Bull. Ind. Soc. Earth. Tech., Paper No. 322, Vol. 29, No. 3, Sept. 1992, pp. 15-35

PROPOSED DRAFT FOR IS: 4326 ON DUCTILE DETAILING OF REINFORCED CONCRETE STRUCTURES

M. S. Medhekar (1), Sudhir K. Jain (2), Anand S. Arya (3)

ABSTRACT

A draft on provisions for ductile detailing of monolithic reinforced concrete structures is proposed for the forthcoming revision of IS: 4326. These provisions endeavour to provide adequate ductility and toughness to the structure so as to improve its seismic performance. Some existing provisions of IS: 4326-1976 have been modified while new ones have been introduced. A detailed commentary giving the rationale of these provisions is also included.

INTRODUCTION

During a severe earthquake, a structure may be subjected to cyclic inelastic deformations. Under these conditions, a properly detailed reinforced concrete structure must have adequate ductility and toughness to sustain these deformations. Reduced stiffness and high damping at large inelastic deformations cause the structure to be subjected to less lateral inertia force as compared to that calculated assuming the structure as linearly elastic. This fact is made use of by seismic design codes, including IS: 1893-1984, when they specify less lateral inertia force for structures which are ductile. Hence, the aim of section 7 of IS: 4326 is to give specifications for design and detailing of monolithic reinforced concrete structures so as to give them adequate toughness and ductility. The specifications intend to prevent the occurrence of brittle failures (i.e., shear failure, compression failure or bond failure) that may cause partial or total collapse of the structure.

The design and construction of reinforced concrete structures is governed by the provisions of IS: 456-1978, except as modified by the provisions of section 7 of IS: 4326. Further, these provisions apply specifically to monolithic reinforced concrete structures. Precast and / or prestressed concrete members may be used only if they can provide the same levels of safety and serviceability as that of a monolithic reinforced concrete structure during and after an earthquake.

IS: 4326 was first published in 1967 and revised subsequently in 1976. This code is currently under revision to incorporate the latest advances in earthquake engineering and the experiences gained from application of the existing provisions. On request of the concerned committee of the Bureau of Indian Standards, a revised draft on ductile

---

(1) Senior Project Associate, Dept. of Civil Engg., Indian Institute of Technology, Kanpur - 208 016.

(2) Assistant Professor, Dept. of Civil Engg., Indian Institute of Technology, Kanpur - 208 016.

(3) Professor Emeritus and Emeritus Scientist, Dept. of Earthquake Engg., University of Roorkee, Roorkee - 247 667.

detailing of reinforced concrete structures has been prepared after an extensive study of seismic design codes elsewhere and the latest research results. It is presumed that design engineers are well versed with IS:456-1978. Hence, the endeavour has been to remain within the framework of IS:456-1978. This paper gives the proposed provisions as well as a detailed commentary on the basis for these provisions. As these provisions may form chapter 7 of the code, the numbering adopted for the provisions is as it is expected in the code.

CODE AND COMMENTARY

7. REINFORCED CONCRETE CONSTRUCTION

7.1 GENERAL REQUIREMENTS

7.1.1 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 section.

7.1.2 In seismic zones IV and V, the minimum grade of concrete shall be M20 (f_{ck} = 20 MPa) for members which are part of the lateral force resisting system of the structure.

Comment: Grade of concrete M15 gives poor ductility to the concrete section. Hence, the minimum is specified as M20 for use in beams and columns of frames, shear walls and trusses that are designed to resist earthquake induced forces on structures in seismic zones IV and V.

7.1.3 Steel reinforcement of grade Fe 500 shall not be used in seismic zones IV and V.

Comment: An increase in the grade of steel reduces ductility of the concrete section. Steel of grade Fe 500 gives poor ductility. Hence, its use is not recommended in seismic zones IV and V.

7.2 FLEXURAL MEMBERS

7.2.1 SCOPE

These requirements apply to frame members resisting earthquake induced forces and designed to resist flexure. These members shall satisfy the following requirements.

7.2.1.1 The factored axial stress on the member under earthquake loading shall not exceed 0.1 f_{ck}.

Comment: If the factored axial stress under earthquake loading exceeds 0.1 f_{ck}, the member shall be designed and detailed as per 7.3.

7.2.1.2 The member shall not have a width-to-depth ratio of less than 0.3.

Comment: There is a lack of experimental data on the behaviour of flexural members having width-to-depth ratio less than 0.3, when subjected to cyclic inelastic deformations. Hence, the minimum width-to-depth ratio is specified as 0.3.

7.2.1.3 The width of the member shall not be less than 200 mm.

7.2.1.4 Clear span for the member shall not be less than four times its effective depth.

Comment: The behaviour of continuous members with span-to-depth ratio less than four is significantly different from that of relatively slender members, when subjected to cyclic inelastic deformations. The ductile detailing provisions of section 7.2 apply to slender members with span-to-depth ratio exceeding four.

7.2.2 LONGITUDINAL REINFORCEMENT

7.2.2.1 (a) The top as well as bottom reinforcement shall consist of at least two bars throughout the member length.

Comment: Both top and bottom bars are required throughout the member length as the zone of moment reversal under the effect of earthquake forces may extend for a considerable distance towards midspan.

(b) The tension steel ratio on any face, at any section, shall not be less than $\rho_{min} = 0.24 \sqrt{f_{ck}}/f_y$; where f_{ck} and f_y are in MPa. This requirement may be relaxed where the area of reinforcement provided is at least one half greater than that required to resist the design moment.

Comment: This provision governs for those members which have a large cross section due to architectural requirements. It prevents the possibility of a sudden failure by ensuring that the moment of resistance of the section with minimum reinforcement is greater than the cracking moment of the section. However, for cantilever T-beams with flange in tension, this minimum reinforcement requirement can be significantly higher (Ref. 1).

7.2.? ~ The maximum tension steel ratio on any face, at any section, shall not exceed $\rho_{max} = 0.025$.

Comment: The maximum tension steel ratio is specified as 0.025 to prevent congestion of beam reinforcement near the joint and to limit shear stresses in beams of usual proportions.

7.2.2.3 The positive moment capacity at a joint face must be at least equal to half the negative moment capacity at that face.

Comment: This clause intends to ensure strength and ductility of the beam under extreme seismic conditions. There is a possibility of positive moment at one end of a beam, due to earthquake induced lateral displacements, exceeding the negative moment due to gravity loads on the span (Fig. C1).

Further, the ductility of a reinforced concrete section increases with an increase in compression reinforcement and a reduction in tension reinforcement. IS: 4326-1976 had provisions for maximum reinforcement ratio that intended to provide curvature ductility of 5 at the ultimate strength of the member, considering the concrete as unconfined. However, there was a discrepancy in the calculation of curvature ductility. The revised maximum reinforcement ratio as per this basis works out to be (Ref. 2)

$$\rho_{max} = \rho_c + 0.180\, f_{ck} / f_y \qquad \text{for } f_y = 250 \text{ MPa}$$

$$\rho_{max} = 0.75\, \rho_c + 0.140\, f_{ck} / f_y \qquad \text{for } f_y = 415 \text{ MPa}$$

$$\rho_{max} = 0.55\, \rho_c + 0.125\, f_{ck} / f_y \qquad \text{for } f_y = 500 \text{ MPa}$$

where ρ_c is the compression reinforcement ratio. These requirements are more stringent than the provision in IS: 4326-1976. However, this method of arriving at the maximum reinforcement ratio by relating it to a specified level of curvature ductility is not being adopted herein. This is because there is no consensus as yet on how much curvature ductility to provide. It may be noted that the ACI (Ref. 3), UBC (Ref. 4) and SEAOC codes (Ref. 5) specify the maximum tension reinforcement ratio as 0.025 in order to avoid congestion of reinforcement in the beam. Moreover, these codes stipulate that the positive moment capacity at joint face should be at least one half of the negative moment capacity at that face. Hence, in these codes reliance is placed on the fact that this positive moment steel acts as compression reinforcement for the usual condition of negative moments at joint face and thus ensures adequate curvature ductility.

7.2.2.4 The positive and negative moment capacity at any section along the member length must be at least equal to one fourth of the maximum moment capacity provided at the face of either joint.

Comment: The positive or negative moment capacity may not necessarily be reached at the joint face; it may be reached within the member. Thus, at any section, the positive and negative moment capacity must be at least one fourth of the maximum moment capacity provided at the face of either joint.

7.2.2.5 In an external joint, both the top and the bottom bars of the beam shall be provided with anchorage length, beyond the inner face of the column, equal to the development length in tension plus 10 times the bar diameter (Fig. 1). In an internal joint, both face bars of the beam shall be taken continuously through the column.

Comment: Proper anchorage of beam reinforcement is essential to prevent brittle failure. During a severe earthquake, the zone of inelastic deformation that exists at the ends of a beam, may extend for some distance within the column. This makes the bond between concrete and steel ineffective in this region. Hence, development length of the bar in tension is provided beyond a section which is at a distance of 10 times the bar diameter from the inner face of the column. Beam stub may be used at external joints for easier detailing and to ensure good seismic performance of the joint (Fig. C2). At an internal joint, the top and bottom bars of the beam should be taken continuously through the joint. This is because the top as well as bottom reinforcement will need to function as tension reinforcement due to the possibility of stress reversal.

7.2.2.6 The longitudinal bars shall be spliced only if hoops are provided over the entire splice length at a spacing not exceeding 150 mm (Fig. 2). The lap length shall be the greater of (a) bar development length in tension, and (b) 30 times the bar diameter. Lap splices shall not be provided (a) within a joint, (b) within a distance of 2d from joint face, and (c) where analysis indicates that flexural yielding may occur under the effect of earthquake forces.

Comment: Lap splices are not permitted in regions where flexural yielding may occur as they are not reliable when subjected to cyclic inelastic deformations. Hoops are to be provided as they ensure good performance of the splice even when the concrete cover has spalled off.

7.2.3 **WEB REINFORCEMENT**

7.2.3.1 Web reinforcement shall consist of vertical hoops. A vertical hoop is a closed stirrup having a 135° hook with a 10 diameter extension (but not < 75 mm) at each end that is embedded in the confined core (Fig. 3a). It may also be made up of 2 pieces of reinforcement; a U-stirrup with a 135° hook and a 10 diameter extension (but not < 75 mm) at each end, embedded in the confined core and a crosstie (Fig. 3b). A crosstie is a bar having a 135° hook with a 10 diameter extension (but not < 75 mm) at each end. The hooks shall engage peripheral longitudinal bars.

7.2.3.2 The minimum diameter of the bar forming a hoop shall be 6 mm. However, in beams with clear span exceeding 5 m, the minimum bar diameter shall be 8 mm.

7.2.3.3 The shear force to be resisted by the vertical hoops shall be the maximum of (a) calculated factored shear force as per analysis, and (b) shear force due to formation of plastic hinges at both ends of the beam plus the factored gravity load on the span. The latter is given by (Fig. 4)

(i) for sway to right

$$(V_a)_{min} = V_a^{D+L} - 1.5\left[\frac{M_{u,lim}^{As} + M_{u,lim}^{Bh}}{L_{AB}}\right]$$

$$(V_b)_{max} = V_b^{D+L} + 1.5\left[\frac{M_{u,lim}^{As} + M_{u,lim}^{Bh}}{L_{AB}}\right]$$

(ii) for sway to left

$$(V_a)_{max} = V_a^{D+L} + 1.5\left[\frac{M_{u,lim}^{Ah} + M_{u,lim}^{Bs}}{L_{AB}}\right]$$

$$(V_b)_{min} = V_b^{D+L} - 1.5\left[\frac{M_{u,lim}^{Ah} + M_{u,lim}^{Bs}}{L_{AB}}\right]$$

where, $M_{u,lim}^{As}$, $M_{u,lim}^{Ah}$ and $M_{u,lim}^{Bs}$, $M_{u,lim}^{Bh}$ are the sagging and hogging moments of resistance of the beam section at ends A and B, respectively. These are to be calculated as per IS:456-1978. L_{AB} is clear span of beam. V_a^{D+L} and V_b^{D+L} are the shears at ends A and B, respectively, due to vertical loads with a factor of safety of 1.2 on loads. The design shear at end A shall be the absolute maximum of $(V_a)_{min}$ and $(V_a)_{max}$. Similarly, the design shear at end B shall be the absolute maximum of $(V_b)_{min}$ and $(V_b)_{max}$.

Comment: During a severe earthquake, beam web reinforcement must be adequate to prevent the occurrence of a brittle shear failure. Thus, beams must be designed to resist the maximum shear that is likely to occur when plastic hinges form at either ends of the beam due to earthquake induced loads. This will prevent a brittle shear failure. The moment capacity of plastic hinges should be calculated taking into account all possible factors that may cause an increase in strength, namely, steel strength being higher than the specified yield strength, effect of strain hardening of the steel at high deformations, and concrete strength being higher than specified. This will give a more realistic maximum shear force. Thus, the plastic moment capacity may be computed assuming the partial safety factors for material strength of concrete and steel as 1.3 and 1.0, respectively, and steel yield stress as 1.25 f_y (Ref. 6). It is seen that using these factors the plastic moment capacity is approximately 1.5 times the moment capacity obtained as per IS:456-1978. Hence, to simplify the calculation procedure, the moment of resistance at the ends of the beam may be calculated as per IS:456-1978 and increased by a factor of 1.5 to obtain the plastic moment capacity. The partial safety factor of 1.2 on vertical loads is intended to include the possible effects of vertical acceleration on gravity loads.

7.2.3.4 In seismic zones IV and V, the contribution of bent up bars and inclined hoops to shear resistance of the section shall not be considered.

Comment: During a severe earthquake, the direction of shear force may reverse. Bent up bars and inclined hoops are effective for resisting shear in one direction only. Hence, their contribution to shear resistance of the section shall not be considered in seismic zones IV and V.

7.2.3.5 The spacing of hoops over a length of 2d at either end of a beam shall not exceed (a) d/4, (b) 8 times the diameter of the smallest longitudinal bar, and (c) 24 times the diameter of the hoop bar (Fig. 5). The first hoop shall be at a distance not exceeding 50 mm from the joint face. Vertical hoops at the same spacing as above, shall also be provided over a length equal to 2d on either side of a section where flexural yielding may occur under the effect of earthquake forces. Elsewhere, the beam shall have vertical hoops at a spacing not exceeding d/2.

Comment: Under the effect of earthquake forces, flexural yielding is likely to occur at the ends of beams. Hence, hoops have to be provided in these regions to confine the concrete and to provide lateral restraint to the compression reinforcement. Sometimes, a column is supported by a beam or a heavy point load acts at the beam midspan. In such cases, flexural yielding may occur near the midspan under the effect of earthquake forces. This reinforcement must also be provided at such sections. The hoop spacing is specified as d/2 over the remaining length of the beam to prevent the occurrence of an unexpected shear failure in this region.

7.3 COLUMNS AND FRAME MEMBERS SUBJECTED TO BENDING AND AXIAL LOAD

7.3.1 SCOPE

These requirements apply to frame members which have a factored axial stress in excess of 0.1 f_{ck} under the effect of earthquake forces.

7.3.1.1 The minimum dimension of the member shall not be less than 200 mm. However, in frames which have beams with clear span exceeding 5 m or columns of unsupported length exceeding 4 m, the shortest dimension of the column shall not be less than 300 mm.

7.3.1.2 The ratio of the shortest cross sectional dimension to the perpendicular dimension shall not be less than 0.4.

Comment: There is not much experimental data on the behaviour of columns with proportions other than specified above, when subjected to cyclic inelastic deformations.

7.3.2 LONGITUDINAL REINFORCEMENT

7.3.2.1 At a joint in a frame resisting earthquake induced forces, the sum of the moment of resistance of the columns shall be at least 1.2 times the sum of the moment of resistance of the beams along each principal plane of the joint (Fig. 6). The moment of resistance of the column shall be calculated considering the factored axial force on the column. The moment of resistance shall be summed such that the column moments oppose the beam moments. This requirement shall be satisfied for beam moments acting in both directions in the principal plane of the joint considered.

Comment: This is the weak beam-strong column concept. It intends to force the formation of plastic hinges in beams rather than in columns. Hinging in columns must be avoided as it causes greater damage to the structure and may also lead to the formation of a panel collapse mechanism.

7.3.2.2 Columns not satisfying 7.3.2.1 shall have special confining reinforcement over their full height instead of the critical end regions only.

7.3.2.3 Lap splices shall be provided only in the central half of the member length. It should be proportioned as a tension splice. Hoops shall be provided over the entire splice length at spacing not exceeding 150 mm center to center.

Comment: During an earthquake, spalling of concrete shell is likely to occur at the ends of the column where large rotations take place. Hence, lap splices should be provided only in the central half of the member where rotations are small and stress reversals are of lower magnitude than at locations near the joints. A lap must be proportioned as a tension splice as there is a possibility of stress reversal occurring under the effect of earthquake forces. The presence of confining reinforcement over the full length of lap splice improves its seismic performance significantly.

7.3.2.4 Any area of a column that extends more than 100 mm beyond the confined core due to architectural requirements, shall have minimum longitudinal and transverse reinforcement as per IS:456-1978 (Fig. 7).

Comment: This provision is for the non-structural portion of a column that extends more than 100 mm beyond the confined core. It aims at preventing a sudden loss in column stiffness due to spalling of this portion.

7.3.3 TRANSVERSE REINFORCEMENT

7.3.3.1 Transverse reinforcement for circular columns shall consist of spiral or circular hoops. In rectangular columns, rectangular hoops may be used. A rectangular hoop is a closed stirrup having a 135° hook with a 10 diameter extension (but not < 75 mm) at each end that is embedded in the confined core (Fig. 8a).

7.3.3.2 The parallel legs of rectangular hoops shall be spaced not more than 350 mm center to center. If the length of any side of the hoop exceeds 350 mm, a crosstie shall be provided (Fig. 8b). Alternatively, a pair of overlapping hoops may be provided within the column (Fig. 8c). The hooks shall engage peripheral longitudinal bars.

7.3.3.3 The spacing of hoops shall not exceed half the least lateral dimension of the column, except where special confining reinforcement is provided as per 7.3.4.

7.3.3.4 The column shall be designed for the maximum of (a) calculated factored shear force as per analysis, and (b) a factored shear force given by

$$V_u = 1.5 \left\{ \frac{M^{bL}_{u,lim} + M^{bR}_{u,lim}}{h_{st}} \right\}$$

where, $M^{bL}_{u,lim}$ and $M^{bR}_{u,lim}$ are moment of resistance, of opposite sign, of beams framing into the column from opposite faces (Fig. 9); and h_{st} is the story height. The beam moment capacity is to be calculated as per IS:456-1978.

Comment: This clause prevents the occurrence of a brittle shear failure in the column before a ductile flexural failure in the beams framing into the column. The maximum shear that can be developed in a column is the sum of maximum moments that can be resisted by the beams framing into opposite faces of the column, divided by the story height. The maximum beam moments are computed assuming the partial safety factors for material strength of concrete and steel as 1.3 and 1.0, respectively, and steel yield stress as 1.25 f_y, for the reasons mentioned in 7.2.3.3 (comment).

However, for convenience in design, the moment of resistance of beams is to be calculated as per IS:456-1978. This value is to be increased by the factor of 1.5 to approximate the maximum moments that can possibly develop in the beams.

7.3.4 SPECIAL CONFINING REINFORCEMENT

This requirement shall be met with unless a larger amount of transverse reinforcement is required from shear strength considerations.

Comment: Special confining reinforcement consists of closely spaced hoops or spiral. It serves three purposes. It provides shear resistance to the member. It confines the concrete core and thereby increases the ultimate strain of concrete which gives greater ductility to the concrete cross section and enables it to undergo large deformations. It also provides lateral restraint against buckling to the compression reinforcement.

7.3.4.1 Special confining reinforcement shall be provided over a length 'l_o' from each joint face, towards midspan, and on either side of any section where flexural yielding may occur under the effect of earthquake forces (Fig. 10). The length 'l_o' shall not be less than (a) depth of member at section where yielding occurs, and (b) 1/6 of clear span of the member

Comment: These regions are subjected to large inelastic deformations. Hence, special confining reinforcement has to be provided to ensure adequate ductility and to provide restraint against buckling to the compression reinforcement.

7.3.4.2 When a column terminates into a footing or mat, special confining reinforcement shall extend at least 300 mm into the footing or mat (Fig. 11).

Comment: During a severe earthquake, a plastic hinge may form at the bottom of a column that terminates into a footing or mat. This region is subjected to large cyclic inelastic deformations. Hence, special confining reinforcement has to be provided in this region.

7.3.4.3 When the calculated point of contraflexure, under the effect of gravity and earthquake loads, is not within the middle half of the member clear height, special confining reinforcement shall be provided over the full height of the column.

Comment: The point of contraflexure is usually in the middle half of the column, except for columns in the top and bottom storeys of a multistorey frame. When the point of contraflexure is not within the middle half of the column, the zone of inelastic deformation may extend beyond the region that is provided with closely spaced hoop reinforcement. This clause requires the provision of special confining reinforcement over the full height of the column so as to give it adequate rotation capacity.

7.3.4.4 Columns supporting reactions from discontinued stiff members, such as walls, shall be provided with special confining reinforcement over their full height (Fig. 12). This reinforcement shall also be placed above the discontinuity for at least the development length of the largest longitudinal bar in the column. Where the column is supported on a wall, this reinforcement shall be provided over the full height of the column; it shall also be provided below the discontinuity for the same development length.

Comment: Observations in past earthquakes indicate that earthquake loads on walls generate significant compressive force on supporting columns. These columns also undergo extensive inelastic deformations. Hence, special confining reinforcement has to be provided over their full height to give them good rotation capacity.

7.3.4.5 Special confining reinforcement shall be provided over the full height of a column which has significant variation in stiffness along its height. This variation in stiffness may result due to the presence of bracing, a mezzanine floor or a R.C.C. wall on either side of the column that extends only over a part of the column height (Fig. 13).

Comment: This situation occurs frequently in building frames, for instance in frames with semi-basements where ventilators are provided between the soffit of beams and the top of the wall. Column stiffness is inversely proportional to the cube of column height. Hence, columns with significantly less height than other columns in the same storey are very stiff and consequently attract much greater seismic shear force. There is a possibility of brittle shear failure occurring in the unsupported zones of such columns. This has been observed in several earthquakes in the past. A mezzanine floor or a loft also results in the stiffening of some of the columns while leaving other columns of the same floor unbraced over their full height. Hence, special confining reinforcement shall be provided over the full height in such columns to give them adequate shear strength.

7.3.4.6 The spacing of hoops used as special confining reinforcement shall not exceed the smaller of (a) 1/4 of minimum member dimension, and (b) 100 mm.

7.3.4.7 The area of cross-section, A_{sh}, of the bar forming circular hoop or spiral to be used as special confining reinforcement shall not be less than

$$A_{sh} = 0.09\ S\ D_k \frac{f_{ck}}{f_y}\left[\frac{A_g}{A_k} - 1.0\right]$$

where

A_{sh} = area of the bar cross section

S = pitch of spiral or spacing of hoops

D_k = diameter of core measured to the outside of the spiral or hoop

f_{ck} = characteristic compressive strength of concrete cube

f_y = yield stress of steel (of circular hoop or spiral)

A_g = gross area of the column cross section

A_k = area of the concrete core = $\frac{\pi}{4} D_k^2$

Example: Consider a column of diameter 300 mm. Let the grade of concrete be M20 and that of steel be Fe 415 for longitudinal and confining reinforcement. The spacing of circular hoops, S, shall not exceed the smaller of (a) 1/4 of minimum member dimension = 1/4 x 300 = 75 mm, and (b) 100 mm. Therefore, S = 75 mm. Assuming 40 mm clear cover to the longitudinal reinforcement and circular hoops of diameter 8 mm, D_k = 300 - 2 x 40 + 2 x 8 = 236 mm. Thus, the area of cross section of the bar forming circular hoop works out to be 47.28 mm^2. This is less than the cross sectional area of 8 mm bar (50.27 mm^2). Thus, circular hoops of diameter 8 mm at a spacing of 75 mm c/c will be adequate.

Comment: This provision intends to provide adequate confining reinforcement to the column. It is obtained by equating the maximum axial load carrying capacity of the column prior to the spalling of its shell, to its axial load carrying capacity at large compressive strains with the spiral reinforcement stressed to its useful limit.

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

$$A_{sh} = 0.18\ S\ h \frac{f_{ck}}{f_y}\left[\frac{A_g}{A_k} - 1.0\right]$$

where

h = longer dimension of the rectangular confining hoop measured

to its outer face. It shall not exceed 350 mm (Fig. 8).
A_k = area of confined concrete core in the rectangular hoop measured to its outside dimensions

Note: The dimension h of the hoop could be reduced by introducing crossties as shown in Fig. 8b. In this case, A_k shall be measured as the overall core area regardless of the hoop arrangement. The hooks of crossties shall engage peripheral longitudinal bars.

Example: Consider a column of 750 mm x 500 mm. Let the grade of concrete be M20 and that of steel be Fe 415 for the longitudinal and confining reinforcement. Assuming clear cover of 40 mm to the longitudinal reinforcement and rectangular hoops of diameter 10 mm, the size of the core is 690 mm x 440 mm. As both these dimensions are greater than 350 mm, either a pair of overlapping hoops or a single hoop with crossties in both directions will have to be provided. Thus, the dimension 'h' will be the larger of (i) 690/2 = 345 mm, and (ii) 440/2 = 220 mm. The spacing of hoops, S, shall not exceed the smaller of (a) 1/4 of minimum member dimension = 1/4 x 500 = 125 mm, and (b) 100 mm. Thus, S = 100 mm. The area of cross section of the bar forming rectangular hoop works out to be 70.38 mm^2. This is less than the area of cross section of 10 mm bar (78.54 mm^2). Thus, 10 mm diameter rectangular hoops at 100 mm c/c will be adequate.

Comment: This provision intends to provide the same confinement to a rectangular core confined by rectangular hoops as would exist in an equivalent spiral column, assuming that rectangular hoops are 50% as efficient as spirals in providing confinement to concrete.

7.4 JOINTS OF FRAMES

7.4.1 The special confining reinforcement as required at the end of column shall be provided through the joint as well, unless the joint is confined as specified by 7.4.2.

7.4.2 A joint, which has beams framing into all vertical faces of it and where each beam width is at least 3/4 of the column width, may be provided with half the special confining reinforcement required at the end of the column. The spacing of hoops shall not exceed 150 mm.

CONCLUSIONS

Specifications for ductile design and detailing of monolithic reinforced concrete structures are proposed for inclusion in the future revision of IS:4326. These provisions endeavour to provide adequate toughness and ductility to the structure so as to improve its seismic response. The main modifications in the existing requirements in IS:4326-1976 may be summarised as follows:

(a) Material Specifications

Material specifications are indicated for lateral force resisting elements of frames in seismic zones IV and V.

(b) Flexural Members

Geometric constraints are imposed on the cross section. Provisions on minimum and maximum reinforcement are revised. The requirements for detailing of longitudinal reinforcement in beams at joint faces, splices and anchorage requirements are made more explicit. Provisions are also included for calculation of design shear force and for detailing of transverse reinforcement in beams.

(c) Members Subjected to Axial Load and Flexure

Dimensional constraints are imposed on the cross section. Provisions are included for detailing of lap splices and for the calculation of design shear force. A comprehensive set of requirements is included on the provision of special confining reinforcement in those regions of a column that are expected to undergo cyclic inelastic deformations during a severe earthquake.

REFERENCES

1. Medhekar, M. S., and Jain, S. K., 1991, Proposed Minimum Tension Reinforcement Requirement for Flexural Members, communicated for publication.
2. Rashad, G. E., Medhekar, M. S., and Jain, S. K., 1990, Suggested Requirements on Maximum Tension Reinforcement for Flexural Members in IS:4326, accepted for publication in **Bulletin of the Indian Society of Earthquake Technology**.
3. ACI 318-89/318R-89, 1990, **Building Code Requirements for Reinforced Concrete and Commentary**, American Concrete Institute, Detroit, USA.
4. **Uniform Building Code**, 1988 edition, International Conference of Building Officials, Whittier, USA.
5. **Recommended Lateral Force Requirements and Commentary**, 1980 edition, Structural Engineers Association of California, San Francisco, USA.
6. Medhekar, M. S., Rashad, G. and Jain, S. K., 1990, Shear Reinforcement for Aseismic Design of Flexural Members, **The Indian Concrete Journal** (in press).
7. IS:456-1978, **Indian Standard Code of Practice for Plain and Reinforced Concrete**, BIS, New Delhi.
8. IS:4326-1976, **Indian Standard Code of Practice for Earthquake Resistant Design and Construction of Buildings**, BIS, New Delhi.

LIST OF SYMBOLS

A_g - gross area of the column cross section

A_k - area of concrete core of column

A_{sh} - area of cross section of bar forming spiral or hoop

D_k - diameter of column core measured to the outside of spiral or hoop

d - effective depth of member

f_{ck} - characteristic compressive strength of concrete cube

f_y - yield stress of steel

h - longer dimension of rectangular confining hoop measured to its outer face

h_{st} - story height

L_{AB} - clear span of beam

l_o - length of member over which special confining reinforcement is to be provided

M_{c1} - moment of resistance of column

M_{c2} - moment of resistance of column

M_{g1} - moment of resistance of beam

M_{g2} - moment of resistance of beam

$M^{Ah}_{u,lim}$ - hogging moment of resistance of beam at end A

$M^{As}_{u,lim}$ - sagging moment of resistance of beam at end A

$M^{Bh}_{u,lim}$ - hogging moment of resistance of beam at end B

$M^{Bs}_{u,lim}$ - sagging moment of resistance of beam at end B

$M^{bL}_{u,lim}$ - moment of resistance of beam framing into column from the left

$M^{bR}_{u,lim}$ - moment of resistance of beam framing into column from the right

N - factored axial force on column

S - pitch of spiral or spacing of hoops

V_a^{D+L} - shear at end A of beam due to dead and live loads with a factor of safety of 1.2 on loads

$(V_a)_{max}$ - maximum shear at end A of beam

$(V_a)_{min}$ - minimum shear at end A of beam

V_b^{D+L} - shear at end B of beam due to dead and live loads with a factor of safety of 1.2 on loads

$(V_b)_{max}$ - maximum shear at end B of beam

$(V_b)_{min}$ - minimum shear at end B of beam

V_u - factored shear force for column

ΣM_c - sum of moment of resistance of columns at a joint

ΣM_g - sum of moment of resistance of beams framing into a joint from opposite faces

ρ_c - compression reinforcement ratio in a beam

ρ_{max} - maximum tension reinforcement ratio for a beam

ρ_{min} - minimum tension reinforcement ratio for a beam

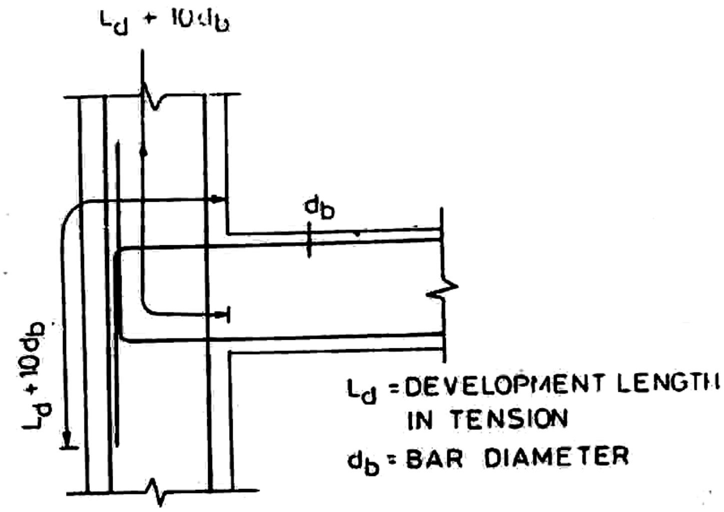


Fig. 1 ANCHORAGE OF BEAM BARS IN AN EXTERNAL JOINT

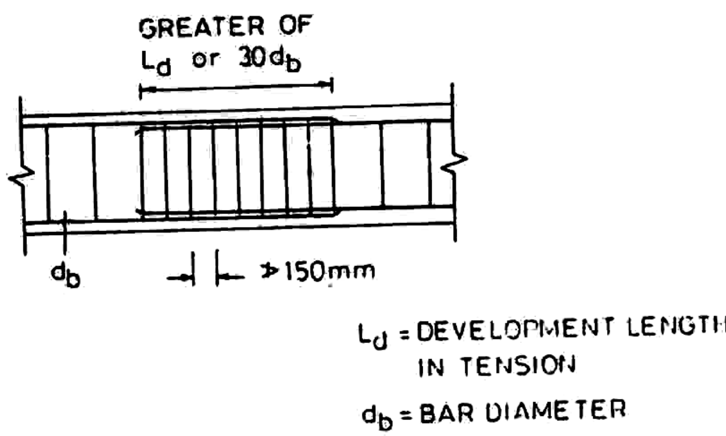


Fig. 2 LAP SPLICE IN BEAM

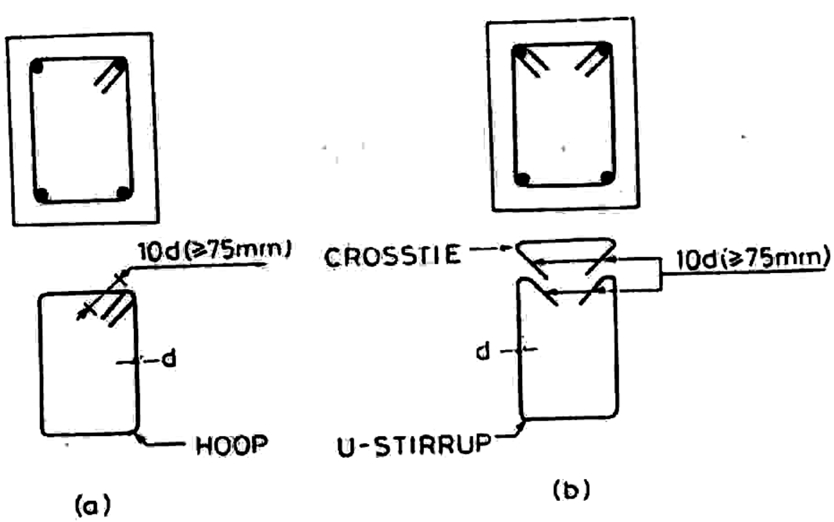


Fig. 3 BEAM WEB REINFORCEMENT

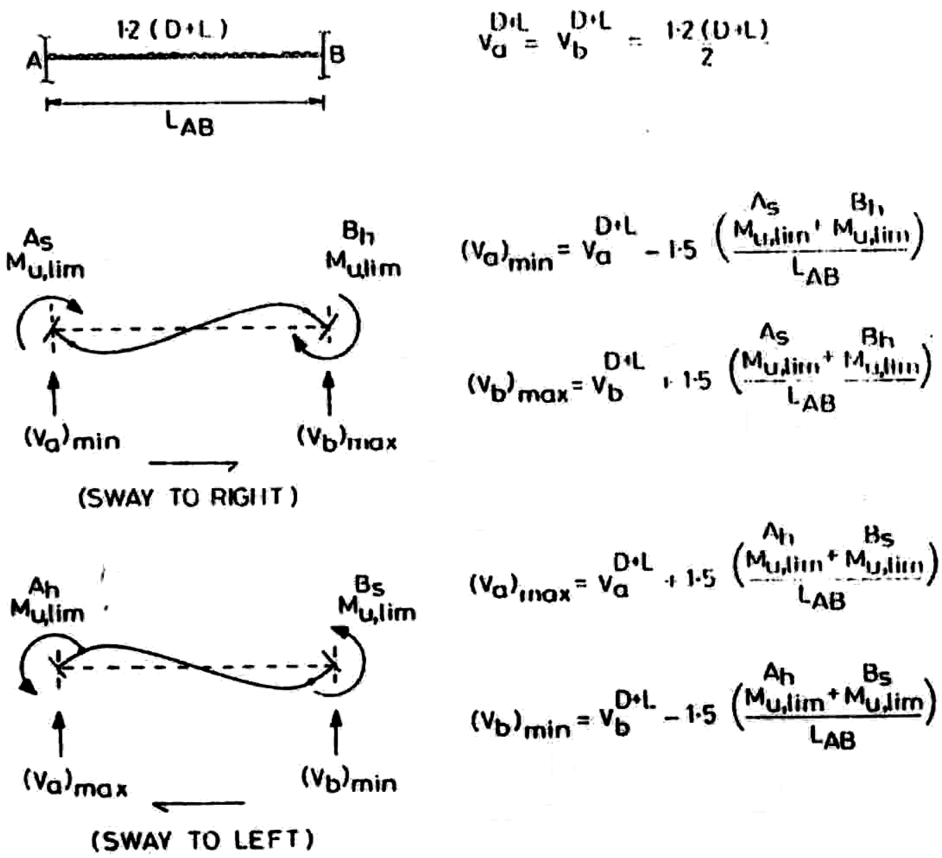


Fig. 4 CALCULATION OF DESIGN SHEAR FORCE FOR BEAM

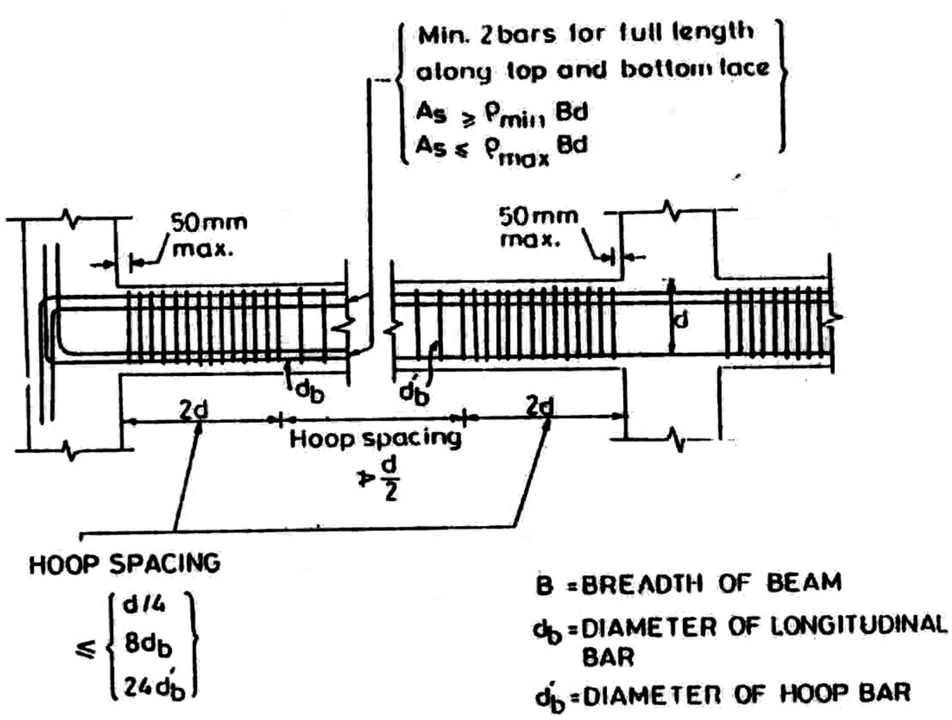
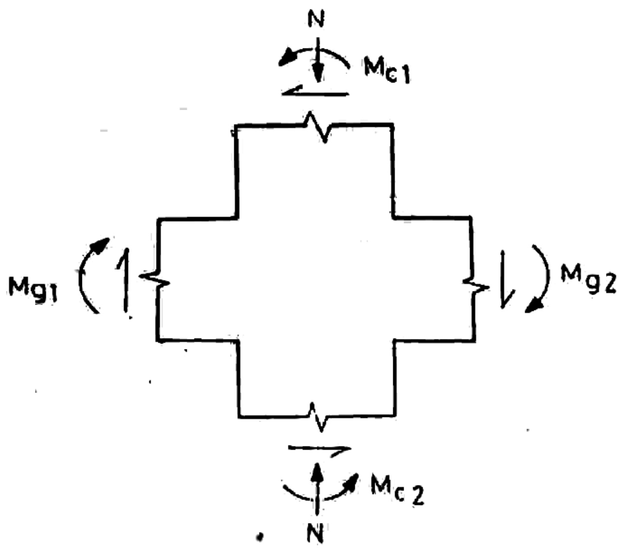


Fig. 5 BEAM REINFORCEMENT



$$\Sigma M_c = M_{c1} + M_{c2}$$

$$\Sigma M_g = M_{g1} + M_{g2}$$

$$\Sigma M_c \geq 1{\cdot}2 \ \Sigma M_g$$

Fig. 6 WEAK BEAM-STRONG COLUMN CONCEPT

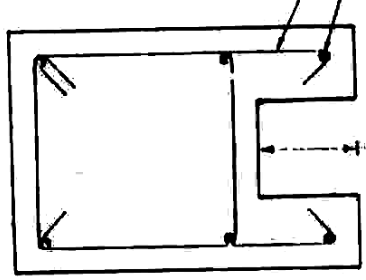


Fig. 7 REINFORCEMENT REQUIREMENT FOR COLUMN WITH MORE THAN 100 mm PROJECTION BEYOND CORE

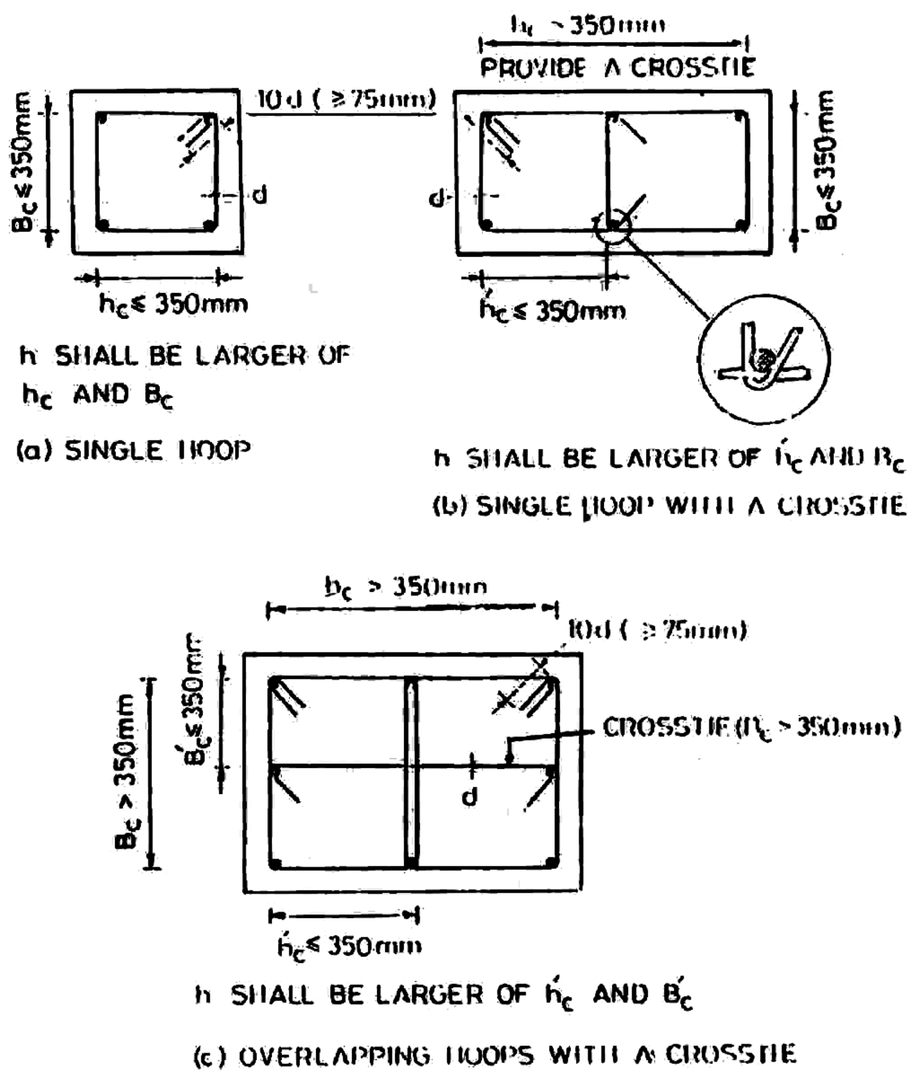
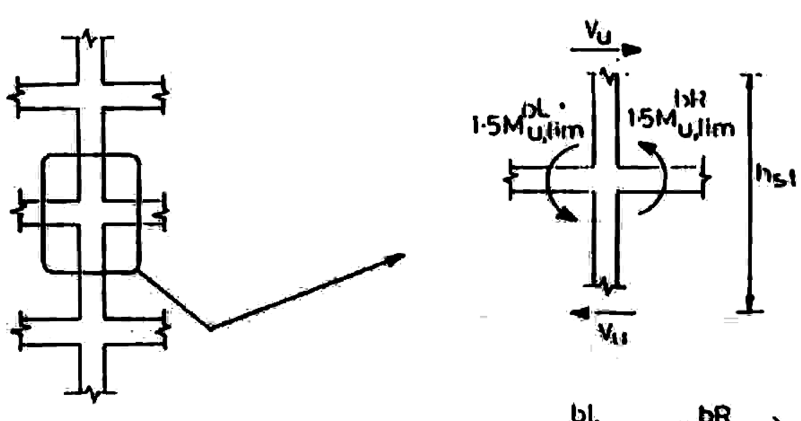


Fig. 8 TRANSVERSE REINFORCEMENT IN COLUMN



$$V_u = 1.5\left(\frac{M^{bL}_{u,lim} + M^{bR}_{u,lim}}{h_{st}}\right)$$

Fig. 9 CALCULATION OF DESIGN SHEAR FORCE FOR COLUMN

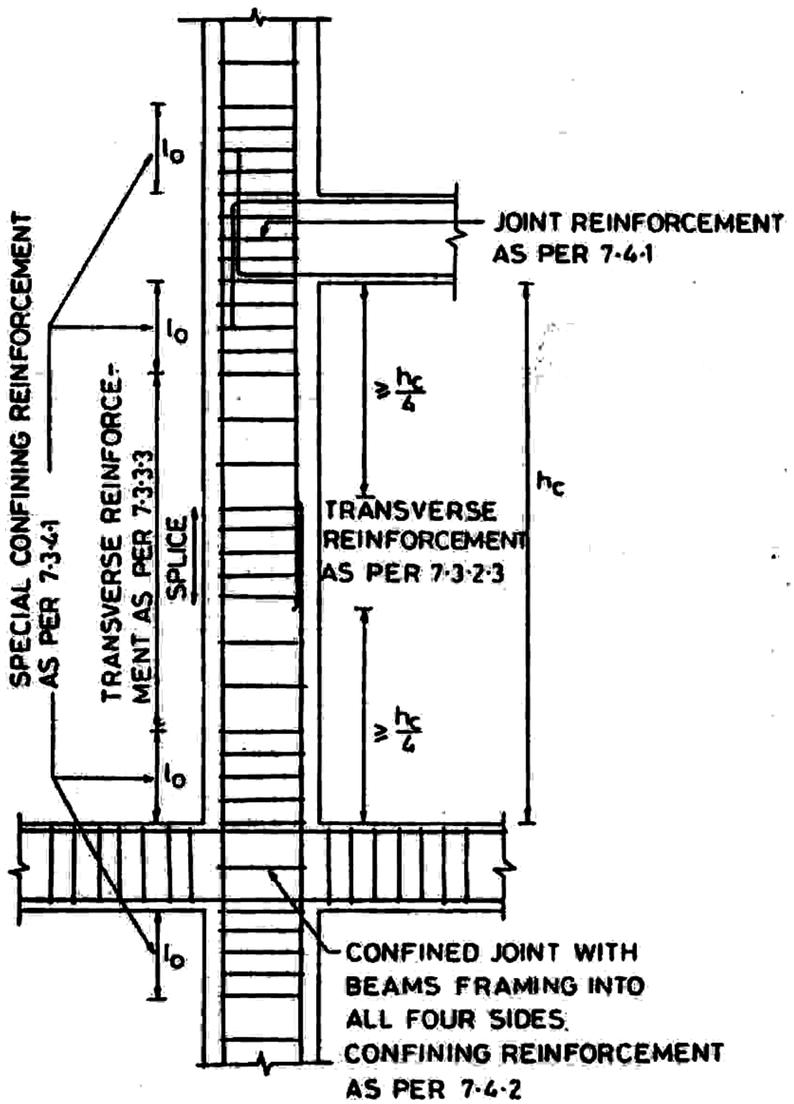


Fig. 10 COLUMN AND JOINT DETAILING

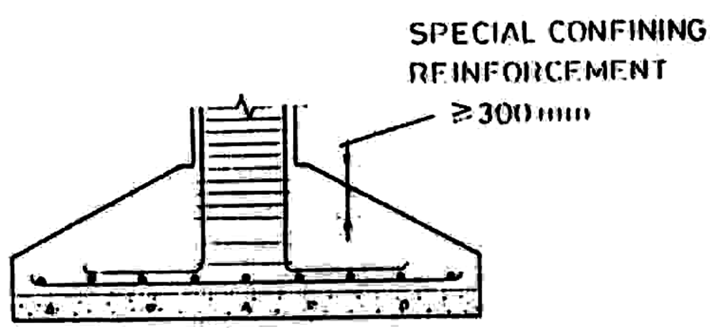


Fig. 11 PROVISION OF SPECIAL CONFINING REINFORCEMENT IN FOOTINGS

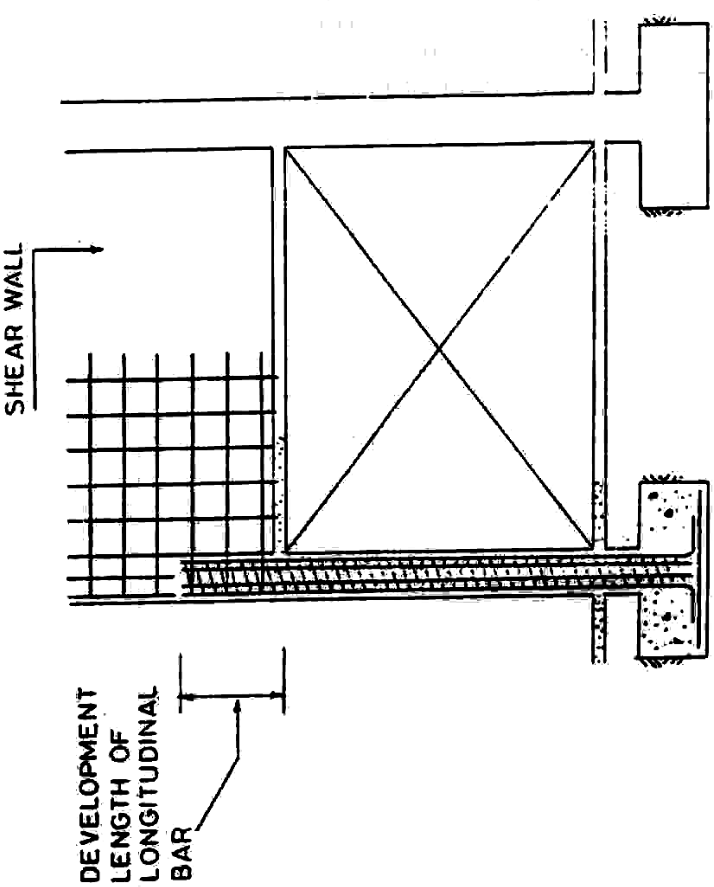
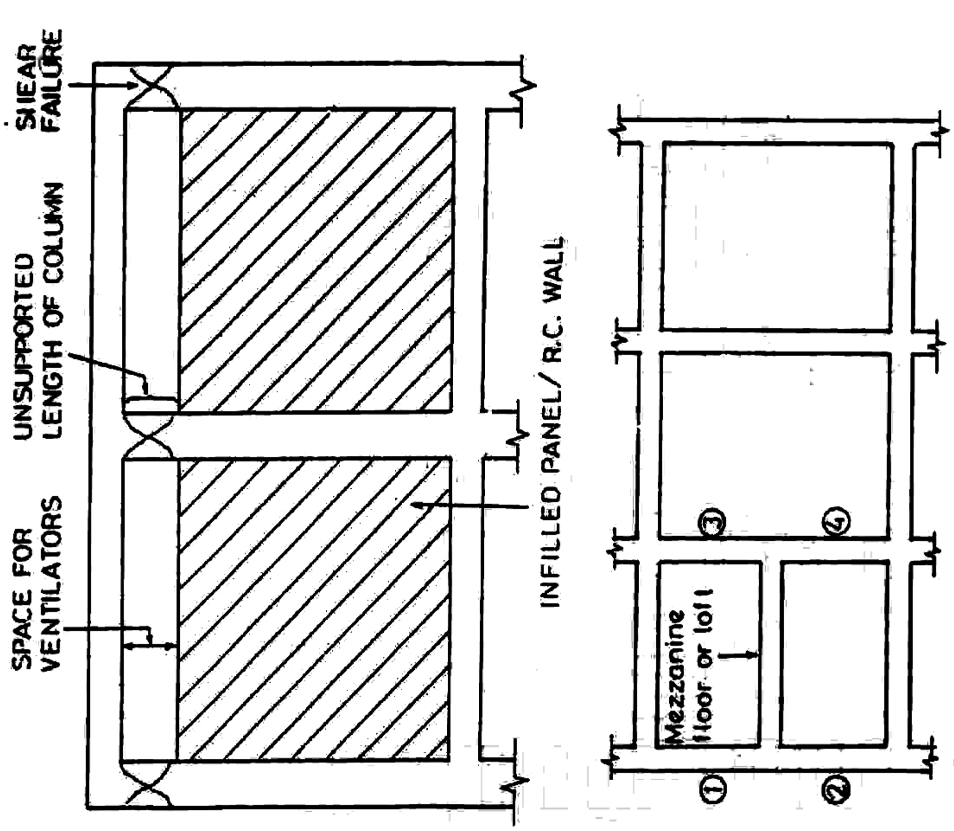


Fig. 12 SPECIAL CONFINING REINFORCEMENT REQUIREMENT FOR COLUMNS UNDER DISCONTINUED WALLS



1, 2, 3 AND 4 ARE RELATIVELY STIFF COLUMNS-THEY ATTRACT LARGE SEISMIC SHEAR FORCE

Fig. 13 COLUMNS WITH VARYING STIFFNESS

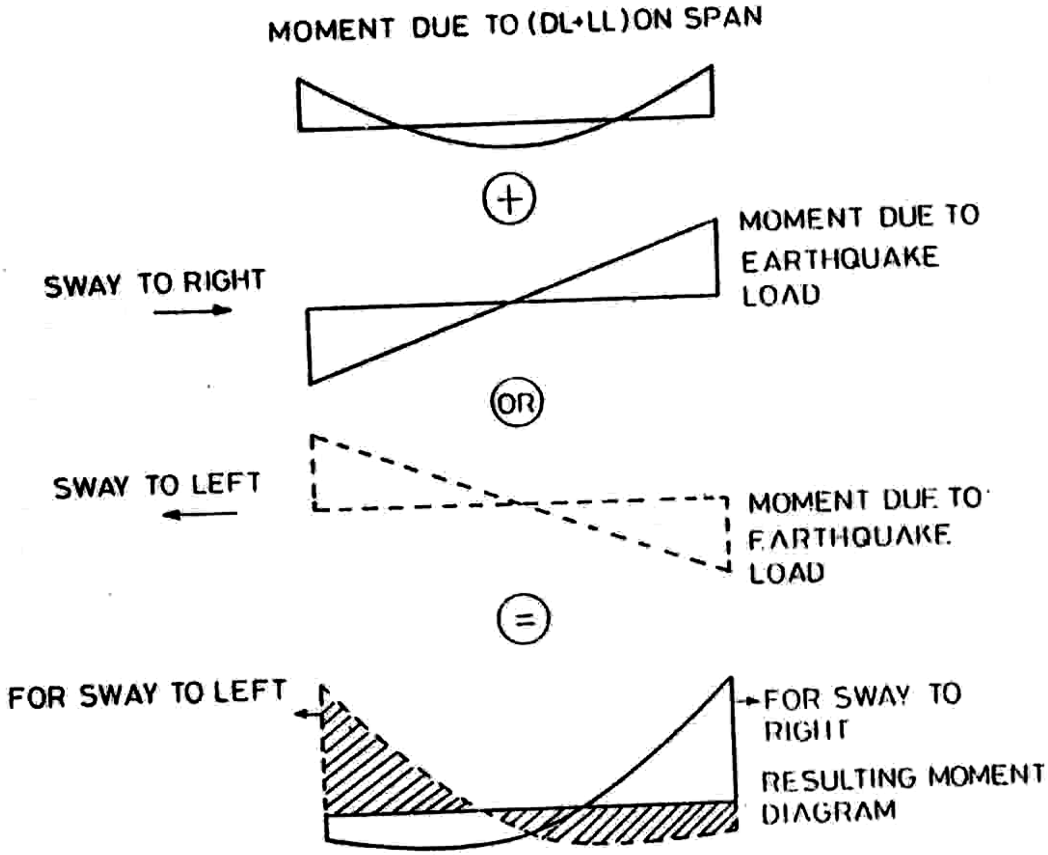


Fig.C1 REVERSAL OF MOMENTS DUE TO EARTHQUAKE LOADING

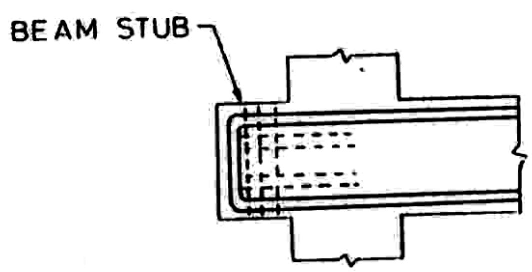


Fig.C2 EXTERIOR BEAM STUB FOR ANCHORING BEAM BARS