endeavoured to ensure that, as far as possible, structures are able to respond, without structural damage to shocks of moderate intensities and without total collapse to shocks of heavy intensities." This is because a structure which can withstand strongest ground shaking without damage will be too expensive to build. Hence, it is obvious that non-linear behaviour of structure, i.e., beyond its yield, will greatly affect its seismic design. It is, therefore,

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important that structures should be more ductile for better performance during earthquakes. Ductility of a structure means capacity to deform to a large extent without loss of strength before collapse, as compared to its deformation at yield point.

Seismic codes around the world ensure adequate ductility of a structure in two ways. Firstly, design seismic forces for a ductile structure are less than those, for a brittle structure. Secondly, it is required that the structures to be built in a highly seismic zone must have a minimum level of ductility. In this paper, IS code provisions for ductility requirements have been thoroughly reviewed and compared with those in American codes. An attempt has been made to identify the areas which are implicit, non-existent or confusing and need to be revised or included in the future revisions of the Indian codes. Also included are the authors' view points on some of the clauses needing thorough study and revision by the Earthquake Engineering Sectional Committee before bringing out the future editions of the codes.

DUCTILITY

Ductility is one of the most important requirements of earthquake resistant design. As per Ref. (12) "High ductility is the ability of a building to sustain large deflections without failure or collapse." It is impractical to expect the structure to respond to a very strong ground shaking within its elastic range. Hence, the structure is allowed to go beyond its yield point due to strong ground motion. Ductility implies that the structure will sustain fairly large deflection beyond its yield before it collapses. This results in reduced earthquake forces experienced by the structure. In the post-yield range, the structure exhibits significant hysteretic damping and this energy dissipation reduces response. Besides, the yielding leads to softening of the structure which increases the time period of structure and it usually means further decrease in response.

Again, when the plastic hinges tend to develop, stresses are transferred elsewhere, to member sections whose energy capacity and absorption have not been fully utilized. Thus the whole structure tends to offer resistance in severe emergencies and it is not limited to one weak section of a member in the elastic range (Ref. 12).

Research has established (e.g., Ref. 34) that it is reasonable to assume that deflections produced by a given earthquake input to be essentially the same, whether a structure responds elastically or yields significantly. If the member force-deformation relation shown in Fig. (1) is considered, the maximum deflection δ_{max} developed in the member is same regardless of its strength property. The ratio of the maximum deformation to the elastic-limit deformation is equal to the ratio of the force developed in purely elastic response to the member yield force, that is

$$\frac{\delta_{max}}{\delta_y} = \frac{f_{max}}{f_y} \qquad \ldots \ldots \ldots (1)$$

The ductility factor μ of the member is

$$\mu = \frac{\delta_{max}}{\delta_y} \qquad \ldots \ldots \ldots (2)$$

i. e.,

$$f_y = \frac{1}{\mu} f_{max} \qquad \ldots \ldots \ldots (3)$$

Equation (3) clearly demonstrates that the design forces of members are reduced with increase in the ductility factor, indicating that the ductility of the members can be advantageously used to reduce member design forces.

Ductility of the structure depends on ductility of the individual members, but there is no way to establish a direct correlation between the two. Some of the important aspects of ductility are :

(a) Axial load in members reduces ductility at column ends, the larger the axial stress, the larger the reduction (e. g., Ref. 35), suggesting that the plastic hinges occur at the ends of beams rather than in columns. Hence, strong column - weak girder concept of design of structures improves ductility.

(b) A flexural member exhibits large ductility before collapse if it undergoes tension failure. Thus by providing under-reinforced sections and providing limits on minimum and maximum reinforcements ductility may be improved substantially.

(c) The formation of plastic hinges involves large rotations in the members. If the failure of a member in diagonal shear is avoided before formation of plastic hinges, the member will be able to develop full curvature and will behave in a ductile manner.

(d) In reality concrete itself is not a ductile material, but if it is reinforced properly and confined by closely spaced transverse steel at proper locations, the combination will behave like a ductile material. Recognizing this, the building codes (Refs. 2, 6) specify the detailing for concrete members to render them ductile.

DUCTILITY PROVISIONS IN INDIAN CODES :

Performance Factor :

In the earlier editions of IS : 1893, in design seismic force calculation, no distinction was made between ductile and brittle structures. However, the 1984 edition has introduced "Performance Factor", K, for buildings which ensures higher design forces for brittle buildings. The values of K have been specified in Table 5 of the code and reproduced in Table 1 of this paper. It must be mentioned that prior of 1984 edition, in the absence of K, it can be taken as 1.0. Thus, for structures not detailed for ductility as per IS:4326, now there is an increase in seismic design force to the extent of 60 percent over what existed prior to 1984 edition, irrespective of the seismic zone.

Ductility Details in Reinforced Concrete Construction :

IS : 4326 gives detailing requirements to ensure proper ductility in the structure. These provisions *shall have to be adopted in all* cases where design seismic coefficient α_h is 0.05 or more. The requirements of IS:4326 for flexural members have been shown in Figure 2. Similarly, the requirements of IS:4326 for columns subjected to axial load and bending are shown in Figure 3. The transverse reinforcement as required at the end of columns (Fig. 3) is to be provided through beam column joints, subjected to fifty percent reduction if the connection is confined by beams from all four sides.

DISCUSSION ON CODE PROVISIONS FOR DUCTILITY :

While most of the provisions in IS : 4326 are similar to those of

contemporary codes elsewhere, many of the recommendations are implicit and incomplete. Since the first revision of IS:4326 in 1976, a lot of research and experimental data (Refs. 19 to 33) have accrued and a second revision of the code has been long felt. With the possible advent of the second revision shortly, full advantage should be taken to thoroughly revise the present form of the code to make it a self sufficient and *explicitly documented code of efficient and economical* practice for earthquake resistant design of structures. A well documented code could also relieve the designer of bureaucratic pressures to reduce the cost of structure and will enable him to provide sufficient protection to the structure from probable earthquake forces.

In the following paras, the codal provisions have been discussed at length. One of the publications of American Concrete Institute, ACI:318-83 (Ref. 6) is a building code giving requirements for reinforced concrete, similar to IS:456. Its Appendix:A describes provisions for seismic design, as in Clause (7) of IS:4326. SEAOC Code (Ref. 15), ANSI-1982 (Ref. 9) and UBC-1985 (Ref. 8) recommend minimum design seismic loads for buildings and other structures, which include values for horizontal force factor, K. This factor has the same meaning as performance factor of IS : 1893. IS Code provisions have been compared with provisions followed in the U.S., where research in earthquake resistant design is in much more advanced stage than in our country- Also listed are some suggestions for inclusions.

Performance Factor :

IS:4326 requires that the ductility requirements specified therein "shall be adopted in all cases where the design seismic coefficient α_h is 0.05 or more." *Moreover, Clause (1.2) of the same* code states, "The provisions of this standard are applicable for building construction in seismic zones III to V. No special provisions are necessary for building constructions in seismic zones I and II. "However, IS : 1893 - 1984 does not allow this distinction and even in seismic zones I and II increases seismic forces by 60% for non-ductile R.C. frames. Thus, there is an undue penalty which adds to the cost of construction even in areas which are seismically inactive. It is not reasonable to expect buildings in zones I and II to conform to ductile detailing at the same level as in zones IV and V. Thus it is surely a rather steep increase (of 60%) in design seismic forces for structures in low seismic zones. The code must be modified to remove this anamoly.

Even in seismically more active zones (III to V), one must carry out a thorough study on cost implications of the new provisions. For instance, a designer should have some idea of the order of magnitude of costs if he designs say a 4-storey R.C. frame office/residential building in zone III as (i) ductile frame with K=1.0; or as (ii) non-ductile frame with K=1.6. At present he does not have any such guidelines. Moreover the authors personal experience is that large number of designers in the country still do not design structures a ductile. Interaction with some designers in private as well as public sector indicates that even two years after the release of IS:1893-1984 (it was released in later part of 1986), they continue to use IS:1893-1975, thereby avoiding K=1.6 for non-ductile frames. This tendency has been encouraged because of steep rise in the cost of the same building in the same zone if new provisions are used. There are bureaucratic pressures on designers to reduce cost of construction and they find it difficult to justify this steep rise instead.

Another difficulty with inclusion of performance factor, K, is that as yet no provisions for ductile detailing of R.C. shear walls or steel bracing members are available in Indian codes, while IS:1893-1984 specifies K for such structures. Also, most multistorey construction in the country consists of R.C. frames with brick in-fill panels wherein the brick in-fills are assumed to be non-structural. Also, there is not enough experimental data available for brick in-fill and frame interaction and hence no guidelines are available on the lateral load response of such in-fills. However, the code seems to allow for incoporation of the structural contribution of such in-fills. This may lead to misinterpretation and improper use by the designer.

The provision of performance factor for buildings and not for other structures has created a very obvious anamoly. In the absence of K for other structures (e.g., water tanks, chimneys, bridges, etc.), it amounts to taking K=1.0 for such structures which is the same as for a building with ductile frames. However, such structures will not exhibit the same ductility as a ductile building, especially when there is no requirement for ductility detailing in other structures. In fact American codes specify a much higher value of K for other structures

because "most of these other structures do not have the multiplicity of structural and non-structural resisting elements characteristics of most buildings; do not have significant natural damping; do not have elements which could be permitted to yield or even fail without jeopardising the safety of the structure" (Ref. 15). For instance, American codes use K=2.0 for structures other than buildings as compared to K=0.67 for buildings with a ductile frame, i.e., three times that for ductile frames. On the other hand, as per IS:1893, other structures have the same value of K as a ductile frame building. This anamoly needs to be removed.

Ductility Detailing Criteria :

IS 4326 requires ductile detailing if α_h is 0.05 or more. This leads to difficulty in application. This is because, in the same city, depending on β and I, the value of α_h will vary. It is difficult to practice different detailing requirements in the same city by builders and engineers. Instead, if this requirement could be based on seismic zone alone, in due course a detailing culture will evolve in each geographical area.

The design provisions contained in the main body of the ACI building code provide some ductility which is sufficient for structures subjected to only minor earthquakes that may occur frequently. For structures that may be subjected to earthquakes of moderate intensities some additional confinement, anchorage and shear reinforcement details are required. For structures that may be subjected to strong intensity earthquakes, appreciable inelastic deformations can be expected so that substantial ductility is required. Special provisions in Appendix:A are intended to provide the additional ductility (Ref. 13). Hence depending on the seismic risk of the zone, three levels of ductility are adopted by ACI, for any reinforced concrete construction. The main body of ACI covers ductility requirements for zones of low seismic risk (equivalent to zones I and II in India) and ACI Appendix:A contains different provisions for zones of moderate seismic risk (equivalent to our zone III) and zones of high seismic risk (equivalent to our zones IV and V).

If we see the provisions of IS codes from this angle of view, prior to 1984, the IS:1893 in effect had performance factor, K, equal to unity and the detailing had to be adopted depending on α_h with the fourth edition of IS:1893 in 1984, performance factor, K, has been troduced, which gave the designer two broad options. One is to rovide ordinary concrete frames but with enhanced lateral loads and econd to provide ductile concrete frames with lower value of lateral oads. The ductility detailing is to be adopted only for the latter case. This option could allow a designer to provide ordinary concrete frames ven in zone V, which is quite contrary to the essence of IS 4326:1976. Again, structures designed to be built in zones IV and V will be xtremely uneconomical if ordinary concrete frames are used with nhanced lateral loads. It would be very appropriate to include the hree levels of ductility requirements for Indian conditions also. The alanced percentage of steel reinforcement as calculated by ACI code or flexural members is much more than that calculated by IS:456, for ame grade of steel and concrete. That means ACI defines the alanced condition for the flexural sections with more steel ratio than hat defined in IS:456 and yet, its provisions of main body are said to provide sufficientt ductility in zones of low seismic risk. Generally for economy, the designs of members are aimed to be done with steel ratios close to balanced steel ratios. Hence, members designed with IS:456 provisions are likely to have less steel ratios and better ductility as compared to sections designed by ACI code. Hence, for zones of low seismic risk (zones I and II) it can be safely said that provisions of IS:456 will provide sufficient ductility. The other two levels of ductility for zones of moderate and high seismic risk are to be included in our codes.

Materials :

There are no provisions in the code for maximum grade of steel and minimum grade of concrete. Minimum grade of concrete of M20 may be considered to be included in zones IV and V. IS:456 mentions the use of high yield strength deformed bars of grade Fe500, but these are not commonly used in India. However a maximum grade of steel of Fe415 may be considered to be specified. Limitation on maximum variation of actual yield strength of longitudinal steel provided in the structure, to that specified by the designer may also be recommended. If the variation is large, shear in the flexural members at the time of formation of plastic hinges will be very high and may cause a brittle shear failure These limits are necessary in view of the unfavourable effects, the decrease in concrete strength and increase in yield strength of steel have on the sectional ductility of members in which they are used (e.g., Refs. 13, 14).

Strong Column-Weak Girder Design :

As mentioned earlier, it is important in earthquake resistant design that hinges must form in beams and not in columns. Hinge formation in columns leads to early collapse of the structure. To reduce likelihood of yielding in columns, ACI 318-83 requires the flexural strength of columns (in regions of high seismic risk) to satisfy.

$$\Sigma M_e \geq (6/5)\, \Sigma M_g \qquad \ldots \ldots \ldots (4)$$

where, ΣM_e = sum of moments, at the centre of the joint, corresponding to the design flexural strength of the columns framing into that joint. Column flexural strength to be calculated for the direction of the lateral forces considered, resulting in the lowest flexural strength.

and, ΣM_g = sum of moments, at the centre of the joint, corresponding to the design flexural strength of the girders framing into that joint.

However, Indian codes do not have any such provision. It is highly desirable to incorporate such a provision in IS:4326 for columns of buildings in zones IV and V.

Flexural Members :

Definition :

Members with average axial stress P/A under earthquake condition less than 0.1 F_c are to be treated as flexural members. ACI code also has a similar requirement. However, ACI code also specifies restrictions on the sectional dimensions which are not specified in Indian code. But none of the codes have minimum specified depth for a flexural member. A minimum depth of beam is desirable to be specified because, beams with depth (d) about 300mm (common in residential buildings) will have clear spacing between stirrups less than 75mm at the potential locations of plastic hinges, to satisfy the maximum spacing (d/4) requirement. This could cause constructional difficulties in placing and compacting the concrete.

(i i) Longitudinal Reinforcement :

The minimum longitudinal reinforcement recommendations given in IS:4326 (e.g., Fig. 2) are based on minimum ductility provisions as given in Ref. 12. However, ACI code recommends higher values for minimum longitudinal reinforcement for the same grade of concrete and ACI provisions seem more reasonable. The minimum steel ratios in ACI code were found by equating the cracking moment of the section of the plain concrete to the moment capacity of the reinforced concrete section and solving for steel ratio (Ref. 14). As a beam is loaded, initially the gross concrete section resists the moment while steel does not get stressed much. However, beyond cracking moment, the tension concret starts cracking. Immediately after the concrete cracks, section capacitye must not be less than what it was before cracking started otherwise the beam will fail suddenly in a brittle manner. Hence, it is recommended that the IS provisions for minimum longitudinal reinforcement be revised upwards following the same criteria as in ACI code.

The maximum tensile steel ratios recommended in Clause (7.2.2.) are from Ref. 12 and are dependent on grade of steel and concrete. ACI recommends a fixed value of 0.025, which is based on criterion of congestion of steel reinforcement.

Moment Capacity in the Member :

ACI recommends positive strength at joint face to be greater than one-half of the negative moment strength provided at that face of the joint. Throughout the member, the moment strength is to be greater than one fourth the maximum strength provided at the face of either joint. This provision is lacking in IS:4326. Since the earthquake loads are reversible in nature, inclusion of this clause may be considered.

Lap Splices :

Considering that the failure in bond between steel and concrete is brittle, and lap splices are not reliable under conditions of cyclic loading into the inelastic range, ACI recommends stringent provisions for locating lap splicings and enclosing them in transverse steel. IS code provisions are incomplete in this aspect.

Design Shear Force :

Avoiding shear failure is one of the most important requirements of earthquake resistant design. This is achieved in IS:4326 through Clause (7.2.5) which reads, "The web reinforcement in the form of vertical stirrups shall be provided so as to develop the vertical shears resulting from all ultimate vertical loads acting on the beam plus those

which can be produced by the plastic moment capacities at the ends of the beam. The spacings of the stirrups shall not exceed d/4 in a length equal to 2d near each end of the beam and d 2 in the remaining length."

The authors have found a lack of understanding of this clause among some design engineers. It was noticed that some of them did not quite follow the first sentence of this clause and assumed that the second sentence ensures compliance of requirement specified in the first sentence. Handbook SP:22 (Ref. 5) has explained some of these requirements. But the expressions given in SP:22 for minimum and maximum design shear forces incorrectly printed. The correct version is given in Fig. 4 of this paper. In conventional design, a beam is designed for moments and shears obtained from analysis for given loads. However, this clause requires shear design from a different viewpoint and is meant to ensure that the beam does not fail in shear before formation of plastic hinges. This is to avoid brittle shear failure. The code should bring this out much more clearly and should preferably separate these two sentences into separate clauses.

This clause for shear reinforcement design has two deficiencies. Firstly it does not account for the effect of strain hardening in longitudinal reinforcement. Research worldwide has established that the main steel in fact goes into its non-linear range during severe earthquakes and shear forces may increase to a large extent. Secondly, it is not clear how to calculate "plastic moment capacities at the end of the beam" as specified in IS:4326. For instance, what stress must be taken in steel and concrete at this condition. ACI code follows the ultimate strength philosophy of design, with appropriate strength reduction factors. So its simple reference to the plastic moment capacity as that corresponding to probable strength using the properties of the members at the joint faces without strength reduction factors and assuming that the stress in the tensile reinforcement is equal to at least 1.25 fy, is adequate. IS code may stipulate plastic moment capacity as that corresponding to γ_m' partial safety factor for material strength, equal to unity and stress in the tensile reinforcement equal to at least 1.25 fy.

Deterioration of shear strength of concrete owing to alternate opening and closing of cracks during repeated reversal of deformations in non-linear range (effect of shear sliding.) may also be taken into account by specifying zero shear strength capacity of concrete for large shear forces associated with the formation of plastic hinges at the ends as in ACI.

Handbook SP:22 on the other hand must contain clear figures (such as Fig. 4) of deformed shape of flexural members and the expressions for the design shear forces. It should contain actual procedure as outlined in Appendix: A and tables corresponding to plastic moment capacities of sections for different grades of steel and concrete similar to those given in SP:16 (Ref. 11) to enable the designer to use them directly without spending time on lengthy calculations.

Studies have revealed (Ref. 35) that, for moderately reinforced sections with slab live loads 3 to 5 KN per square metre, 8 mm tor steel provided at maximum spacing of d/4, almost always governs the design of transverse steel, i.e., the shear caused by plastic hinges does not govern in most cases, if the shear capacity of concrete in the section is taken into account.

The possibility of correlating the curves and tables already available in SP:16 with the plastic moment capacities of the sections is to be thoroughly explored to save designer's time.

Minimum Diameter of Transverse Steel :

It is also suggested that in zones IV and V, minimum bar diameter of 8 mm be specified for transverse reinforcement. This is because percentage reduction of the gross sectional area of 6 mm bars, due to rusting of the ribs and surface, upto placing of concrete is much more than that of bars of higher diameter. Hoops are required not only to provide shear strength, so that full flexural capacity of the member can be developed, but also to help ensure adequate rotation capacity at plastic hinging region by confining concrete in the compression steel. Thus, the improved ductile behaviour of the member due to higher diameter stirrups would be disproportionately large as compared to the negligible increase in the quantity of steel. It may be mentioned here that ACI provides for a minimum 3 bar (i.e., 10mm dia) to be used as transverse steel even in ordinary concrete frames.

Diagonal Shear Reinforcement :

Clause (7.1.4) of IS:4326 contains provisions for limiting the value of maximum shear carrying capacity of diagonal bars to 50% of the design shear. This is a provision similar to that given in British code CP:110 (Ref. 10). However, Britain is not seismically active. ACI does not allow even 50% contribution of diagonal bars. Keeping in view the possible shear reversal during an earthquake, disallowing the use of diagonal bars may be considered for zones IV and V.

COLUMNS SUBJECTED TO AXIAL LOADS AND BENDING

Definition :

Clause (7.3.1) defines "columns subjected to axial load and bending" if the member is subjected to average stress P/A greater than 0.1 F_c. IS code may consider reclassifying this group as "members subjected to axial load and bending", as members satisfying these conditions need not necessarily be columns. ACI code gives additional requirements on section sizes of members subjected to axial load and bending. Of particular importance is the one restricting shortest dimension of the cross-section measured on a straight line passing through the geometric centroid to be not less than 300 mm.

(ii) Confinement Reinforcement :

Clause (7.3.2) of IS:4326 specifies the minimum amount of confinement reinforcement for spiral and rectangular closed hoops. These expressions are comparable to the ACI recommendations. The only difference is because ACI refers to cylinder strength, while IS code refers to cube strength. For circular hoops or spirals used for confinement of concrete, IS code requires,

$$A_{sh} = 0.08\ s\ D_k \frac{F_c}{F_y}\left[\frac{A}{A_k} - 1.0\right] \qquad \text{... ... (5)}$$

On the other hand Clause (38.4.1) of IS:456 requires for all constructions the ratio of volume of helical reinforcement to the volume of the core (ρ_s) shall not be less than 0.36 $\left[\frac{A_g}{A_c} - 1.0\right]\frac{f_{ck}}{f_y}$

since $\rho_s = \dfrac{\pi D_k A_{sh}}{\frac{\pi}{4} D_k^2 s}$,

therefore $$A_{sh} = 0.09\, s\, D_k \frac{f_{ck}}{f_y}\left[\frac{A_g}{A_c} - 1.0\right] \quad \ldots\ \ldots\ \ldots\ (6)$$

Thus, IS:4326 requirement is less than that of IS:456 which is inconsistent. Hence, expression in IS:4326 can be either removed or its provision may be revised upwards to match that in IS:456. The requirement for confinement by the rectangular hoops is derived from this requirement for spiral reinforcement. Hence, for rectangular hoop reinforcement, corresponding value of cross sectional area in IS:4326 may be changed from

$$A_{sh} = 0.16\, s h \frac{f_{ck}}{f_y}\left[\frac{A_g}{A_c} - 1.0\right] \quad \ldots\ \ldots\ \ldots\ (7)$$

to. $$A_{sh} = 0.18\, s h \frac{f_{ck}}{f_y}\left[\frac{A_g}{A_c} - 1.0\right] \quad \ldots\ \ldots\ \ldots\ (8)$$

A minimum volumetric ratio of spiral or circular hoop reinforcement $\rho_s = 0.12 \frac{f_c}{f_{yh}}$ is recommended by ACI to specify a lower bound which governs for larger columns with gross cross sectional area, A_g, less than approximately 1.25 times the core area, A_c (Ref. 7). Here f_c is cylinder strength which may be related to cube strength as $f_{ck} = 0.85\, f_c$. IS code may incorporate this requirement as $\rho_s = 0.14 \frac{f_{ck}}{f_y}$ For rectangular hoops, ACI recommends minimum total cross-sectional area $A_{sh} = 0.12\, s h \frac{f_c}{f_{yh}}$ which may be adopted in IS:4326 as

$$A_{sh} = 0.14\, s\, h \frac{f_{ck}}{f_y}$$

Shear Reinforcement :

Clause (7.3.4) of IS:4326 requires provision of shear reinforcement to resist shear resulting from the lateral and vertical loads at ultimate load conditions of the frame and specifies a maximum spacing of d/2

throughout the member. This implies checking for shear reinforcement in addition to minimum confinement steel. However, it does not clearly mention what is meant by "ultimate load conditions". ACI code requires the design shear force, V_e, to be determined from consideration of the forces on the member, with the nominal moment strengths calculated for the factored axial compressive forces, resulting in the largest moment, acting at the face of the joints. It specifies the nominal moment strength as the limiting moment of the section with strength reduction factors equal to unity.

If such a requirement is to be specfied in IS code, the procedure to arrive at nominal moment capacity must be included. It may say that the nominal moment capacity is to be arrived at by setting γ_m, partial safety factor for materials, equal to one.

Development Length of Hoops :

In seismic design, the development length or anchorage of closed stirrups or hoops is usually more than that for ordinary concrete construction. The members rely on full development of yield strength in transverse steel for confinement of concrete and rotation capacity of the members. Providing closed hoops in beam-column joints also causes constructional difficulties. ACI allows hoops in two pieces (Fig. 5 a) with a stirrup having 135° bends and ten diameter extensions, anchored in the confined core and a crosstie to make a closed hoop. Such hoops are uncommon in Indian construction industry, but these are highly essential to enable the placement of transverse steel in joints. Further, IS:4326 does not have provision for extra extension beyond the bends in stirrups for ductile frame members. In Clause (25.2.2.4b) IS:456 requires continuation of stirrup or transverse steel for 8 diameters for 90° bend, 6 diameters for 135° bend or 4 diameters for 180° bend. On the other hand IS:4326 seems to require extension of 10 diameters for closed hoops and intermediate ties in column sections as implied from its Fig. 5, but it does not explicitly spell out this requirement. There is altogether no such provision for hoops in flexural members. In the absence of an explicit provision in this regard, the industry would continue using detailing as per IS:456. Hence this provision needs to be explicitly stipulated in the form of a clause.

Minimum Diameter of Transverse Steel :

For reasons listed earlier, a minimum of 8 mm diameter may be specified for hoops required for confinement. Transverse steel results in improvement of both strength and ductility alongwith prevention of buckling of longitudinal steel, keeps the core concrete confined and prevents shear failure from propagating through the section. Higher transverse reinforcement also helps in fully developing required ultimate curvature. Fig. 5 of IS:4326 shows the details of rectangular hoops to be provided, but it has a misprint showing the diameter of one of the hoops to be equal to 10d instead of d. The same has been rectified and shown in Fig. 5b.

Columns Supporting Reactions from Discontinuous Stiff Members :

ACI has additional provisions for columns supporting reactions from discontinuous stiff members, which are not present in IS code. Such provisions are highly desirable for zones IV and V in India.

Beam-Column Connections :

Clause (7.1.3) in IS: 4326 says that the beam-column connections shall preferably be made monolithic. However field and laboratory experience which has led to ductile detailing requirements has been predominantly with monolithic reinforced concrete building structures (Ref. 7). So, use of 'preferably' in this clause is not warranted.

IS code limits the amount of transverse reinforcement in beam-column connections to that required at the ends of the columns. ACI code requires that the shear capacity be checked as per the actual forces caused by taking into account the column shears and the shears developed from yield forces in beam reinforcement after reaching strain hardening state. It clearly quantifies shear strength in joints and gives requirements of lapping for beam reinforcement. IS code requirement of transverse reinforcement in joints as presently specified needs to be thoroughly reviewed and revised.

General :

ACI code also contains provisions for structural walls, diaphragms and trusses and provisions for frame members not proportioned to resist forces induced by earthquake motions which IS code is lacking.

Handbook SP:22, can play a very vital role in spreading the letter and spirit of IS:4326 in engineering profession in the country. It should encourage the designer to opt for ductile frames in zones IV and V and to design structures which are uniform in strength and ductility through-out rather than with individual and isolated over-strong members. In ordinary design for static loads, the presence of overstrong members does not decrease the strength of the structure. In seismic design, however, when structure relies on energy dissipation by ductile plastic hinges to survive earthquakes, the presence of overstrong members leads to collapse because of the very high inelastic deformations enforced elsewhere. The handbook should illustrate use of IS:4326 with examples and neat detailing methods. It should contain design tables and procedures for determination of plastic moment capacity and the ultimate moment capacity of sections.

It should also guide the designer to provide minimal reinforcement, which avoids steel congestion in joints and members, and also provides better ductility. The designer should also be encouraged to lavishly use transverse reinforcements in the frame members because, extra ties and hoops are inexpensive due to their low weight and minimal fabrication costs but their use can substantially better the performance both in ductility and confinement.

CONCLUSIONS :

A structure is expected to go into its inelastic range of response during a severe earthquake and so, needs large ductility for efficient dissipation of energy. Ductility provisions of IS codes have been reviewed and compared with those in ACI code. The following major suggestions have been made.

(i) Rationalization of performance factor, K, specified in IS:1893-1984

(ii) Introduction of ductility detailing depending on zones rather than α_h.

(iii) Introduction of three levels of ductility requirements, i.e., ductility detailing for zones of low, moderate and high seismic risks.

(iv) Inclusion of limits on strength of concrete and steel used in structures.

(v) Introduction of strong column-weak girder concept in the design of structures.

vi) Revision of minimum longitudinal reinforcement criteria, provisions for design of transverse reinforcement in flexural members, etc.

vii) Inclusion of guidelines for arriving at plastic moment capacity of sections.

viii) Modification of amount of confinement steel in columns.

ix) Inclusion of minimum diameter for transverse reinforcement in columns and beams.

x) Inclusion of provisions for design of beam-column joints specifying the design shear force on joints and the shear capacity of joints.

(xi) Inclusion of provisions for structural walls, diaphragms, trusses, etc.

(xii) Revision of SP:22 for clear explanation on seismic code provisions alongwith numerical illustrations and design tables for plastic moment capacity.

APPENDIX A

PROCEDURE FOR CALCULATION OF PLASTIC MOMENT CAPACITY FOR R.C. BEAMS

Steps for calculation of plastic moment capacity of reinforced concrete section have been given below. From any standard book (Refs. 16, 17) on design of reinforced concrete structures, following relations in accordance with IS:456 can be derived. The effect of strain hardening in tension steel has been incorporated in these relations. Taking $\varepsilon_c = 0.0035$,

$$\varepsilon_y = 0.002 + \frac{f_s}{\gamma_m . E}, \quad f_s = 1.25\, f_y,\ \gamma_m = 1.0,\ E = 2.0 \times 10^5 \text{ MPa},$$

for balanced failure,

$$x_{umax} = \frac{35d}{55 + 0.0625\, f_y} \qquad \dots \quad \dots \quad (1)$$

$$z_{max} = (d - 0.416\, x_{umax}) \qquad \dots \quad \dots \quad (2)$$

$$\frac{M_{u,lim}}{bd^2} = 0.54\, f_{ck} \left[\frac{x_{umax}}{d}\right] . \left[\frac{z_{max}}{d}\right] \qquad \dots \quad \dots \quad (3)$$

$$P_{lim} = 43.2 \left[\frac{x_{umax}}{d}\right] . \left[\frac{f_{ck}}{f_y}\right] \qquad \dots \quad \dots \quad (4)$$

For an under reinforced section :

$$x_u = 2.315 . \left[\frac{f_y}{f_{ck}}\right] . \left[\frac{p_t}{100.0}\right] . d \qquad \dots \quad \dots \quad (5)$$

$$\frac{z}{d} = \left[1.0 - 0.416 \frac{x_u}{d}\right] = \left[1.0 - 0.963 \frac{f_y}{f_{ck}} . \frac{P_t}{100.0}\right] \qquad \dots \quad \dots \quad (6)$$

$$\frac{M_p}{bd^2} = 1.25\, f_y . \frac{z}{d} . \frac{P_t}{100.0} \qquad \dots \quad \dots \quad (7)$$

where :

ε_c - Useful limit of strain in concrete.

ε_y - Useful limit of strain in steel.

f_s - Stress in steel.

E - Young's Modulus of steel.

γ_m - Partial safety factor for material strength.

x_u - Depth of neutral axis.

x_{umax} - Limiting value of x_u.

d - Effective depth of section.

d^1 - Cover to compression steel.

f_y - Characteristic strength of reinforcement.

f_{sc} - Stress in compresion steell.

f_{ck} - Characteristic strength of concrete.

b - Width of the compression face.

$M_{u,lim}$ - Limiting moment of resistance of a section without compression steel.

z - Lever arm.

z_{lim} - Limiting value of lever arm, z.

p_c - Percentage area of compression reinforcement.

p_t - Percentage area of tension reinforcement.

P_{lim} - Limiting value of tension reinforcement in singly reinforced sections.

ε_{sc} - Strain at the level of compression steel.

M_p - Plastic moment capacity of the section.

Using the above equations and the steps given in Fig. (A1) design tables and charts can be prepared as in SP:16 and included in SP:22. A brief parametric study, not described here, has revealed that the plastic moment capacity of an under reinforced section is about 44% to 50% higher than its design moment capacity. Corresponding increase in capacity as per ACI is about 40%. In addition, ACI also reduces the allowable shear stresses by about 40% by specifying strength reduction factor of 0.60 for ductile frames as against 0.85 for ordinary concrete frame members.

APPENDIX B : REFERENCES

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APPENDIX C : LIST OF NOTATIONS

Symbols have been defined where they first appear and they are summarised here. Symbols of Appendix:A are given in the text itself.

A	- Gross concrete area of the column section
A_k	- Area of Concrete core = $\frac{\pi D_k^2}{4}$
A_{sc}	- Area of compression steel
A_{sh}	- Area of hoop reinforcement
A_{st}	- Area of tension steel
d	- Effective depth of the section
D_k	- Diameter of core
F_c, f_{ck}	- Characteristic strength of concrete (IS code)
f_c	- Specified cylinder strength of concrete (ACI)
f_y, F_y	- Specified yield strength of steel
h	- Dimension of the stirrups
I	- Importance Factor as per IS:1893
K	- Performance factor as per IS:1893
M_{pa}	- Hogging moment capacity at A
M'_{pa}	- Sagging moment capacity at A
M_{pb}	- Hogging moment capacity at B
M'_{pb}	- Sagging moment capacity at B
s	- Spacing of transverse steel
S_{max}	- Maximum spacing of transverse steel

α_h - Design seismic coefficient

p - Reinforcement ratio

p_{min} - Minimum reinforcement ratio

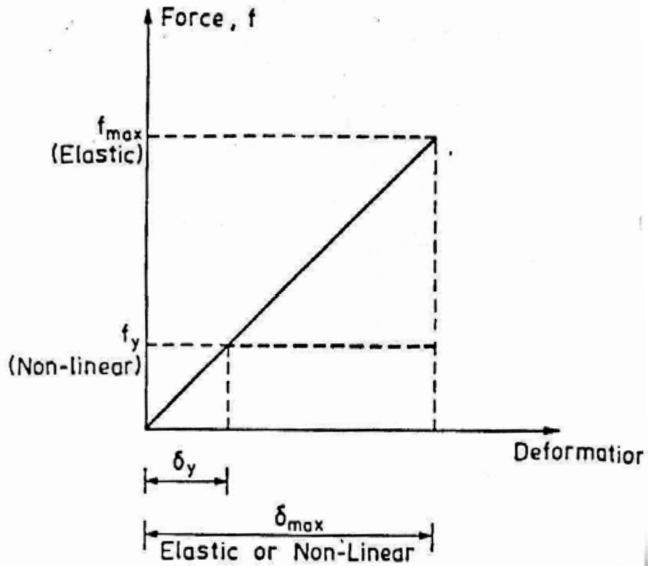
p_{max} - Maximum reinforcement ratio

β - Coefficient depending on soil-foundation system (IS : 1983)

TABLE 1 : VALUES OF PERFORMANCE FACTOR, K
[From Ref. (1)]

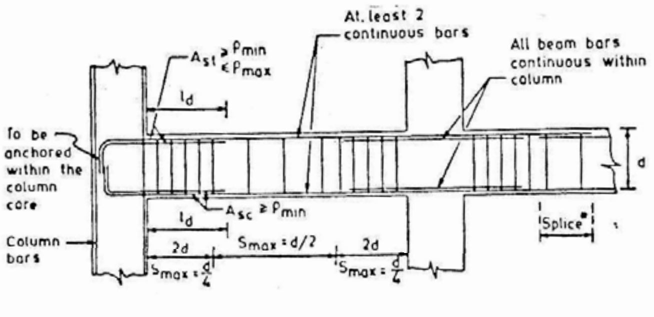
Sl.No.	Structural Framing System	Values of Performance Factor, K	Remarks
1	(2)	(3)	(4)
(i) (a)	Moment resistant frame with appro priate ductility details as given in IS:4326-1976* in reinforced concrete or steel.	1.0	
(b)	Frame as above with R.C. shear walls or steel bracing membersdesigned for ductility.	1.0	) These factors will) apply only if the steel) bracing members) and the infill panels
(ii) (a)	Frame as in (i) (a) with either steel bracing members or plain or nominally reinforced concrete infill panels.	1.3	) are taken into consi-) deration in stiffness) as well lateral stren-) gth calculations pro-) vided that the frame
(b)	Frame as in (i) (a) in combination with masonry infills.	1.6	) acting alone will be) able to resist atleast) 25 percent of the) design seismic) forces.
iii)	Reinforced concrete framed buildings [Not covered by (i) or (ii) above]	1.6	

* Code of practice for earthquake resistant design and construction of buildings (first revision).



$$\text{Ductility Factor} : \mu = \frac{\delta_{max}}{\delta_y} = \frac{f_{max}}{f_y}$$

Fig. 1 : Definition of Ductility Factor (From Ref. - 34)



Minimum Reinforcement

M15 Concrete and Mild Steel Bars : $P_{min} = 0.0035$

Other Concrete and Steel Reinforcement : $P_{min} = 0.06\, F_c / F_y$

Maximum Reinforcement

M15 Concrete and Mild Steel Bars : $P_{max} = P_c + 0.011$

Other Concrete and Mild Steel Bars : $P_{max} = P_c + 0.19\, F_c / F_y$

For Concrete Reinforced with Other Bars .

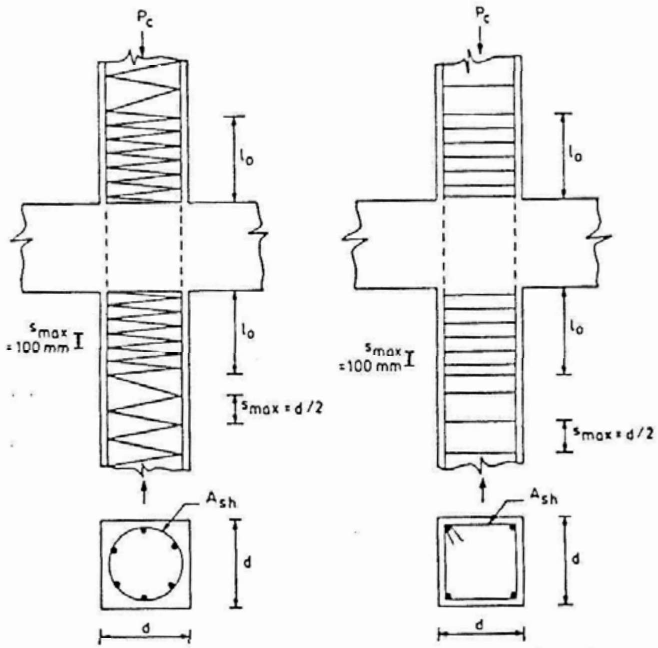
$$P_{max} = P_c + 0.15\, F_c / F_y$$

Web Reinforcement

Max. Spacing of Stirrups d/4 in a Length of 2d Near Each End of the Beam and d/2 in the Remaining Length.

*Splice to Be Contained Within At Least 2 Closed Stirrups, and Not to Be Provided at Sections of Max. Tension.

Fig. 2 : Special Ductility Details for Flexural Members.



$$A_{sh} \geq 0.08\, s\, D_k \cdot \frac{F_c}{F_y} \cdot \left(\frac{A}{A_k} - 1\right) \qquad A_{sh} \geq 0.16\, sh \cdot \frac{F_c}{F_y} \cdot \left(\frac{A}{A_k} - 1\right)$$

(a) Spiral Confinement Reinforcement.

(b) Rectangular Hoop Confinement Reinforcement.

$$l_o \geq \begin{cases} \text{1/6 Height of Column} \\ \text{Larger Lateral Dimension} \\ \text{450 mm} \end{cases}$$

Fig. 3 : Special Ductility Details for Columns.

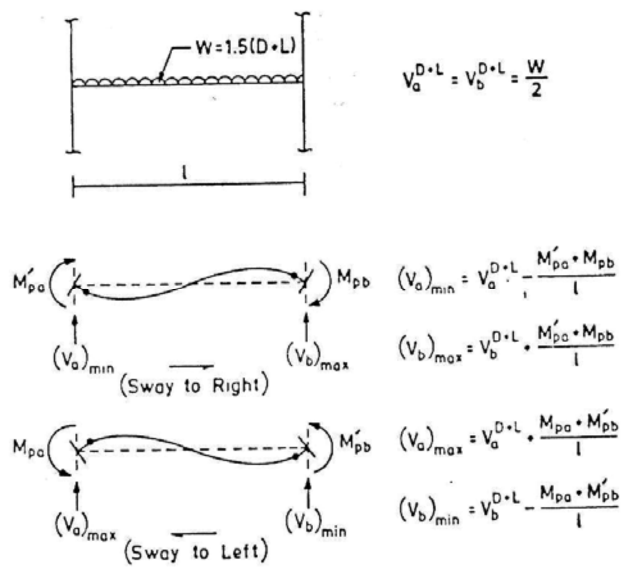


Fig. 4 : Loading Condition for Design of Shear Reinforcement in Flexural Members (Uniformly Distributed Loading).

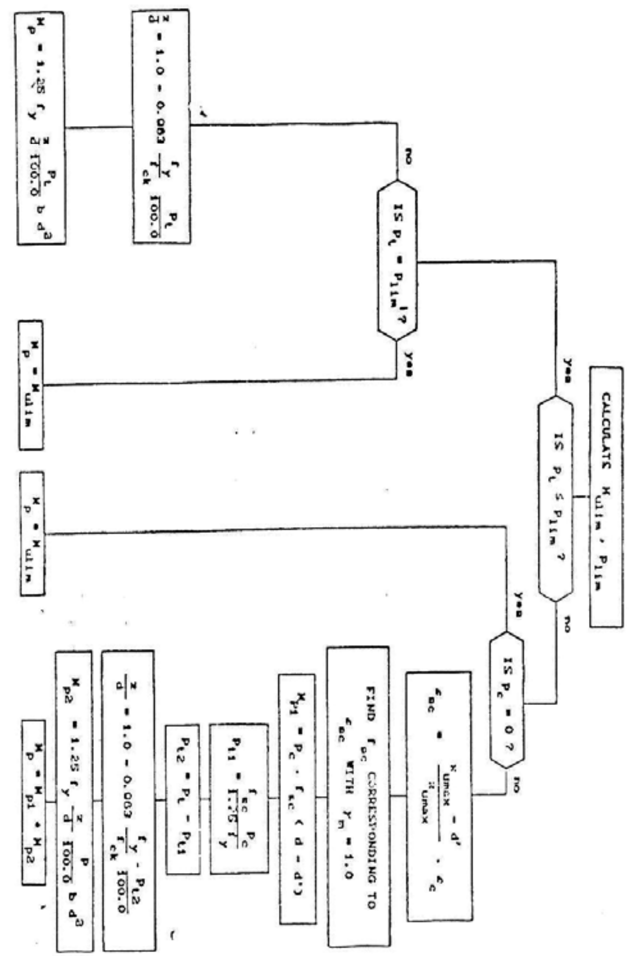
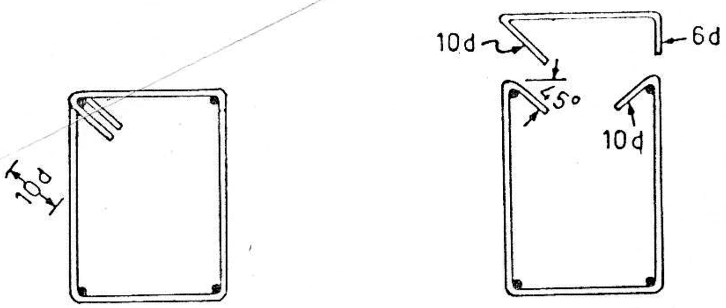
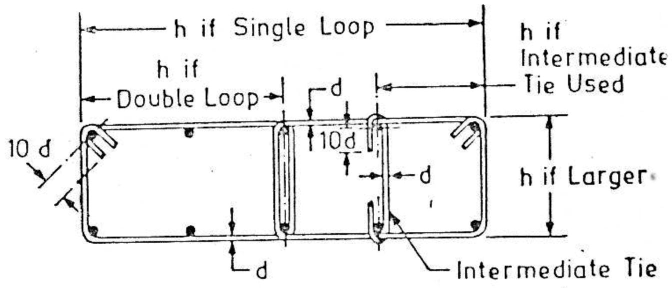


Fig. A1. : Steps for Calculation of Plastic Moment Capacity of R.C. Bear



(a) Details of Hoops (From Ref.- 9)



(b) Details of Hoops for Columns and Dimension h in Rectangular Hoop (Modified from Ref.- 2)

Fig. 5 : Details for Transverse Reinforcement.