SOME VIEWS ON CODE FOR DUCTILE DETAILING FOR SEISMIC DESIGN OF R.C. STRUCTURES (IS : 13920 - 1993)

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ABSTRACT

The paper reviews the code IS: 13920-1993 on ductile detailing of reinforced concrete structures subjected to seismic forces. The basic philosophy behind the codal provisions is highlighted. Weaknesses and shortcomings of the code, which need to be addressed in the next edition, are discussed. Also, typographical and editorial corrections are listed.

INTRODUCTION AND BACKGROUND

The earthquake-resistant design philosophy anticipates that in the event of a severe ground shaking, the structure will undergo some damage, and hence behave inelastically. Therefore, it is important that the structure should respond in a ductile manner beyond its yield point, and this is essentially the objective of ductile detailing. Until recently, IS: 4326-1976 contained some design and detailing provisions for R.C. structures with the objective to achieve good seismic performance; it was felt that those provisions were inadequate (e.g., Patnaik and Jain, 1989). As a result, efforts were made to develop comprehensive codal provisions on the same (Medhekar, Jain and Arya, 1992). Moreover, the provisions were developed for ductile detailing of shear walls (Medhekar and Jain, 1993 a and b). It was then decided that the provisions on ductile detailing of R.C. structures should form an independent code which has now been published by the Bureau of Indian Standards (IS: 13920-1993; hereinafter referred to as "the code"). This paper discusses the basic philosophy behind the code and how these are attempted to be achieved through the various provisions. Moreover, weaknesses and shortcomings of the code are highlighted. Also, typographical and editorial corrections are listed.

EXPECTED RESPONSE OF THE STRUCTURE

The basic expectation is that the structure should exhibit good ductility if in the event of an earthquake shaking it is loaded beyond its yield point. Here, the term "ductility" implies that the structure should be able to undergo large displacements without collapse, even after the yielding has occured (Figure 1). It is important to note that due to inherent overstrength in the structure, the yield point itself may occur at seismic load which is several times higher than the design seismic load (Figure 2) (e.g., Jain and Navin, 1995). An R.C. frame may undergo basically two types of yield mechanisms: beam-hinge mechanism (Figure 3b), or storey mechanism (Figure 3c). Vis-a-vis the beam hinge mechanism, the storey mechanism requires much higher member ductility in

order to have the same overall structural ductility. Moreover, it is much more difficult to achieve the same amount of member ductility in a column than in a beam. Hence, the structure should be designed to yield in a beam-hinge mechanism and the storey mechanism must be avoided. This is possible if the yielding in beam occurs while the columns are still elastic, i.e., the well-known "strong column - weak beam" philosophy.

Structures with regular configuration (i.e., without abrupt changes in mass or stiffness either in plan or in elevation) exhibit much better ductility than similarly detailed irregular structures. IS:1893-1984 prescribes "dynamic analysis" for irregular structures, which in many cases is of no help at all, and in most cases is not a sufficient remedy to the problems caused by irregularity. IS:4326-1993 does provide some requirements for ensuring reasonably regular configurations; however, in most practical cases irregular configurations are difficult to avoid. Hence, IS:13920-1993 recognises that some irregularities are bound to occur in the structural configuration and suggests some remedies in the form of detailing; this, even though not a complete solution to the problem, should improve the seismic response.

Ductility in a structure can be improved by (a) having a regular structural configuration, (b) having more redundancy in its lateral load resisting systems, (c) avoiding failure of columns and foundations, (d) avoiding all possible brittle modes of failure, and (e) by improving the ductility of individual members. The bond and shear failures are brittle modes of failure. Also, the compression failure (over-reinforced beams) is brittle while the tension failure (under-reinforced beams) is ductile. Hence, it must be ensured that (i) bond or shear failure does not precede the flexural failure, and (ii) the member is designed as an under-reinforced section (i.e., with adequate compression reinforcement).

Of the two components that constitute reinforced concrete, the concrete is very brittle with failure strain of about 0.0035, while the reinforcement is ductile and has a failure strain of about 0.15 to 0.25. However, confinement of concrete with longitudinal and transverse reinforcements can increase the failure strain in concrete by several times, besides adding to the maximum stress (e.g., Mander et al. 1988, Saatcioglu and Razvi, 1992). The ductility of a R.C. member increases as (i) concrete grade increases, (ii) steel grade decreases, (iii) concrete confinement increases, (iv) tension reinforcement decreases, (v) compression reinforcement increases, and (vi) axial force in the member decreases. Therefore, the code suggests a minimum grade of concrete as M20 and maximum grade of steel as Fe415.

SOME HIGHLIGHTS OF PROVISIONS ON MOMENT RESISTING FRAMES

Minimum Dimensions
The code imposes some restrictions on the column and beam member sizes. For instance, in case of columns (a) least lateral dimension should be 300 mm unless spans are less than 5 m, and (b) column width to depth ratio should not be less than 0.4. Such restictions have been placed because (a) minimum column dimensions are required to ensure that the beam bars passing through the column can transfer the stress, and (b) confinement of concrete is much better in a square column than in a column with large depth-to width ratio.

Minimum Bottom Face Reinforcement in Beams

Due to the facts that (a) the earthquake force is reversible and (b) the maximum seismic force may exceed the design force by several times, the bottom reinforcement in beams near the joint may undergo tension and compression alternately. Hence, both the top and bottom reinforcements are required to be continuous through the joint. Conventional detailing of the joint, wherein the bottom bars in beams stop just before the column joint, is not acceptable in seismic regions (e.g., Figure 4 which is from SP:34(S&T)-1987). Moreover, the code requires that the bottom face reinforcement at the joint face should not be less than 50% of the top face reinforcement. This ensures two things: (1) adequate compression reinforcement is required for good ductility, and (ii) in the event of strong shaking the beam may have significant positive moment at the joint face requiring adequate bottom-face reinforcement.

Shear Design

To ensure that the shear failure does not precede the flexural failure, seismic codes require that the design shear force be the higher of (a) calculated factored shear force, and (b) maximum shear force that can develop when the beam is undergoing flexural yielding at its two ends. To achieve this, IS:4326-1976 had the following statement: "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." However, this statement was not adequately explained and designers usually did not carry out this calculation. In fact, the term "plastic moment capacity" was not even defined in that code! The IS:13920-1993 states this clause explicitely; in fact, this calculation has been made simpler by taking the plastic moment capacity as 1.4 times the calculated moment capacity. The basis of the multiplier 1.4 is that the plastic moment capacity is usually calculated by assuming the stress in flexural reinforcement as $1.25f_y$ (as against $0.87f_y$ in the moment capacity calculation). A similar clause has also been added for the shear design of columns.

Confinement of Columns with Irregular Configuration

Considering that abrupt changes in stiffness may cause "concentration" of ductility demand, the code prescribes confinement reinforcement (closely-spaced stirrups) throughout the column length when (a) a column supports a stiff element such as a shear wall, or (b) when significant variation in stiffness occurs along the column height, e.g., due to the presence of a mezzanine floor or filler wall panels.

WEAKNESSES IN THE CODE

Strong Column - Weak Girder Philosophy

A major weakness of the code is the absence of the "strong column -weak girder" clause (Figure 5). This clause requires that the columns should have adequate flexural strength with the objective to force hinging in the beams while the columns are still elastic. This usually requires increase in the column size over what one would get in the absence of this clause. In a building with fairly uniform frame, the interior columns tend to have large axial load and negligible gravity moments; this results in most such columns being "overreinforced", i.e., the failure occurs due to crushing of concrete. Most codes do not

specifically require that the columns be designed to lie in the tension failure region of interaction diagram; however, the increased size caused by the "strong column - weak girder" clause perhaps takes care of this problem in most situations. In fact, this clause was included in the original proposal for the code (e.g., Medhekar, Jain, and Arya, 1992). It was dropped at later stages of code development. However, there is a clear need for this clause and it should be included in the next revision of the code.

Joint Detailing

The beam column joint in a moment resisting frame is a very important component and it should be carefully designed. The design provisions in IS:13920 for the joints are rather weak and the major flaws are discussed herein.

External Joints :: .

The code provides that the beam bars be properly anchored in the joint as per Figure 6 which is not sufficient. If the beam bars are of large diameter and the column width is small, this will cause the crushing of compression strut formed by the joint concrete, which is a brittle failure. Hence, we need provisions such as those given in the ACI 318 (Figure 7).

Internal Joints ::

Figure 8 shows the type of force in the beam reinforcement at the joint face. When the lateral loads are dominant, we need adequate column dimension to anchor the beam bars to enable the beam to develop its desired moment strength. Seismic codes require that the ratio of column's width or depth to the beam bar diameter should not be less that around 20; i.e., if beam bars are of 25 mm dia, then the column dimensions should not be less than 500 mm. Similar situation occurs for the beam depth, except that due to large gravity loads, the column bars tend to have large compressive stress which reduces the problem somewhat. Even though the committee ACI 352 recommends such a clause for beam depth also, it has not been included in ACI:318. IS:13920 should include such a provision at least for the column dimensions.

Shear Strength of the Joint ::

Under lateral loads, the joint has to carry large shear force (Figure 9). The code does not have provisions for ensuring appropriate shear strength in the joint region. The U.S. and the Japanese codes prescribe the maximum average shear stress that can be carried by the joint; the contribution of shear stirrups in the joint region is not considered explicitly. As against this, the New Zealand practice is to carry out specific calculation for the shear carried by the concrete and by the shear reinforcement. Either way, we must include provisions for shear design of the joint.

SUMMARY AND CONCLUSION

The code requires major modifications regarding the design and detailing of joints. It should also incorporate the requirement of "strong column -weak beam". These may require use of larger size columns than what the profession in India is used to at the present time. This should not cause unduly serious problems since professionals in many other seismic countries already use such seismic detailing

specifications. Moreover, the difficulties on member sizes of frame members can be alleviated by use of shear walls in the building. Shear wall buildings in general show good seismic performance. In buildings where shear walls provide significant lateral load capacity, two options become available for reducing the burden of seismic detailing on the frames:

(a) The frames may then be designed with ordinary detailing. It may be acceptable even though it will increase the overall design lateral force on the building due to lower value of the "response reduction factor" (e.g., Jain and Murty, 1995).

(b) The frames may be assumed to be non-seismic, i.e., the entire seismic load is assumed to be carried by the shear walls. This will require that the frames should be capable of undergoing same lateral deformation as the shear walls under design load times the "response reduction factor".

APPENDIX :: TYPOGRAPHICAL AND EDITORIAL ERRORS

1. Clause 6.2.1(b) :: The expression for minimum longitudinal reinforcement should read as

$$\rho_{min} = 0.24 \, \frac{\sqrt{f_{ck}}}{f_y} \, ,$$

and not as

$$\rho_{min} = 0.24 \, \sqrt{\frac{f_{ck}}{f_y}} \, .$$

2. Page 2 :: Definition of S should read as

"pitch of spiral or spacing of hoops".

3. To make the nomenclature in the code consistent with that in IS:456-1978, the notation $M_{u,lim}$ should be replaced by M_u (with all other superscripts and subscripts retained as before) in the following instances in the code:

- Clause 4 (Definitions of moments of resistance, on page 2)
- Clause 6.3.3 (Expressions for shear force due to formation of plastic hinges at both ends of the beam plus the factored gravity loads, on page 4))
- Figure 4 (Expressions for calculation of design shear force for beam, on page 5)
- Clause 7.3.4(b) (Expression for factored shear force and the associated description below it, on page 6)
- Figure 8 (In the figure showing moments at beam ends, and in the expression for calculating the design shear force for columns, on page 8)

REFERENCES

ACI Committee 318, Building Code Requirements for Reinforced Concrete (ACI 318-89), American Concrete Institute, Detroit, Michigan, USA.

ACI Committee 352, "Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures," ACI Journal, Proceedings, Vol.82, No.3, May-June 1985, pp 266-283.

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Saatcioglu,M. and Razvi,S.R., 1992, "Strength and Ductility of Confined Concrete," Journal of Structural Engineering, ASCE, 118(6), 1590-1607.
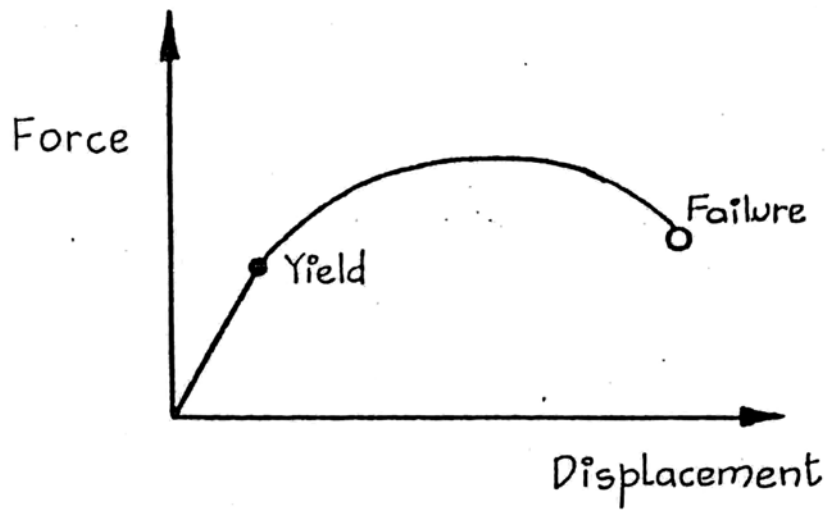
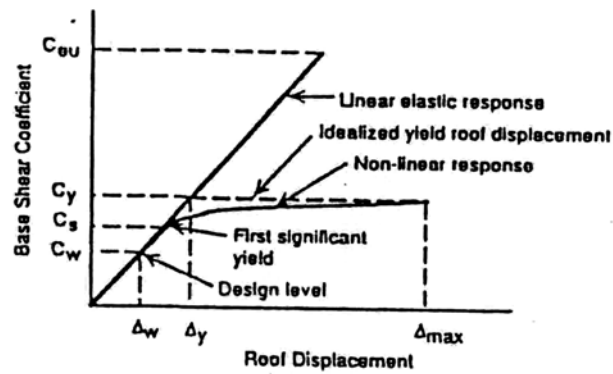Fig. 1: Typical ductile response of a structure.

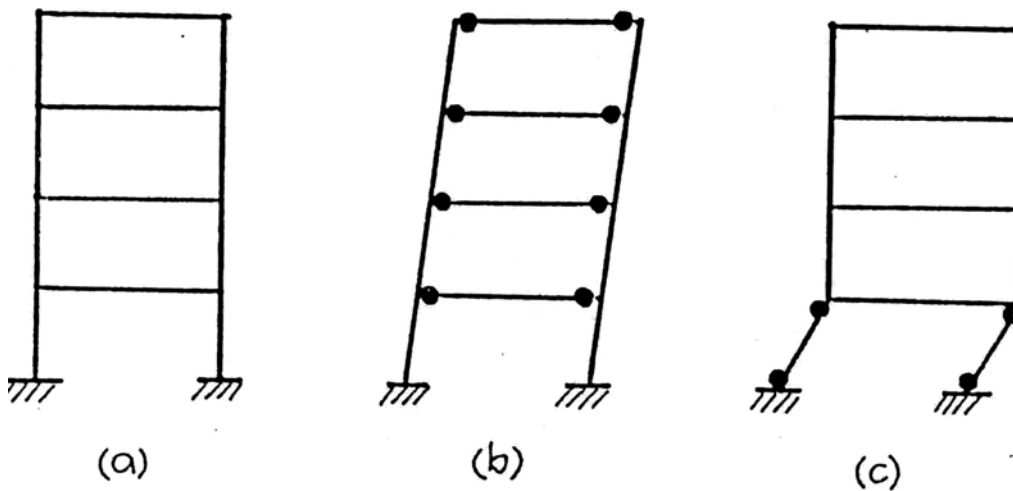Fig. 2: Typical global structure response indicating overstrength in structures.

Fig. 3: Failure mechanisms of frames (a) frame, (b) beam hinge mechanism, and (c) column hinge mechanism.
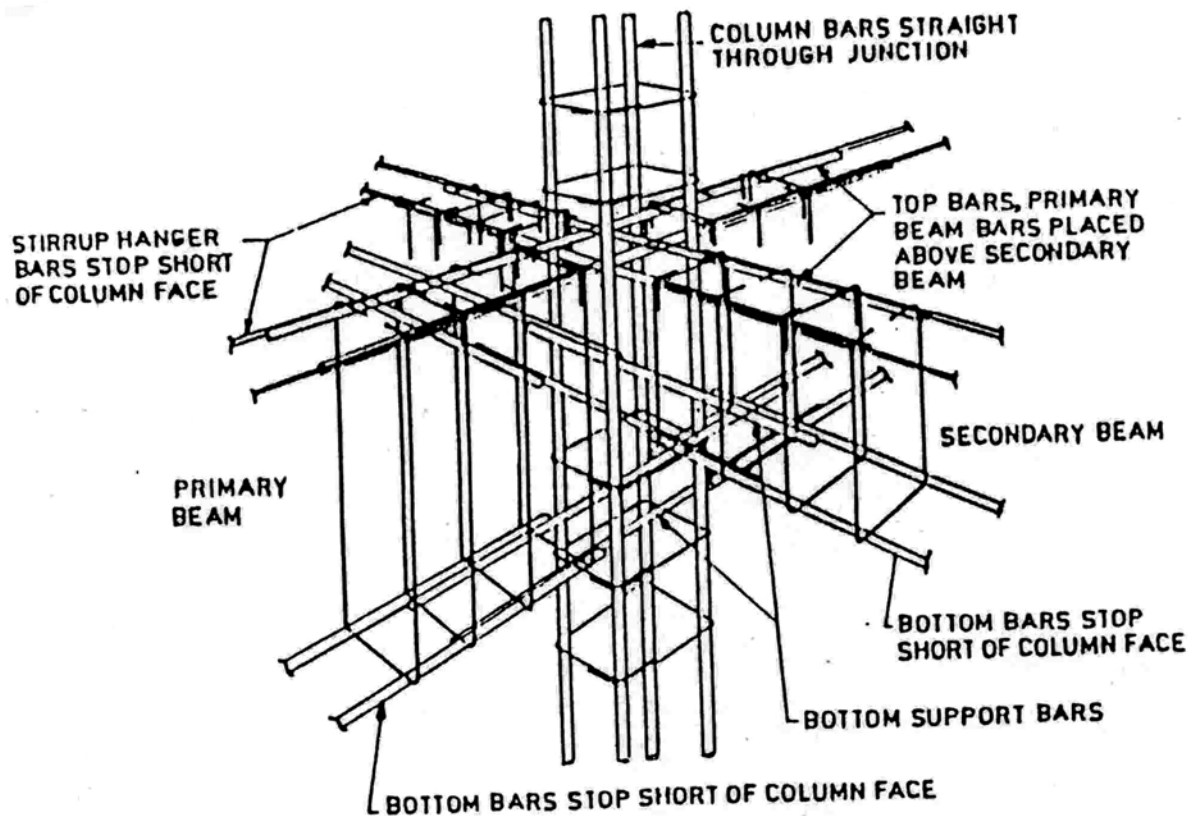
COLUMN BARS STRAIGHT THROUGH JUNCTION

TOP BARS, PRIMARY BEAM BARS PLACED ABOVE SECONDARY BEAM

STIRRUP HANGER BARS STOP SHORT OF COLUMN FACE

SECONDARY BEAM

PRIMARY BEAM

BOTTOM BARS STOP SHORT OF COLUMN FACE

BOTTOM SUPPORT BARS

BOTTOM BARS STOP SHORT OF COLUMN FACE

Fig. 4: Conventional beam-column joint with bottom face bars in beams not continuous through the joint; this is not acceptable in seismic design (Figure from SP:34(S&T)-1987).
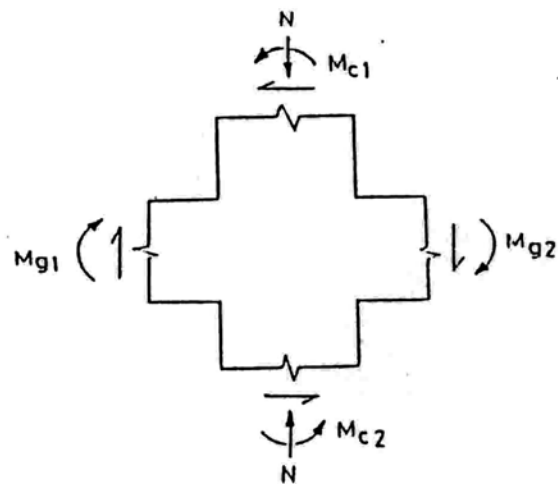
$$\le M_c = M_{c1} + M_{c2}$$

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

$$\le M_c \ge 1.2 \le M_g$$

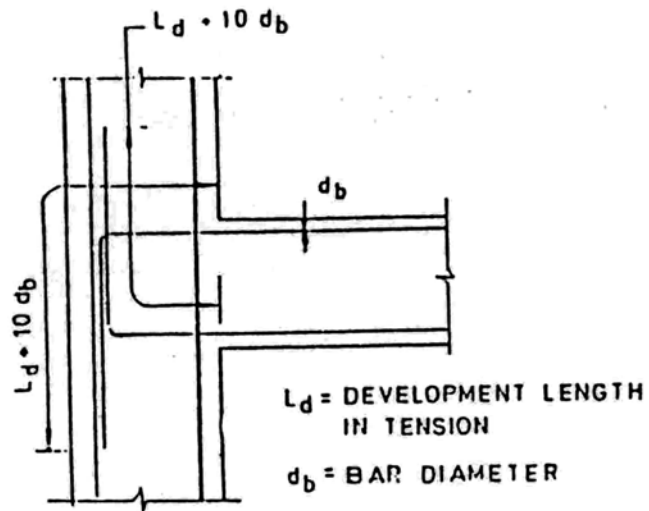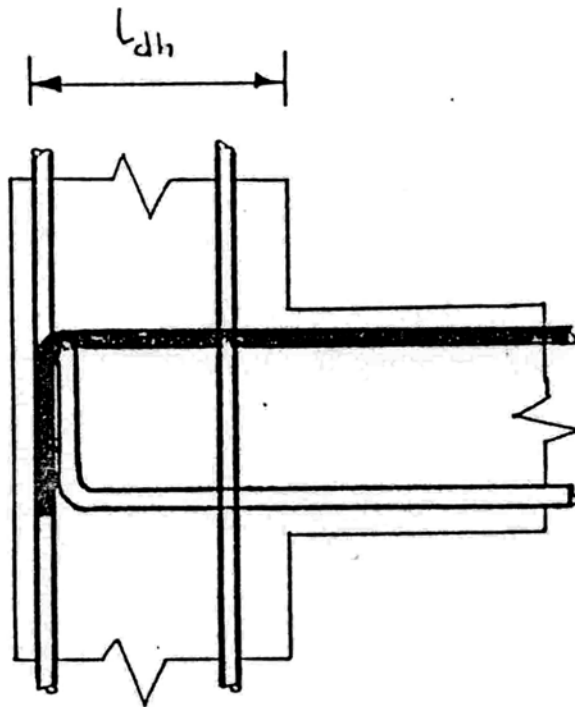Fig. 5: Strong column - weak girder philosophy.

Fig. 6: Anchorage of beam bars in an external joint (Figure from IS: 13920-1993).

$$L_{dh} = \frac{f_y\, d_b}{65\sqrt{f_c'}}$$

Fig. 7: Anchorage of beam bars in an external joint as per ACI: 318-1989

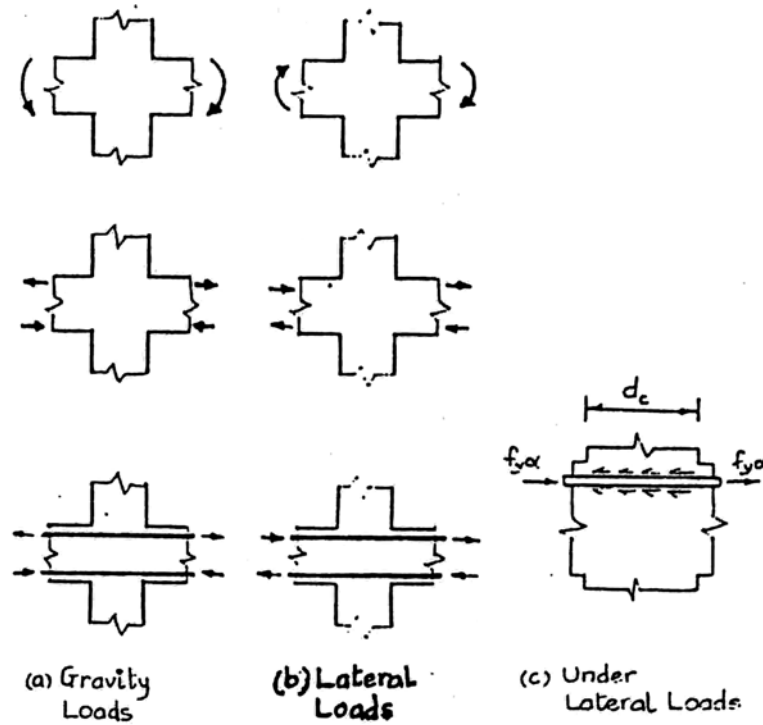(a) Gravity Loads (b) Lateral Loads (c) Under Lateral Loads

Fig. 8: Bar stresses at an interior joint under (a) gravity loads and (b) lateral loads. (c) Demand on anchorage length (through adequate column width and depth) under lateral loads.
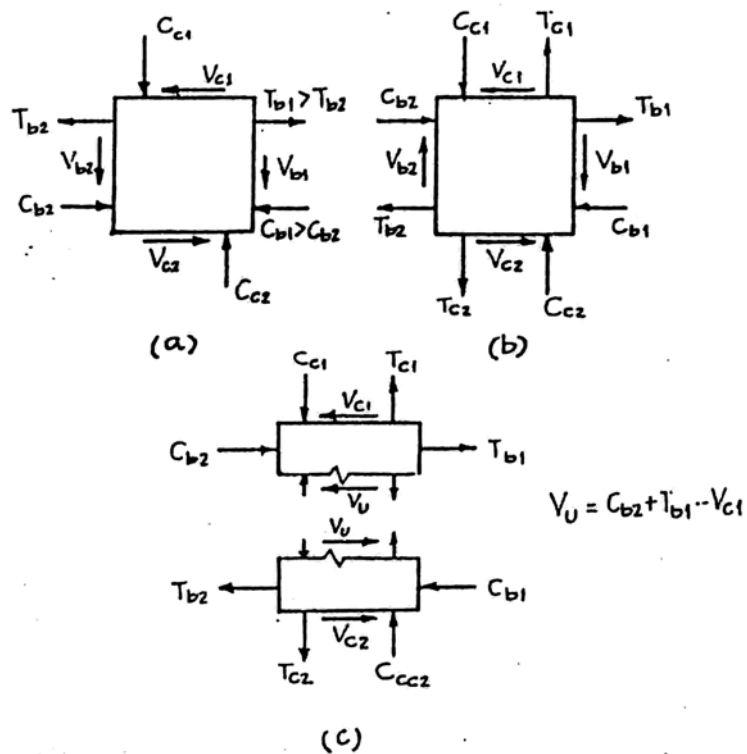

$$V_U = C_{b2} + T_{b1} - V_{c1}$$

Fig. 9: Free body diagrams showing the shear force in the joint (adapted from ACI: 352-1989).