Document No. :: IITK-GSDMA-EQ11-V4.0
:: IITK-GSDMA-EQ16-V3.0
Final Report :: A - Earthquake Codes
IITK-GSDMA Project on Building Codes

Proposed Draft Provisions and Commentary on Ductile Detailing of RC Structures Subjected to Seismic Forces

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Foreword

This Indian Standard was adopted by the Bureau of Indian Standards, after the draft finalized by the Earthquake Engineering Sectional Committee had been approved by the Civil Engineering Division Council.

IS 4326: 1976 'Code of practice for earthquake resistant design and construction of buildings' while covering certain special features for the design and construction of earthquake resistant buildings included some details for achieving ductility in reinforced concrete buildings. With a view to keep abreast of the rapid developments and extensive research that has been carried out in the field of earthquake resistant design of reinforced concrete structures, the technical committee decided to cover provisions for the earthquake resistant design and detailing of reinforced concrete structures separately.

First edition (1993) of this~~This~~ code incorporated~~incorporates~~ a number of important provisions ~~hitherto~~ not covered in IS 4326: 1976. The major thrust in the formulation of this standard was~~is~~ one~~ of~~ the following lines:

a) As a result of the experience gained from the performance, in recent earthquakes, of reinforced concrete structures that were designed and detailed as per IS 4326: 1976, many deficiencies thus identified ~~have been~~were corrected~~ in~~

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~~this code~~.

b) Provisions on detailing of beams and columns ~~have been~~were revised with an aim of providing them with adequate toughness and ductility so as to make them capable of undergoing extensive inelastic deformations and dissipating seismic energy in a stable manner.

c) Specifications on a seismic design and detailing of reinforced concrete shear walls ~~have been~~were included.

The other significant changes incorporated in 1993 edition of the~~this~~ code ~~are~~ were as follows:

a. Material specifications were~~are~~ indicated for lateral force resisting elements of frames.

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

c. For members subjected to axial load and flexure, the dimensional constraints have been 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.

d. Provisions ~~have been~~ were included for estimating the shear strength and flexural strength of shear wall. Provisions ~~are~~ were also given for detailing of reinforcement in the wall web, boundary elements, coupling beams, around openings, at construction joints, and for the development, splicing and anchorage of reinforcement.

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After the Bhuj earthquake of 2001, it was felt that this code needs further improvement. This edition of the code was facilitated through a project entitled "Review of Building Codes and Preparation of Commentary and Handbooks" awarded to IIT Kanpur by the Gujarat State Disaster Management Authority (GSDMA), Gandhinagar through World Bank finances during 2003-2004.

Whilst the common methods of design and construction have been covered in this code, special systems of design and construction of any plain or reinforced concrete structure not covered by this code may be permitted on production of satisfactory evidence regarding their adequacy for seismic performance by analysis or tests or both.

The Sectional Committee responsible for the preparation of this standard has taken into consideration the view of manufacturers, users, engineers, architects, builders and technologists and has related the standard to the practices followed in the country in this field. Due weightage has also been given to the need for international co-ordination among standards prevailing in different seismic regions of the world.

In the formulation of this standard, assistance has been derived from the following publications:

i) ACI 318-02~~89/318R-89~~, Building code requirements for reinforced concrete and commentary, published by American Concrete Institute.

~~ii)ATC-11. Seismic resistance of reinforced concrete shear walls and frame joints: Implications of recent research for design engineers, published by Applied Technology Council, USA.~~

~~iii)CAN3-A23. 3-M84, 1984, Design of concrete structures for buildings, Canadian Standards Association.~~

~~iv)SEADC, 1980, Recommended lateral force requirements and commentary, published by Structural Engineers Association of California, USA~~

ii) IBC 2003 International building code, published by International Code Council,

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Inc.

iii) prEN 1998-1:2003(E)Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings, European Committee for Standardization, Brussels.

iv) NZS 3102:Part 1:1995 Concrete structure standard, published by Standards Council, New Zealand.

v) Medhekar, M.S., Jain, S.K, and Arya, A.S, "Proposed Draft for IS:4326 on Ductile Detailing of Reinforced Concrete Structures," Bulletin of the Indian Society of Earthquake Technology, Vol 29, No. 3, September 1992, 15 - 35.

vi) Medhekar, M.S., and Jain, S.K., "Seismic Behaviour, Design, and Detailing of RC Shear Walls, Part I: Behaviour and Strength," The Indian Concrete Journal, Vol. 67, No. 7, July 1993, 311-318.

vii) Medhekar, M.S., and Jain, S.K., "Seismic Behaviour, Design, and Detailing of RC Shear Walls, Part II: Design and Detailing," The Indian Concrete Journal, Vol. 67, No. 8, September 1993, 451-457.

viii) Sheth, A., "Use of Intermediate RC Moment Frames in Moderate Seismic Zones," The Indian Concrete Journal, Vol. 77, No. 11, November 2003, 1431-1435.

The composition of the technical committees responsible for formulating this standard is given in Annex ~~A~~ B.

~~This edition 1.2 incorporates Amendment No. 1 (November 1995) and Amendment No. 2 (March 2002). Side bar indicates modification of the text as the result of incorporation of the amendments.~~

CODE

1. – Scope

1.1 –

This standard covers the requirements for designing and detailing of monolithic reinforced concrete buildings so as to give them adequate toughness and ductility to resist severe earthquake shocks without collapse.

1.1.1 –

Provisions of this code shall be adopted in all reinforced concrete structures which are located in seismic zone III, IV or V. However:

a) moment resisting frame structures located in seismic zone III are permitted to comply with only section 10 of this code, i.e., as Intermediate Moment Resisting Frame.

b) moment resisting frame structures located in seismic zones IV and V shall comply with sections 5 to 8 of this code, i.e., as Special Moment Resisting Frames.

1.1.2 –

The provisions for reinforced concrete construction given herein apply specifically to monolithic reinforced concrete construction. Precast and/or prestressed concrete members may be used only if they can provide the same level of ductility as that of a monolithic reinforced concrete construction

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C1. – Scope

C1.1 –

The code is targeted at buildings even though its title says “structures”. There will be cases of large size bridge piers where the code may be inadequate.

C1.1.1 –

Originally, IS 13920 requirements were mandatory only for all structures in zones IV and V, and for important buildings, industrial structures and more than 5 storey buildings in zone III. Many reinforced concrete buildings in Ahamedabad (zone III) collapsed in the 2001 Bhuj earthquake. Therefore, ductile requirements were made mandatory for all structures in zone III, IV and V. However, ductile detailing requires substantially higher effort in design, construction, and quality control. Hence it is desirable to have the option for zone III to provide lower level of ductility.

Members where load combinations involving earthquake load do not govern the design, this code should still be followed, for instance, when wind loads are higher than seismic loads. This issue is clarified in last paragraph of Clause 6.1.3 of IS 1893: 2002 which states, “*The specified earthquake loads are based upon post-elastic energy dissipation in the structure and because of this fact, the provision of this standard for design, detailing and construction shall be satisfied even for structures and members for which load combinations that do not contain the earthquake effect indicate larger demands than combinations including earthquake.*”

This code is to be followed only for the lateral load resisting system. For example, the secondary beams do not require ductile detailing.

C1.1.2 –

Until Indian codal provisions are developed for precast and prestressed structures, specialist literature should be referred to. Listed below are some of the relevant references:

1. Englekirk, R.E., *Seismic Design of Reinforced and Precast Concrete Buildings*,

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during or after an earthquake. Specialist literature must be referred to for such structures.

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John Wiley & Sons, Inc., 2003.

2. *Guidelines for the Use of Structural Precast Concrete Buildings*, Center for Advanced Engineering, University of Canterbury Christchurch, New Zealand, December 1999.
3. Park, R., "Precast Concrete in Seismic Resisting Buildings in New Zealand", *Concrete International*, American Concrete Institute, Vol. 12, No. 11, 1990, 43-51.
4. *PCI Design Hand Book for Precast and Prestressed Concrete*, Precast and Prestressed Concrete Institute, Chicago, 1999.

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2. – References

2.1 –

The Indian Standards listed below are necessary adjunct to this standard:

IS No.	*Title*
456: 2000 ~~1978~~	Code of practice for plain and reinforced concrete (fourth ~~third~~ revision)
1786: 1985	Specification for high strength deformed steel bars and wires for concrete reinforcement (third revision)
1893: ~~1984~~(Part 1) 2002	Criteria for earthquake design of structures (~~fourth~~ fifth revision)

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3. – Terminology

For the purpose of this standard, the following definitions shall apply.

3.1 – Boundary Elements

Portions along the edges of a shear wall that are strengthened by longitudinal and transverse reinforcement. They may have the same thickness as that of the wall web.

3.2 – Crosstie

Is a continuous bar having a 135^0 hook with a 6~~10~~-diameter extension (but not < 65~~75~~ mm) at one~~each~~ end and a hook not less than 90^0 with a 6 diameter extension (but not < 65 mm) at the other end. The hooks shall engage peripheral longitudinal bars. In general, the 90^0 hooks of two successive crossties engaging the same longitudinal bars shall be alternated end for end.

3.3 – Curvature Ductility

Is the ratio of curvature at the ultimate strength of the section to the curvature at first yield of tension steel in the section.

3.4 – Gravity Column

Columns which are not part of the lateral load resisting system and are designed only for gravity loads. However, they should be able to resist the gravity loads at lateral displacement imposed by the earthquake forces.

~~3.4~~3.5 – Hoop

Is a closed stirrup having a 135^0 hook with a 6~~10~~-diameter extension (but not < 65~~75~~ mm) at each end that is embedded in the confined core of the section. It may also be made of two pieces of reinforcement; a U-stirrup with a 135^0 hook and a 6~~10~~-diameter extension (but not < 65~~75~~ mm) at each end, embedded in the confined core and a crosstie.

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~~3.5~~3.6 – Lateral Force Resisting System

Is that part of the structural system which resists the forces induced by earthquake.

~~3.6~~3.7 – Shear Wall

A wall that is primarily designed to resist lateral forces in its own plane.

~~3.7~~3.8 – Shell Concrete

Concrete that is not confined by transverse reinforcement, is also called concrete cover.

~~3.8~~3.9 – Space Frame

A three dimensional structural system composed of interconnected members, without shear or bearing walls, so as to function as a complete self-contained unit with or without the aid of horizontal diaphragms or floor bracing systems.

~~3.8.1~~3.9.1 – Vertical Load Carrying Space Frame

A space frame designed to carry all vertical loads.

~~3.8.2~~3.9.2 – Moment Resisting Space Frame

A vertical load carrying space frame in which the members and joints are capable of resisting forces primarily by flexure.

3.9.2.1 – Special Moment Resisting Frame (SMRF)

It is a moment-resisting frame specially detailed to provide ductile behaviour as per the requirements specified in 5., 6., 7. and 8. of this standard.

C3.9.2.1 –
SMRFs are expected to provide ductile behaviour desired in earthquake resistant structures. All moment resisting frame structures in zones IV and V must comply with requirements of SMRF.

3.9.2.2 – Intermediate Moment Resisting Frame

C3.9.2.2 –
Moment Resisting Frame structures in zone III may comply with requirements either as SMRF or

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It is a moment resisting frame detailed as per 10. of this standard.

3.9.2.3 – Ordinary Moment Resisting Frame (OMRF)

It is a moment-resisting frame not meeting special detailing requirements for ductile behaviour specified in IS: 13920.

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as IMRF.

IMRFs are expected to possess ductility that is intermediate between that of OMRFs and SMRFs.

C3.9.2.3 –
OMRFs are those designed as per IS:456-2000. Engineer has the option to design moment resisting frame structures in seismic zone II as OMRF, IMRF, or SMRF.

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4. – Symbols

For the purpose of this standard, the following letter symbols shall have the meaning indicated against each; where other symbols are used, they are explained at the appropriate place. All dimensions are in mm, loads in Newton and stresses in MPa (N/sq mm) unless otherwise specified.

A_e	effective cross sectional area of a joint
A_{ej}	effective shear area of a joint
A_g	gross cross sectional area of column, wall
A_h	horizontal reinforcement area within spacing S_v
A_k	area of concrete core of column
A_{sd}	reinforcement along each diagonal of coupling beam
A_{sh}	area of cross section of bar forming spiral or hoop
A_{st}	area of uniformly distributed vertical reinforcement
A_v	vertical reinforcement at a joint
b_b	width of beam
b_c	width of column
b_j	effective width of a joint
C_w	centre to centre distance between boundary elements
D	overall depth of beam
D_k	diameter of column core measured to the outside of spiral or hoop
d	effective depth of member
d_w	effective depth of wall section
E_s	elastic modulus of steel
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_c	depth of column

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h_j	effective depth of a joint
h_{st}	storey height
L_{AB}	clear span of beam
L_d	development length of bar in tension
l_o	length of member over which special confining reinforcement is to be provided
l_w	horizontal length of wall
l_s	clear span of couplings beam
M_u	factored design moment on entire wall section
M_{c1}	Moment of resistance of column section
M_{c2}	Moment of resistance of column section
M_{g1}	Moment of resistance of beam section
M_{g2}	Moment of resistance of beam section
$\cancel{M_{u,lim}^{Ah}}$ M_u^{Ah}	hogging moment of resistance of beam at end A
$\cancel{M_{u,lim}^{As}}$ M_u^{As}	sagging moment of resistance of beam at end A
$\cancel{M_{u,lim}^{Bh}}$ M_u^{Bh}	hogging moment of resistance of beam at end B
$\cancel{M_{u,lim}^{Bs}}$ M_u^{Bs}	sagging moment of resistance of beam at end B
$\cancel{M_{u,lim}^{b_L}}$ $M_u^{b_L}$	moment of resistance of beam framing into column from the left
$\cancel{M_{u,lim}^{b_R}}$ $M_u^{b_R}$	moment of resistance of beam framing into column from the right
M_{uv}	flexural strength of wall web
P_u	factored axial load
S	pitch of spiral or spacing of hoops

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S_v vertical spacing of horizontal reinforcement in web

t_w thickness of wall web

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

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

V_j shear resistance at a joint

V_u factored shear force

V_{us} shear force to be resisted by reinforcement

x_u, x_u^* depth of neutral axis from extreme compression fibre

$\sum M_c$ Sum of moment of resistance of columns meeting at a joint

$\sum M_g$ Sum of moment of resistance of beams framing into a joint from opposite faces

α inclination of diagonal reinforcement in coupling beam

ρ vertical reinforcement ratio

ρ_c compression reinforcement ratio in beam

ρ_{max} maximum tension reinforcement ratio for a beam

ρ_{min} minimum tension reinforcement ratio for a beam

τ_c shear strength of concrete

$\tau_{c,\,max}$ maximum permissible shear stress in section

τ_v Nominal shear stress

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5. – General Specification

C5. – General Specification

5.1 –

The design and construction of reinforced concrete buildings shall be governed by the provisions of IS 456: ~~1978~~2000, except as modified by the provisions of this code.

C5.1 –

One should note that provisions of IS 456 are still valid for earthquake resistance structures, and IS 13920 provisions are over and above those of IS 456.

5.2 –

~~For all buildings which are more than 3 storeys in height, the m~~Minimum grade of concrete shall be M20 (f_{ck} = 20 MPa). However, for all buildings which are either more than 4 storeys or more than 15m in height in Zones IV and V, minimum grade of concrete shall be M25.

C5.2 –

1978 version of IS 456 allowed M15 Grade concrete, but minimum grade of concrete as per IS 456: 2000 is restricted to M20.

Higher grade of concrete facilitates ductile behaviour. Most of the codes world wide specify higher grade of concrete for seismic regions than that for non-seismic constructions. For example; (i) ACI allows M20 for ordinary constructions, but a minimum of M25 for seismic constructions. (ii) Euro code allows M15 for non seismic, but requires a minimum grade of M20 for low-seismic and M25 for medium and high seismic regions.

It is therefore reasonable to require minimum M25 grade of concrete in zones IV and V for buildings higher than 4 storeys.

5.3 –

Steel reinforcement resisting earthquake-induced forces in frame members and in boundary elements of shear walls shall comply with 5.3.1, 5.3.2 and 5.3.3.

5.3.1 ~~5.3~~–

Steel reinforcements of grade Fe 415 (*see* IS 1786: 1985) or less only shall be used.

However, high strength deformed steel bars, produced by the thermo-mechanical treatment process, of grades Fe 500 and Fe 550, having elongation more than 14.5 percent and conforming to other requirements of IS 1786: 1985 may also be used for the reinforcement.

C5.3.1 –

Higher grade of steel reduces ductility and hence 1993 version of the code required that steel reinforcement of grade Fe415 or less only be used. Later, the code permitted use of Fe 500 and Fe 550 subjected to the restrictions on ductility of bars.

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5.3.2

The actual yield strength based on tensile test of steel must not exceed the specified yield strength by more than 120 MPa.

5.3.3 –

The ratio of the actual ultimate strength to the actual yield strength should be at least 1.25.

5.4 –

In RC frame buildings, lintel beams shall not be integrated into the columns, unless it is shown that the introduction of the reinforcement of the lintel beam does not weaken the column during construction.

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C5.3.2 –

If the difference of actual yield strength and specified yield strength is very high, the shear or bond failure may precede the flexural hinge formation, and the capacity design concept may not work. Hence, a restriction is imposed on the maximum difference allowed between the actual yield strength and the specified yield strength of steel.

C5.3.3 –

To develop inelastic rotation capacity, a structural member needs adequate length of yield region along axis of the member. The larger the ratio of ultimate to yield moment, the longer is the yield region. Therefore, the code requires that the ratio of actual tensile strength to actual yield strength be not less than 1.25. Members with reinforcement not satisfying this condition can also develop inelastic rotation, but their behaviour is quite different so as to exclude them from provisions of this code.

C5.4 –

Use of lintel bands is a concept for masonry structures. The use of lintel integrated with the columns in RC frame buildings may weaken the columns due to constructional difficulties.

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6. – Flexural Members

C6. – Flexural Members

6.1 – General

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

6.1.1 –

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

C6.1.1 –

If the factored axial stress exceeds the specified limit, the member will be considered to be in significant compression, and will be detailed as per clause 7.

6.1.2 –

The member shall preferably have a width-to-depth ratio of more than 0.3.

C6.1.2 –

Clause 6.1.2 restricts the applicability of these ductility provisions to normally proportioned members. This is because there is a lack of experimental data on the behaviour of flexural members having very low width-to-depth ratio under cyclic inelastic deformations. Also, it is difficult to confine concrete through stirrups in narrow beams, which exhibit a poor performance in comparison to well-confined concrete.

6.1.3 –

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

b) Width of beam shall not be more than the width of the column (measured on a plane perpendicular to the longitudinal axis of the beam) plus distances on each side of the column not exceeding half of the depth of the beam. Also, the width of beam shall not be more than two times the width of column (b_c), in the considered direction, under any circumstances.

$$b_w \leq \min\left[(b_c + h_w);(2b_c)\right]$$

Where b_w is the width and h_w is the depth of the beam.

C6.1.3 –

Where wide beams frame into columns, an upper limit on the width of beams is specified to ensure that the longitudinal beam steel needed for seismic forces is kept reasonably close to the column core. At least 75% of the effective longitudinal beam reinforcement must pass through or be anchored within the column core. This will ensure the favourable effect of column compression on the bond of horizontal bars passing through the joint.

6.1.4 –

The depth D of the member shall preferably be not more than 1/4 of the clear span.

C6.1.4 –

When the ratio of total depth of a member to the clear span is greater, the member behaves like a deep beam. The behaviour of deep beam is

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significantly different from that of the relatively slender members under cyclic inelastic deformations. Therefore, design rules for relatively slender members do not apply directly to members with length-to-depth ratio less than four, especially with respect to shear strength. Restriction on total depth is imposed as clause 6 applies to slender members only.

6.2 – Longitudinal Reinforcement

6.2.1 –

a) The top as well as bottom reinforcement shall consist of at least two bars of 12 mm diameter throughout the member length.

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.

C6.2 – Longitudinal Reinforcement

C6.2.1 –

a) Under the effect of earthquake forces, the zone of moment reversal may extend for a considerable distance towards midspan. Therefore, the code recommends at least two bars of 12 mm diameter throughout the member length.

b) This provision is derived from following considerations:

At small loads on an RC member, entire concrete section participates. As load increases, a point comes when tension cracks develop in concrete. It must then transfer the tensile force to the reinforcement present in the tension region. If the tension steel available is not adequate to carry the tensile force transferred by the concrete upon cracking, the section will fail suddenly causing a brittle failure. This clause is meant to ensure adequate tension reinforcement to take the tensile force that was carried by the concrete prior to cracking.

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 is greater than the cracking moment of the section. However, cantilever T-beams with flange in tension will require significantly higher reinforcement than specified in this clause to prevent brittle failure.

Derivation of this equation and more detailed discussion can be found in :

Medhekar, M.S., and Jain, S.K., "Proposed Minimum Reinforcement Requirements for

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Flexural Members," *The Bridge and Structural Engineer, ING-IABSE*, Vol. 23, No. 2, June 1993, 77-88.

6.2.2 –

The maximum steel ratio on any face at any section, shall not exceed ρ_{max} = 0.025.

C6.2.2 –

This provision is primarily to avoid congestion of reinforcement, as that may cause insufficient compaction or a poor bond between reinforcement and concrete. This is a fairly generous limit and in most situations one will use lower amount of reinforcement.

6.2.3 –

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

C6.2.3 –

This provision covers two aspects:

- The seismic moments are reversible. Further, design seismic loads may be exceeded by a considerable margin during strong earthquake shaking. Therefore, substantial sagging moment may develop at beam ends during strong shaking which may not be reflected through analysis (Fig. C1).

  Example: Let us say at the beam end, gravity moment = –500 kNm, seismic moment = ±700 kNm. The analysis, therefore, will indicate a hogging moment of 1200 kNm and sagging moment of 200 kNm. Application of this clause ensures capacity of 1200 kNm in hogging and 600 kNm in sagging. During earthquake shaking, actual seismic moment may be higher, say ±1400 kNm under elastic behaviour; in this case, gravity plus seismic will be –1900 kNm and +900 kNm. Note that design negative moment has increased from 1200 kNm to 1900 kNm (58%) but positive moment from 200 kNm to 900 kNm (450%). Hence, this clause is crucial under moment reversal.

- Compression reinforcement increases ductility, and this clause ensures adequate compression reinforcement at the locations of potential yielding.

Fig. C2 shows an example for the application of this clause.

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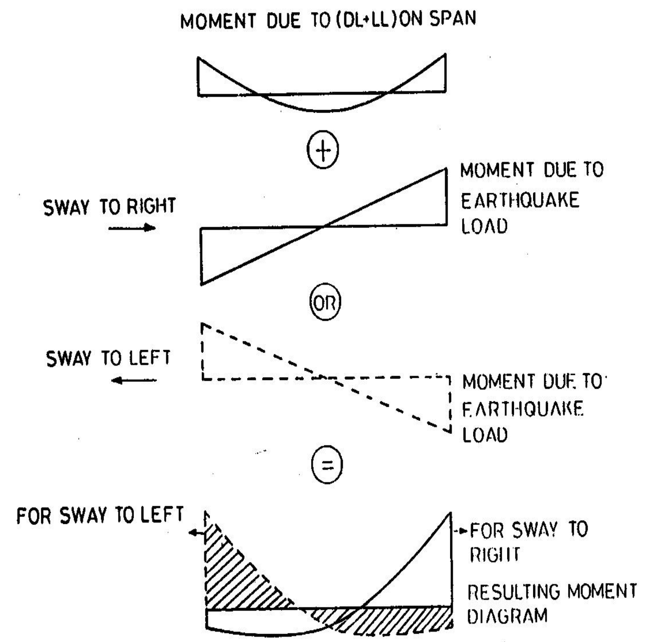

Figure C1 – REVERSAL OF MOMENTS DUE TO EARTHQUAKE LOADING

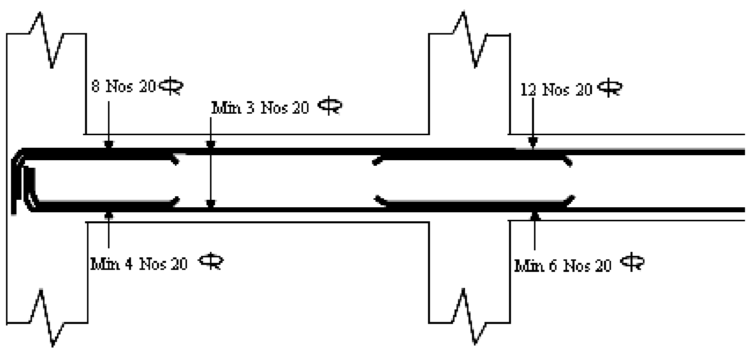

Figure C 2 – Longitudinal reinforcement at the joints in a beam

6.2.4 –

The steel provided at each of the top and bottom face of the member at any section along its length shall be at least equal to one-fourth of the maximum negative moment steel provided at the face of either joint. ~~It may be clarified that redistribution of moments permitted in IS 456: 1978~~ 2000 ~~(clause **36.1**) will be used only for vertical load moments and not for lateral load moments.~~

C6.2.4 –

Sufficient reinforcement should be available at any section along the length of the member to take care of reversal of loads or unexpected bending moment distribution. Hence, the code specifies the steel to be provided at each of the top and bottom face of the member at any section along its length as some fraction of the maximum negative moment steel provided at the face of either joint. Fig. C2 is an example for the application of this clause.

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6.2.5 –

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 minus the allowance for 90 degree bend(s) (see Fig.1). In an internal joint, both face bars of the beam shall be taken continuously through the column.

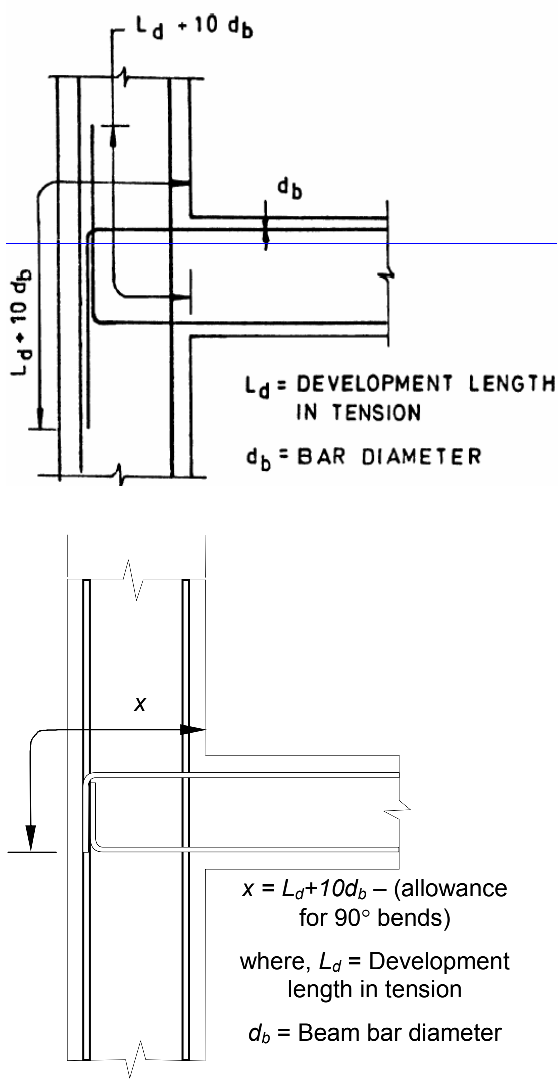

Figure 1 – Anchorage of beam bars in an external joint

6.2.6 – Lap Splices

The longitudinal bars shall be spliced, only if hoops are provided over the entire splice length, at a spacing not exceeding 150 mm

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C6.2.5 –

During an earthquake, the zone of inelastic deformation that exists at the end of a beam, may extend for some distance into 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 diameter of bar from the inner face of the column.

The extension of top bars of beam into column below soffit of the beam causes construction problem, as one would cast the columns up to beam soffit level before fixing the beam reinforcement. If column is wide enough to satisfy the anchorage requirement within the beam column joint, the above mentioned construction problem will not arise. Therefore, it is important to use adequate depth of the column members.

C6.2.6 –

Lap splices are not reliable under cyclic inelastic deformations, and hence, should not to be provided in critical regions. Closely spaced hoops

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(see Fig. 2). The lap length shall not be less than the bar development length in tension. Lap splices shall not be provided (a) within a joint, (b) within a distance of 2*d* from joint face, and (c) within a quarter length of the member where flexural yielding may generally occur under the effect of earthquake forces. Not more than 50 percent of the bars shall be spliced at one section.

help improve performance of splice when cover concrete spalls off.

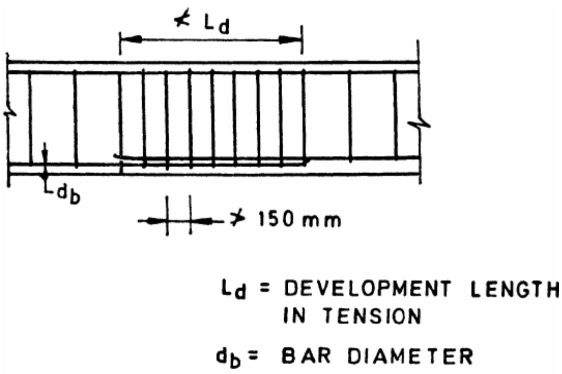


Figure 2 – Lap~~,~~ splice in beam

6.2.7 –

6.2.8 Redistribution of moments (clause 37.1 of IS: 456-2000) will be restricted to 10 percent)

~~6.2.7~~6.2.9 –

Use of welded splices and mechanical connections may also be made, as per **26.2.5.2~~25.2.5.2~~** of IS 456: 2000~~1978.~~ provided not more than alternate bars in each layer of longitudinal reinforcement are spliced at a section and the centre to centre distance between splices of adjacent bars is 600 mm or more measured along the longitudinal axis of the member. However, welding of stirrups, ties, inserts or other similar elements to longitudinal reinforcement required by design shall not be permitted. ~~However, not more than half the reinforcement shall be spliced at a section where flexural yielding may take place.~~ The location of splices shall be governed by **6.2.6.**

C6.2.9 –

Welding of stirrups, ties or other similar elements to longitudinal reinforcement can lead to local embrittlement of the steel. If welding of these bars to longitudinal bars is required to facilitate fabrication or placement of reinforcement, it should be done only on bars added for such purposes.

CODE

6.3 – Web Reinforcement

6.3.1 –

Web reinforcement shall consist of vertical hoops. A vertical hoop is a closed stirrup having a 135^0 hook with a 6~~10~~ diameter extension (but not < 65~~75~~ mm) at each end that is embedded in the confined core *(see* Fig. 3a). In compelling circumstances, it may also be made up of two pieces of reinforcement; a U-stirrup with a 135^0 hook and a 6~~10~~ diameter extension (but not < 65~~75~~ mm) at each end, embedded in the confined core and a cross tie (*see* Fig. 3b). A crosstie is a bar having a 135^0 hook at one end and 90^0 hook at other end with a ~~10~~ 6 diameter extension (but not< ~~75~~ 65 mm) at each end. The hooks shall engage peripheral longitudinal bars. Consecutive crossties engaging the same longitudinal bars shall have their 90^0 hooks at opposite sides of the flexural member. If the longitudinal reinforcement bars secured by the crossties are confined by a slab on only one side of the flexural frame member, the 90^0 hooks of the crossties shall be placed on that side.

COMMENTARY

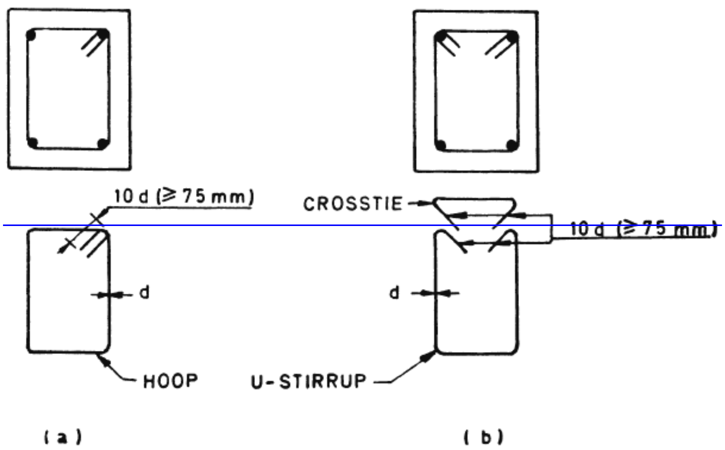
C6.3 – Web Reinforcement

Stirrups in RC beams help in three ways, namely (i) they carry the vertical shear force and thereby prevent the diagonal shear cracks, (ii) they confine the concrete, and (iii) they prevent the buckling of the compression bars.

C6.3.1 –

Vertical hoops should be bent into a 135^0 hook and extended sufficiently into the confined concrete beyond this hook to ensure that the stirrup does not open out during strong earthquake shaking. 1993 edition of the code required 10 diameter (but not < 75 mm) as extension which is changed to 6 diameter (but not < 65 mm). Large value of extension leads to considerable construction difficulties. Laboratory testing in the United States has shown that 6 diameter extension may be adequate. As a result, the ACI 318 has changed the requirement of 10 diameter extension to 6 diameter extension.

Crossties with a 90° hook are not as effective as either crossties with 135° hook or hoops in providing confinement. Construction problem arises in placing crossties with 135° hooks at both ends. Tests show that if crosstie ends with 90° hooks are alternated, confinement will be sufficient. Therefore, it is allowed to use cross ties with 90° hook at one end and 135° hook at other end.



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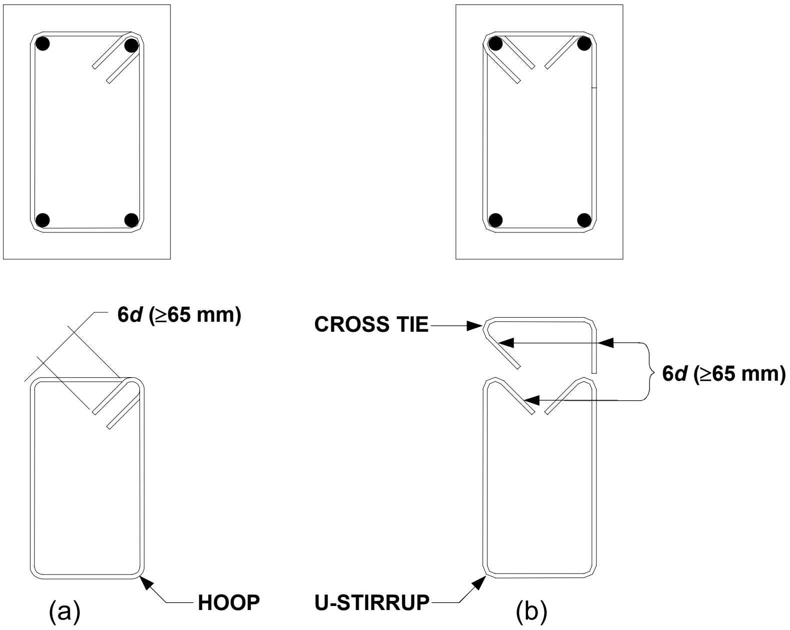


Figure 3 – Beam web reinforcement

6.3.2 –

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.

C6.3.2 –

The minimum bar diameter for hoop reinforcement in beams is specified to ensure a minimum ductility and to prevent local buckling of longitudinal compression bars.

6.3.3 –

The shear ~~force to be resisted by the vertical hoops~~capacity of the beam shall be ~~the maximum of~~more than:

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. This is given by (*see* Fig. 4)

i) for sway to right:

$$\cancel{V_{u,b} = V_a^{D+L} \quad 1.4\left[\frac{M_{u,lim}^{As} + M_{u,lim}^{Bh}}{L_{AB}}\right]}$$ ~~and~~

$$\cancel{V_{u,b} = V_b^{D+L} \quad 1.4\left[\frac{M_{u,lim}^{As} + M_{u,lim}^{Bh}}{L_{AB}}\right]}$$ ~~, and~~

$$V_{u,a} = V_a^{D+L} - 1.4\left[\frac{M_u^{As} + M_u^{Bh}}{L_{AB}}\right]$$ and

C6.3.3 –

This clause ensures that a brittle shear failure does not precede the actual yielding of the beam in flexure. Clause 6.3.3(b) simplifies the process of calculating plastic moment capacity of a section by taking it to be 1.4 times the calculated moment capacity with usual partial safety factors. This factor of 1.4 is based on the consideration that plastic moment capacity of a section is usually calculated by assuming the stress in flexural reinforcement as 1.25 f_y as against $0.87f_y$ in the moment capacity calculation.

The notation $M_{u,lim}$ used in 1993 edition of the code was not consistent with IS 456: 2000 and to ensure consistency, the earlier notation of $M_{u,lim}$ has now been replaced by M_u.

CODE

COMMENTARY

$$V_{u,b} = V_b^{D+L} + 1.4\left[\frac{M_u^{As} + M_u^{Bh}}{L_{AB}}\right], \text{ and}$$

ii) for sway to left:

$$\cancel{V_{u,a} = V_a^{D+L} - 1.4\left[\frac{M_{u,lim}^{Ah} + M_{u,lim}^{Bs}}{L_{AB}}\right] \text{ and}}$$

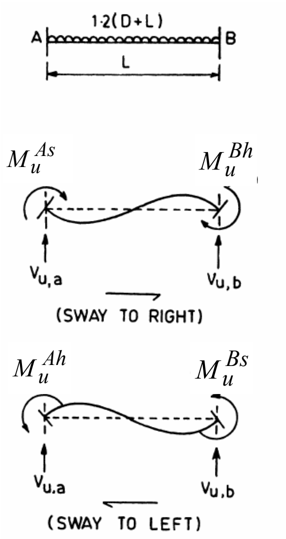
$$\cancel{V_{u,a} = V_a^{D+L} - 1.4\left[\frac{M_{u,lim}^{Ah} + M_{u,lim}^{Bs}}{L_{AB}}\right],}$$

$$V_{u,a} = V_a^{D+L} - 1.4\left[\frac{M_u^{Ah} + M_u^{Bs}}{L_{AB}}\right] \text{ and}$$

$$V_{u,b} = V_b^{D+L} + 1.4\left[\frac{M_u^{Ah} + M_u^{Bs}}{L_{AB}}\right],$$

where $\cancel{M_{u,lim}^{As}, M_{u,lim}^{Ah}}$ and $\cancel{M_{u,lim}^{Bs}, M_{u,lim}^{Bh}}$ M_u^{As}, M_u^{Ah} and M_u^{Bs}, M_u^{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~~2000. 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 partial safety factor of 1.2 on loads. The design shear at end A shall be the larger of the two values of $V_{u,a}$ computed above. Similarly, the design shear at end B shall be the larger of the two values of $V_{u,b}$ computed above.

CODE | COMMENTARY



$$V_{u,a} = V_a^{D+L} - 1.4\left[\frac{M_u^{As} + M_u^{Bh}}{L_{AB}}\right]$$

$$V_{u,b} = V_b^{D+L} + 1.4\left[\frac{M_u^{As} + M_u^{Bh}}{L_{AB}}\right]$$

$$V_{u,a} = V_a^{D+L} + 1.4\left[\frac{M_u^{Ah} + M_u^{Bs}}{L_{AB}}\right]$$

$$V_{u,b} = V_b^{D+L} - 1.4\left[\frac{M_u^{Ah} + M_u^{Bs}}{L_{AB}}\right]$$

Figure 4 – Calculation of design shear force for beam

6.3.4 –

The contribution of bent up bars and inclined hoops to shear resistance of the section shall not be considered.

C6.3.4 –

Due to cyclic nature of seismic loads, shear force can change direction. The inclined hoops and bent up bars, effective in one direction for resisting shear force, will not be effective for opposite direction of shear force.

6.3.5 –

The spacing of hoops over a length of 2*d* at either end of a beam shall not exceed (a) *d/4*, and (b) 8 times the diameter of the smallest longitudinal bar; however, it need not be less than 100 mm (*see* 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 2*d* 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.

C6.3.5 –

Closely spaced hoops at the two ends of the beam are recommended to obtain large energy dissipation capacity and better confinement.

To ensure space for needle vibrator, the minimum hoop spacing has been restricted to 100 mm.

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. IS 456 allows *3d/4* as against the requirement of *d/2* in this clause. One should bear in mind that the provisions of IS 13920 are over and above those in IS 456.

CODE **COMMENTARY**

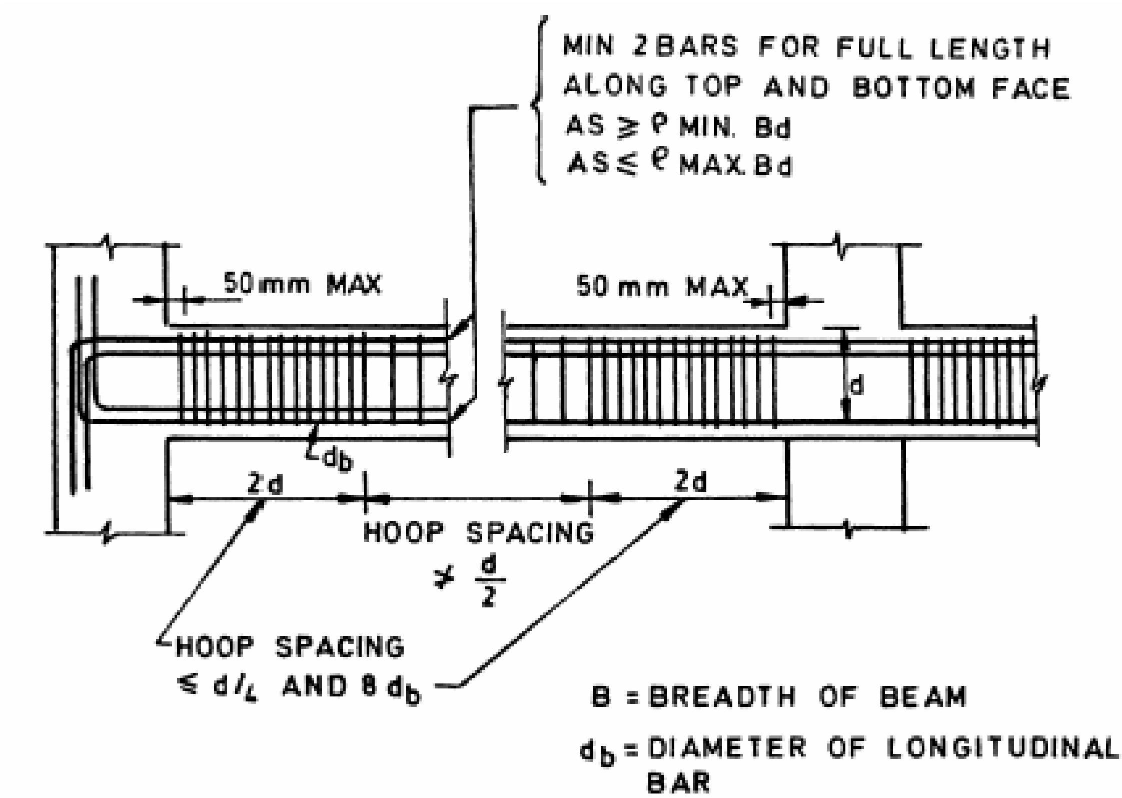


Figure 5 – Beam reinforcement

CODE

7. – Columns and Frame Members Subjected to Bending and Axial Load

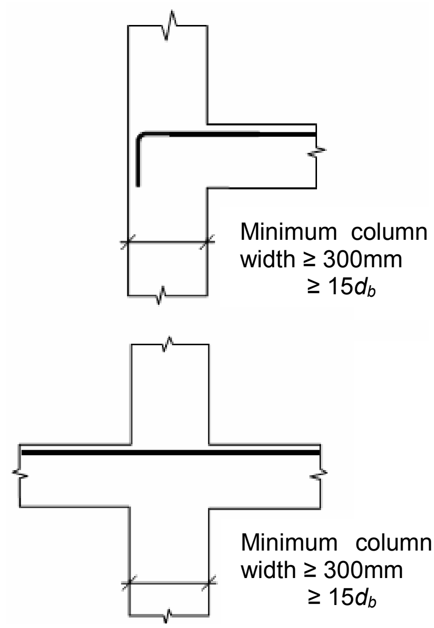
7.1 – General

7.1.1 –

These requirements apply to the frame members which have a factored axial stress in excess of ~~$0.1f_{ck}$~~ $0.08f_{ck}$ under the effect of earthquake forces.

7.1.2 –

The minimum dimension of the member shall not be less than ~~200mm~~ (a) 15 times the largest beam bar diameter of the longitudinal reinforcement in the beam passing through or anchoring into the column joint, and (b) 300 mm (Fig.6). ~~However, in frames which have beams with center the center Span exceeding 5 m or columns of unsupported length exceeding 4m, the shortest dimension of the column shall not be less than 300 mm.~~



d_b = Largest diameter of beam bars

Figure 6 – Longitudinal reinforcement in beam-column joint

COMMENTARY

C7. – Columns and Frame Members Subjected to Bending and Axial Load

C7.1 – General

C7.1.1 –

If the factored axial load is less than the specified limit, the frame member will be considered as a flexural member and will be detailed as per clause 6.

C7.1.2 –

A small column width may lead to following two problems: the moment capacity of column section is very low since the lever arm between the compression steel and tension steel is very small, and beam bars do not get enough anchorage in the column (both at exterior and interior joints).

Hence, many seismic codes recommend that the dimension of an interior column should not be less than 20 times the diameter of largest beam bar running parallel to that column dimension. That is, if beams use a 20 mm diameter bars, minimum column width should be 400 mm. Similarly, codes have a provision for minimum width of exterior columns.

CODE

7.1.3 –

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

COMMENTARY

C7.1.3 –

Confinement of concrete is better in a relatively square column than in a column with large width-to-depth ratio. Clause 7.1.3 is provided to ensure better confinement on concrete.

7.2 – Longitudinal Reinforcement

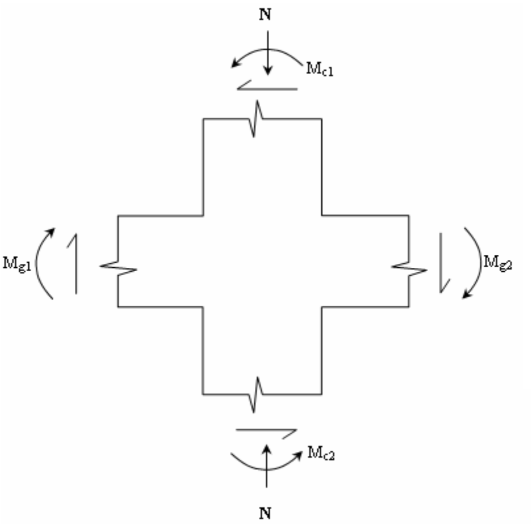
7.2.1 –

At a joint in a frame resisting earthquake forces, the sum of the moment of resistance of the columns shall be at least 1.1 times the sum of the moment of resistance of the beams along each principal plane of the joint (Fig. 7). The moment of resistance of the column shall be calculated considering the factored axial forces on the column. The moment of resistance shall be summed such that the column moments oppose the beam moments. This requirement shall satisfy for beam moments acting in both directions in the principal plane of the joint considered. Columns not satisfying this requirement shall have special confining reinforcement over their full height instead of the critical end regions only.

C7.2 – Longitudinal Reinforcement

C7.2.1 –

This clause is based on strong-column weak-beam theory. It is meant to make the building fail in beam-hinge mechanism (beams yield before columns do) and not in the storey mechanism (columns yield before the beams). Storey mechanism must be avoided as it causes greater damage to the building. Therefore, column should be stronger than the beams meeting at a joint. ACI 318 requires the sum of the moment of resistance of the columns to be at least 20% more than the sum of the moment of resistance of the beams. NZS 3101:1995 recommends that the sum of the design flexural strength of column is at least 40% in excess of the overstrength of adjacent beams meeting at the joint.



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

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

$$\sum M_c \geq 1.1 \sum M_g$$

Figure 7 – Weak beam strong column concept

CODE

7.2.2 –

At least one intermediate bar shall be provided between corner bars along each column face.

~~7.2.1~~7.2.3 –

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. Not more than 50 percent of the bars shall preferably be spliced at one section. If more than 50 percent of the bars are spliced at one section, the lap length shall be 1.3 L_d where L_d is the development length of bar in tension as per IS 456:2000.

COMMENTARY

C7.2.2 –

This clause implies that rectangular columns in lateral load resisting frames shall have a minimum of eight bars. Intermediate bars are required to ensure the integrity of beam-column joint and to increase confinement to the column core.

C7.2.3 –

Seismic moments are maximum in columns just above and just below the beam (Fig. C3). Hence, reinforcement must not be changed at those locations. Since the seismic moments are minimum away from the ends, lap splices are allowed only in the central half of the columns. This also implies that the structural drawings must specify the column reinforcement from a mid-storey-height to next mid-storey-height.

This clause has a very important implication pertaining to dowels to be left for future extension. Inadequate projected length of column reinforcement for future vertical extensions is a very serious seismic threat. This creates a very weak section in all the columns at a single location and all upper storeys are prone to collapse at that level.

When subjected to seismic forces, columns can develop substantial reversible moments. Hence, all the bars are liable to go under tension, and only tension splices are allowed.

The restriction on percentage of lapping bars at one location means that in buildings of normal proportions, only half the bars can be spliced in one storey and the other half in the next storey. In case of construction difficulty in lapping only half the column reinforcement in a storey, the code now allows all bars to be lapped at the same location but with increased lap length of $1.3L_d$, where L_d is the development length in tension as per IS 456:2000.

CODE

COMMENTARY

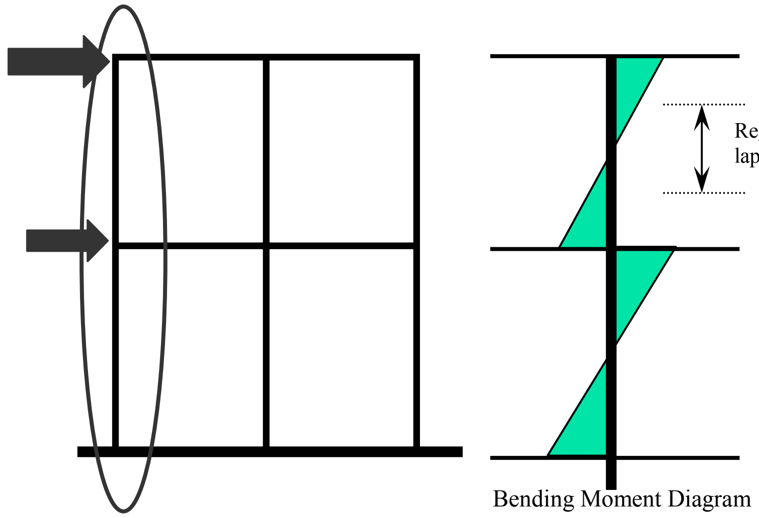


Figure C 3 – Region for lap splices

~~7.2.2~~7.2.4 –

Any area of a column that extends more than 100 mm beyond the confined core due to the architectural requirements, shall be detailed in the following manner. In case of contribution of this area to strength has been considered, then it will have the minimum longitudinal and transverse reinforcement as per this code.

However, if this area has been treated as non-structural, the minimum reinforcement requirements shall be governed by IS 456: 2000~~1978~~ provisions minimum longitudinal and transverse reinforcement, as per IS 456: 2000~~1978~~ (*see* Fig. ~~6~~ 8).

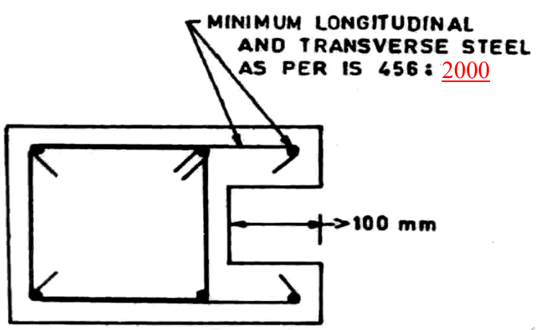


Figure 8 – Reinforcement requirement for column with more than 100 mm projection beyond core

C7.2.4 –

Even when column extensions are considered as non-structural, they contribute to the stiffness of the column. If the extensions are not properly tied with the column core, a severe shaking may cause spalling of this portion leading to a sudden change in the stiffness of the column. Therefore, the code requires that such extensions be detailed at least as per IS 456 requirements for columns.

CODE

7.3 – Transverse Reinforcement

7.3.1 –

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 135^0 hook with a 6~~10~~ diameter extension (but not < 65~~75~~ mm) at each end, that is embedded in the confined core (*see* Fig 9~~7~~A).

7.3.2 –

The parallel legs of rectangular hoop shall be spaced not more than 300 mm centre to centre. If the length of any side of the hoop exceeds 300 mm, a crosstie shall be provided (Fig. 9~~7~~B). Alternatively, a pair of overlapping hoops may be provided within the column (see Fig. 9~~7~~C). The hooks shall engage peripheral longitudinal bars. Consecutive crossties engaging the same longitudinal bars shall have their 90^0 hooks at opposite sides of the flexural member.

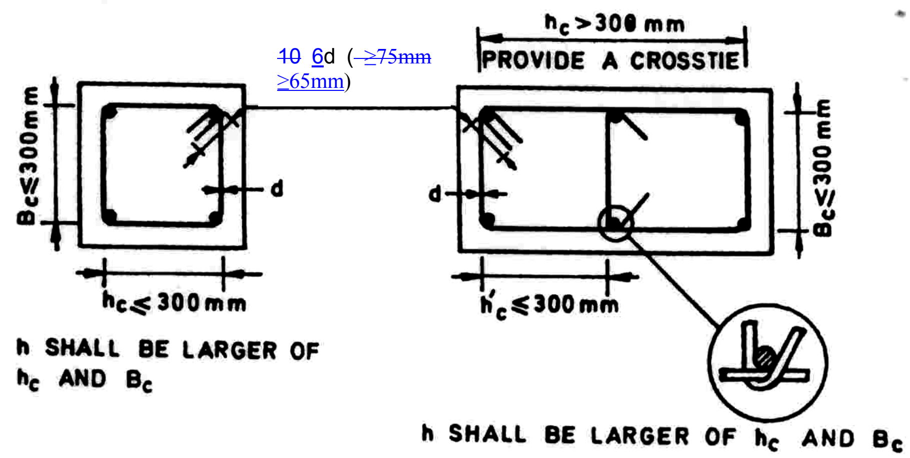
COMMENTARY

C7.3 – Transverse Reinforcement

Transverse reinforcement serves three purposes: (a) it provides shear resistance to the member, (b) it confines the concrete core and thereby increases the ultimate strain of concrete which improves ductility, and (c) it provides lateral resistance against buckling to the compression reinforcement.

C7.3.1 –

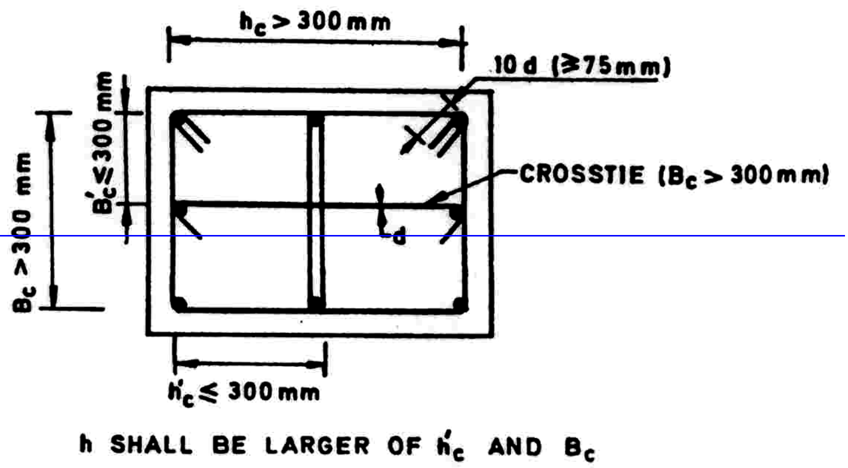
See commentary of clause 6.3.1.

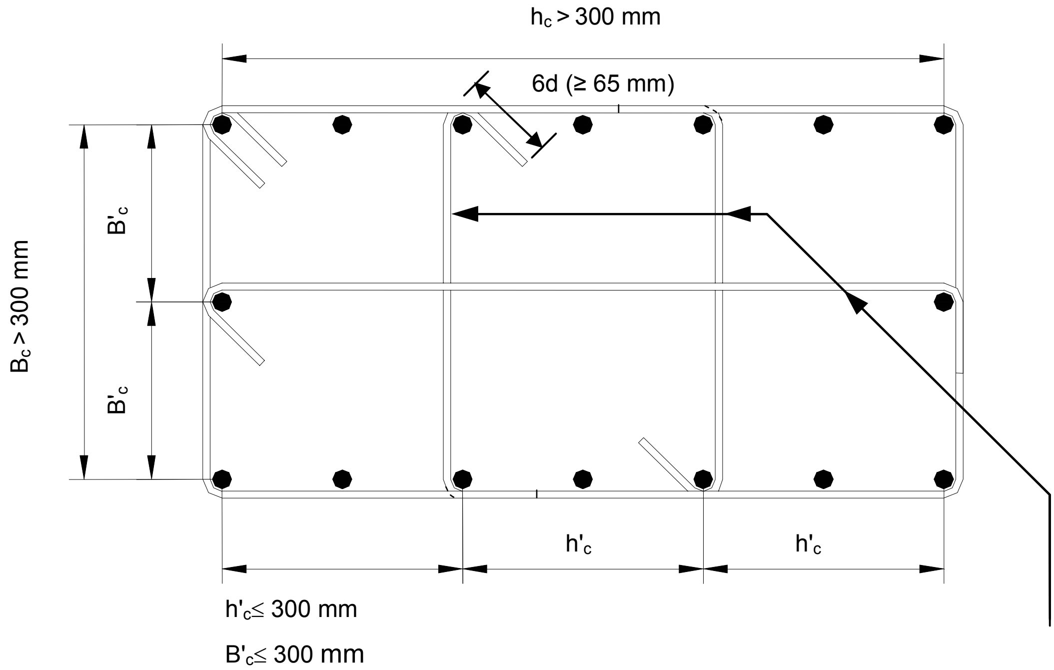


~~7~~9A Single Hoop

~~7~~9B Single hoop with a crosstie

CODE **COMMENTARY**





~~7C~~ 9C Overlapping hoops with a crosstie

Figure 9 – Transverse reinforcement in column

7.3.3 –

The spacing of hoops shall not exceed half the least lateral dimension of the column, except where special confining reinforcement is provided, as per **7.4.**

C7.3.3 –

IS : 456 allows the hoop spacing to be equal to the least lateral dimension of the column while this clause restricts it to half the least lateral dimension. Closer spacing of hoops is desirable to ensure better seismic performance.

7.3.4 –

The design shear force for columns shall he

C7.3.4 –

This clause is based of strong column-weak beam

Consecutive cross ties engaging the same longitudinal bar have their 90° hooks on opposite sides of column.

CODE

the maximum of:

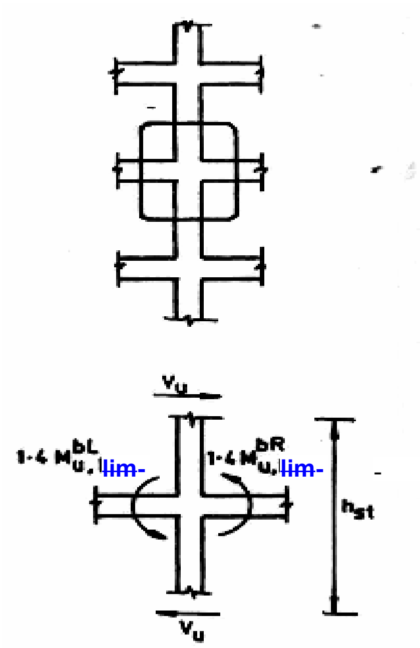
a)calculated factored shear force as per analysis, and

b)a factored shear force given by

$$\cancel{V_u = 1.4\left[\frac{M^{bL}_{u,lim} + M^{bR}_{u,lim}}{h_{st}}\right]}$$

$$V_u = 1.4\left[\frac{M^{bL}_{u} + M^{bR}_{u}}{h_{st}}\right]$$

where $\cancel{M^{bL}_{u,lim}}$ M^{bL}_{u} and $\cancel{M^{bR}_{u,lim}}$ M^{bR}_{u} are moment of resistance, of opposite sign, of beams framing into the column from opposite faces (see Fig. ~~8~~10); and h_{st} is the storey height. The beam moment capacity is to be calculated as per IS 456: ~~1978~~2000.



$$V_u = 1.4\left[\frac{M^{bL}_{u} + M^{bR}_{u}}{h_{st}}\right]$$

Figure 10 – Calculation of design shear force for column

7.3.5 –

The minimum diameter of the transverse reinforcement shall be 8 mm. However, for columns with longitudinal bar diameter larger than 25mm, minimum diameter of transverse reinforcement shall be 10mm.

COMMENTARY

theory. Here, column shear is evaluated based on beam flexural yielding with the expectation that yielding will occur in beams rather than in columns. The factor of 1.4 is based on the consideration that plastic moment capacity of a section is usually calculated by assuming the stress in flexural reinforcement as 1.25 f_y as against 0.87f_y in the moment capacity calculation.

C7.3.5 –

The minimum bar diameter for transverse reinforcement in columns is specified to ensure a minimum ductility and to prevent local buckling of longitudinal bars.

CODE

COMMENTARY

7.4 – Special Confining Reinforcement

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

7.4.1 –

Special confining reinforcement shall be provided over a length l_0 from each joint face, towards midspan, and on either side of any section, where flexural yielding may occur under the effect of earthquake forces (*see* Fig. ~~9~~ 11). The length 'l_0' shall not be less than (a) larger lateral dimension of the member at the section where yielding occurs, (b) 1/6 of clear span of the member, and (c) 450 mm.

C7.4.1 –

These regions may be subjected to large inelastic deformations during strong ground shaking. Hence, special confining reinforcement is provided to ensure adequate ductility and to provide restraint against buckling to the compression reinforcement.

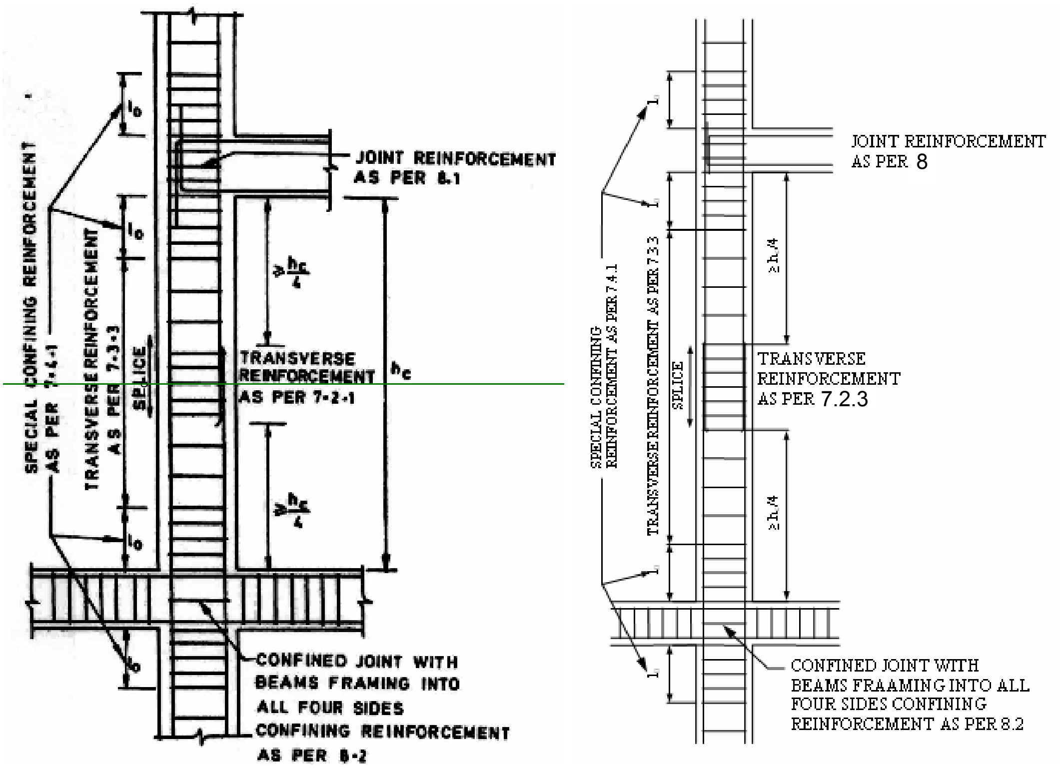


Figure 11 – Columns and joint detailing

7.4.2 –

When a column terminates into a footing or mat, special confining reinforcement shall extend at least 300 mm into the footing or

C7.4.2 –

During severe shaking, a plastic hinge may form at the bottom of a column that terminates into a footing or mat. Hence, special confining reinforcement of the column must be extended to

CODE

mat (*see* Fig. ~~10~~12).

COMMENTARY

at least 300 mm into the foundation.

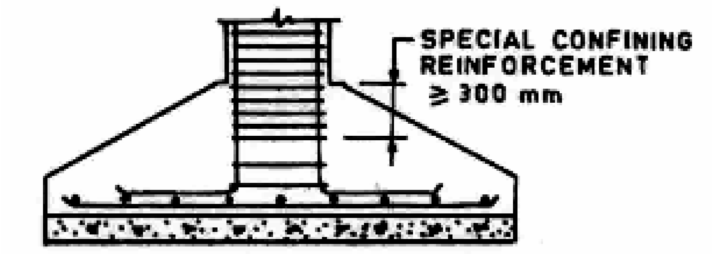


Figure ~~10~~12 – Provision of special confining reinforcement in footing

7.4.3 –

When the calculated point of contra-flexure, 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.

C7.4.3 –

The point of contraflexure is usually in the middle half of the column, except for columns in the top and bottom storeys of a multistory frame. When the point of contraflexure in 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 in such situations.

7.4.4 –

Columns supporting reactions from discontinued stiff members, such as walls, shall be provided with special confining reinforcement over their full height (see Fig. ~~11~~13). 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.

C7.4.4 –

Observations in past earthquakes indicate very poor performance of buildings where a wall in the upper storeys terminates on columns in the lower storeys. Hence, special confining reinforcement must be provided over full height in such columns.

CODE

COMMENTARY

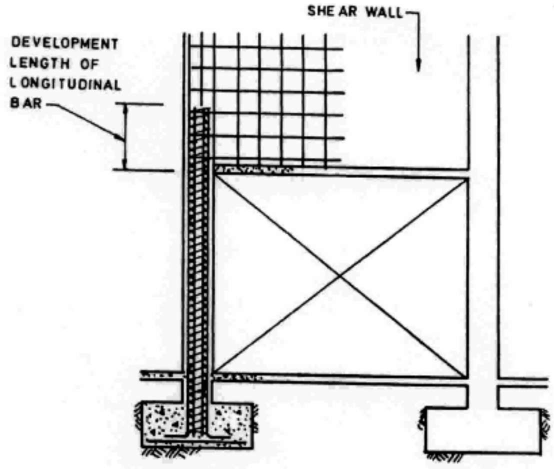


Figure ~~11~~13 – Special confining reinforcement requirements for columns under discontinued walls

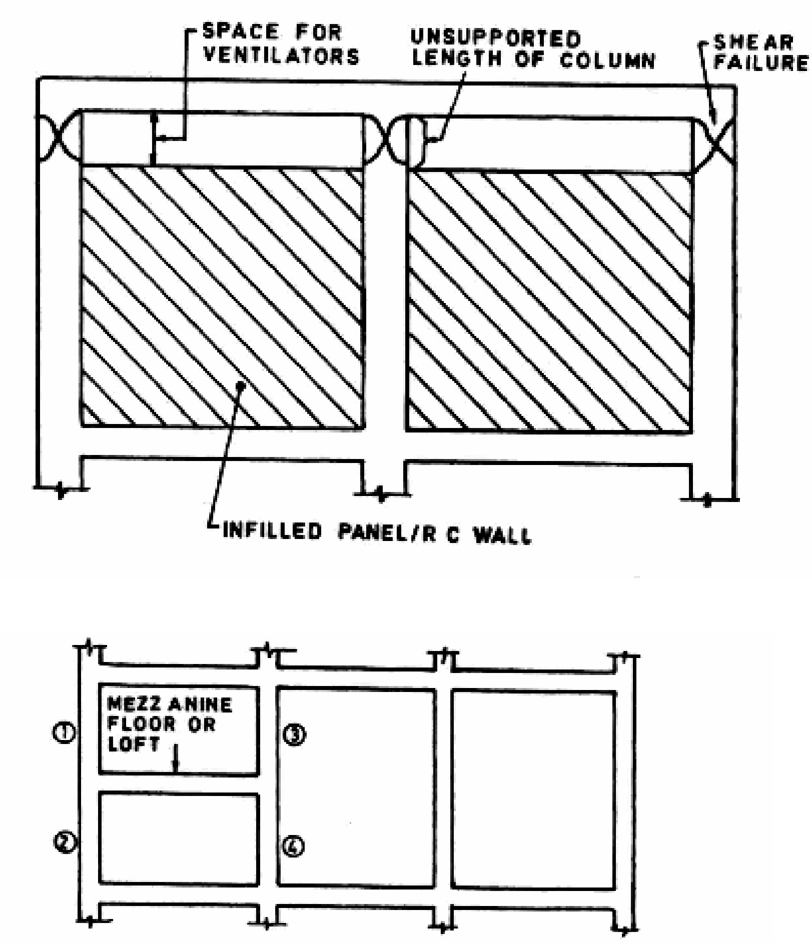
7.4.5 –

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 (*see* Fig. ~~12~~ 14).

C7.4.5 –

Column stiffness is inversely proportional to the cube of column height. Hence, columns with significantly less height than other columns in the same storey have much higher lateral stiffness, and consequently attract much greater seismic shear force. There is a possibility of a brittle shear failure occurring in the unsupported zones of such short 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 storey unbraced over their full height. Another example is of semi-basements where ventilators are provided between the soffit of beams and the top of the wall; here, the outer columns become the "short-columns" as compared to the interior columns. Hence, special confining reinforcement shall be provided over the full height in such columns to give them adequate confinement and shear strength.

CODE — COMMENTARY



(1), 2), (3) and (4) relatively stiff columns — They attract parge seismic shear force.

Figure ~~12~~14 – Columns with variable stiffness

7.4.6 –

The spacing of hoops used as special confining reinforcement shall not exceed 1/4 of minimum member dimension or 6 times diameter of the longitudinal bar, but need not be less than 75 mm nor more than 100 mm.

C7.4.6 –

This requirement is to ensure adequate concrete confinement. Restriction of spacing of 75 mm is to ensure concrete can be compacted properly. In large bridge piers, spacing larger than 100 mm may need to be allowed. For example, the Japanese code permits spacing of up to 150 mm in bridge piers.

7.4.7 –

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

C7.4.7 –

This provision is intended to provide adequate confining reinforcement to the column. The first equation is obtained by equating the maximum axial load carrying capacity of the column prior to be the spalling of the shell, to its axial load carrying capacity at large compressive strain with the spiral reinforcement stressed to its useful

CODE

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

or, $$A_{sh} = 0.024 SD_k \frac{f_{ck}}{f_y}$$

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, and
- 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~~M25, and that of steel 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~~ nominal cover ~~to the longitudinal reinforcement and circular hoops of diameter 8 mm,~~ D_k = 300 - 2 x 40 ~~+ 2 x 8~~ = 2~~36~~20 mm. ~~Thus, the area of cross section of the bar forming circular hoop works out to be 47.28 mm². This is less than the cross sectional area of 8 mm bar (50.27 mm²).~~ The value of A_{sh} as per the first equation above is 76.89 mm² and is 23.85 mm as per the second equation. Thus 10 mm diameter (A_{sh} = 78.5 mm²) will meet the ~~Thus~~requirement. Thus, circular hoops of diameter 10~~8~~ mm at a spacing of 75 mm centre to centre ~~will~~ should be provided~~adequate~~.

7.4.8 –

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

COMMENTARY

limit.

For very large column sections A_g/A_k tends to be close to 1.0, and hence, the first equation in this clause gives a very low value of the confining reinforcement. The second equation governs the large sections, for instance the bridge piers.

C7.4.8 –

The first equation in this clause is intended to provide the same confinement to a rectangular core confined by rectangular hoops as would exist in an equivalent circular column, assuming that rectangular hoops are 50% as efficient as spirals in improving confinement to concrete.

CODE

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

or, $$A_{sh} = 0.05 Sh \frac{f_{ck}}{f_y}$$

where

h = longer dimension of the rectangular confining hoop measured to its outer face. It shall not exceed 300 mm (see Figure ~~7~~9), and

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. ~~7B~~ 9B. 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 650 mm x 500 mm. Let the grade of concrete be M25~~0~~ and that of steel Fe 415, for the longitudinal and confining reinforcement. Assuming ~~clear~~ nominal cover of 40 mm ~~to the longitudinal reinforcement and rectangular hoops of diameter 10 mm,~~ the size of the core is 570~~590~~ mm x ~~440~~ 420 mm. As both these dimensions are greater than 300 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) ~~590~~570/2 = ~~295~~ 285 mm, and (ii) 420~~440~~/2 = 210~~220~~ mm. The spacing of hoops, S, shall not exceed the smaller of (a) 1/4 of minimum member dimensions = 1/4 x 500 = 125 mm, and (b) 100 mm. Thus, S = 100 mm. The value of A_{sh} as per first equation above is 100 mm^2 and is 85.84 mm as per second ~~The~~equation. Thus 12 mm diameter (A_{sh} = 113 mm^2) will be meet the requirement. ~~area of cross section of the bar forming rectangular hoop works out to be 64.47 mm^2. This is less than the area of cross section of 10 mm bar (78.54 mm^2).~~ Thus, rectengular hoops of diameter 12 mm at a spacing of 100 mm centre to centre should be provided.~~Thus, 10 mm diameter rectangular hoops at 100 mm c/c will be adequate. Similar calculations indicate that, as an alternative, one could also provide 8 mm diameter rectangular hoops at 70 mm c/c.~~

COMMENTARY

Second equation governs the large column section.

CODE

8. – Joints of Frames

8.1 – Transverse Reinforcement

8.1.1 –

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 **8.1.3.**

8.1.2 ~~8.2~~ –

~~A joint which has beams framing into all vertical faces of it and where each beam width is at least ¾ of the column width, may be provided with half the special confining reinforcement required at the end of the column.~~ For a joint which is confined by structural members as specified by **8.1.3**, transverse reinforcement equal to at least half the special confining reinforcement required at the end of the column shall be provided within the depth of the shallowest framing member. The spacing of the hoops shall not exceed 150 mm.

8.1.3 –

A member that frames into a face is considered to provide confinement to the joint if at least three-quarters of the face of the joint is covered by the framing member. A joint is considered to be confined if such confining members frame into all faces of the joint.

COMMENTARY

C8. Joints of Frames

C8.1–

C8.1.1 –

Quite often joints are not provided with stirrups because of construction difficulties. Similarly, in traditional constructions the bottom beam bars were often not continuous through the joint. Both these practices are not acceptable when building has to carry lateral loads.

Following are the main concerns with joints:

- Serviceability – Cracks should not occur due to diagonal compression and joint shear.
- Strength – Should be more than that in the adjacent members.
- Ductility – Not needed for gravity loads, but needed for seismic loads.
- Anchorage – Joint should be able to provide proper anchorage to the longitudinal bars of the beams.
- Ease of construction – Joint should not be congested.

C8.1.2 –

Transverse reinforcement can be reduced as per 8.1.2 if structural members frame into all four sides of the joints.

C8.1.3 –

A joint can be confined by the beams/slabs around the joint, longitudinal bars (from beams and columns, passing though the joint), and transverse reinforcement.

CODE

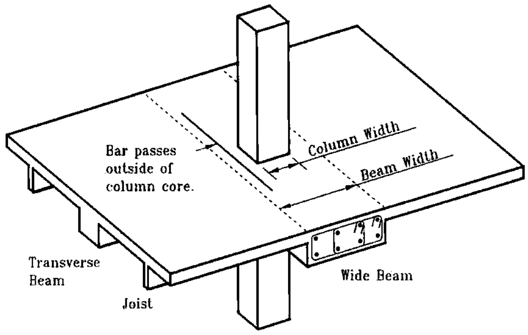
8.1.4 – Wide Beam

If the width of beam exceeds corresponding column dimension, transverse reinforcement as required by 7.4.7 and 7.4.8 shall be provided through the joint to provide confinement for longitudinal beam reinforcement outside the column core if such confinement is not provided by a beam framing into the joint.

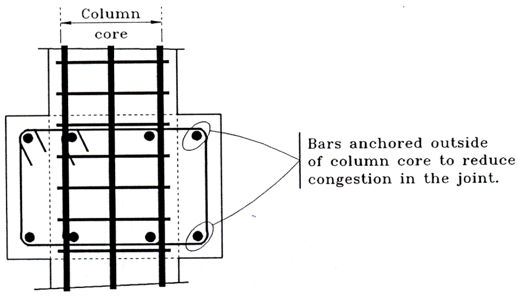
COMMENTARY

C8.1.4 –

This clause refers to the wide beam, i.e, the width of the beam exceeds the corresponding column dimension as shown in Fig. C4. In that case, the beam reinforcement not confined by the column reinforcement should be provided lateral support either by a girder framing into the same joint or by transverse reinforcement.



(a)



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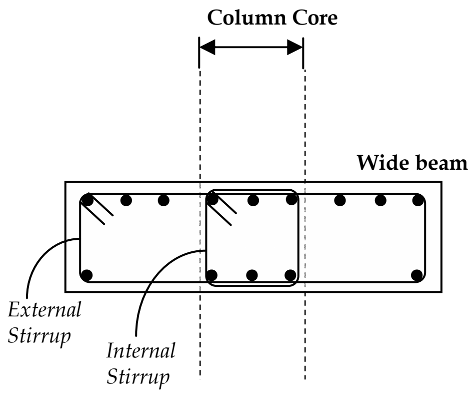


(c)

Figure C4 – Wide beam [from Gentry and Wight, 1992]

8.1.5 –

In the exterior and corner joints, all the 135° hook of the crossties should be along the

C8.1.5 –

135° hook in a crosstie is more effective than a 90° hook to confine core concrete. As the interior

CODE

outer face of the column.

8.2 – Shear Design

8.2.1 – Shear Strength

The nominal shear strength of the joint shall not be taken greater than

$1.5\sqrt{f_{ck}}A_{ej}$ for joints confined on all four faces~~:~~, $1.2\sqrt{f_{ck}}A_{ej}$ for joints confined on three faces or on two opposite faces, and ~~:~~ $1.0\sqrt{f_{ck}}A_{ej}$ for others~~:~~,

where

A_{ej} = Effective shear area of the joint $= b_j h_j$,

b_j = Effective width of joint as per 8.2.2,

h_j = Effective depth of joint as per 8.2.3.

8.2.2 – Effective Width of Joint

The effective width of joint, b_j (Fig. 15) shall be obtained from the following

$$\begin{cases} \mathrm{Min}[b_c; b_b + 0.5D_c] & \text{if } b_c > b_b \\ \mathrm{Min}[b_b; b_c + 0.5D_c] & \text{f } b_c < b_b \end{cases}$$

where,

b_b = width of beam,

b_c = width of column and

D_c = Depth of column in the considered direction of shear.

8.2.3 – Effective Depth of Joint

The effective depth of joint h_j can be taken as depth of the column, h_c as shown in Fig. 15.

COMMENTARY

face of the exterior beam-column joint is confined by beams it is preferable to place the cross ties such that all the 90° hooks are on the inner side and 135° hooks at the exterior side of the joint.

C8.2 – Shear Design

C8.2.1 – Shear Strength

The concept and values of nominal shear strength specified are in line with ACI 318M-02 provisions. The nominal shear strength value specified includes the shear carried by the concrete as well as the joint (shear) reinforcement.

CODE **COMMENTARY**

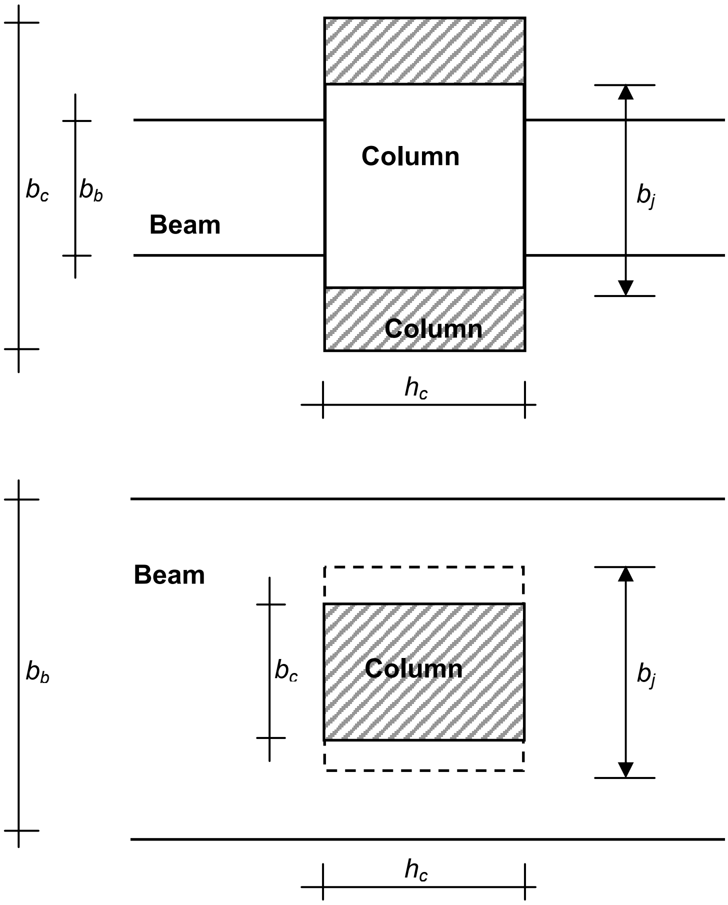


Figure15 – Effective width of joint (plan view)

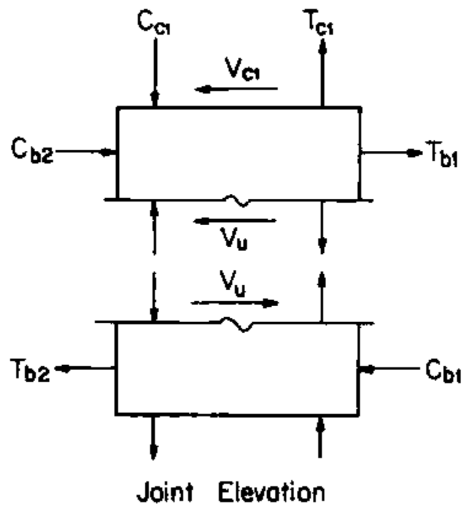
8.2.4 –

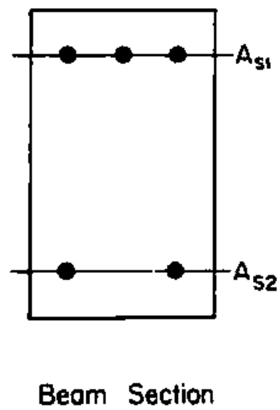
Shear force in the joint shall be calculated assuming that the stress in flexural tensile reinforcement is $1.25f_y$.

C8.2.4 –

Shear force in the joint due to earthquake load can be calculated as shown in Fig. C5.

CODE **COMMENTARY**





Joint shear, $V_u = T_{b1} + C_{b2} - V_{c1}$

where,

$C_{b1} = T_{b1} = 1.25 f_y A_{s1}$

$T_{b2} = C_{b2} = 1.25 f_y A_{s2}$

V_{C1} is column shear as per clause 7.3.4

Figure C5 – Evaluation of horizontal joint shear. T = tension force; C = compression force; V = shear force; subscript b for beam; subscript c for column; and subscript s for steel (adapted from ACI 352-1989).

CODE

9. – Shear Walls

9.1 – General Requirements

9.1.1 –

The requirements of this section apply to the shear walls, which are part of the lateral force resisting system of the structure.

9.1.2 –

The thickness of any part of the wall shall preferably, not be less than 150 mm. In case of coupled shear walls, the thickness of the wall shall be at least 200mm.

9.1.3 –

The effective flange width, to be used in the design of flanged wall sections, shall be assumed to extend beyond the face of the web for a distance which shall be the smaller of (a) half the distance to an adjacent shear wall web, and (b) 1/10th of the total wall height.

COMMENTARY

C9. – Shear Walls

C9.1 – General Requirements

C9.1.1 –

Walls have a very large in-plane stiffness, and therefore, they are very useful in resisting lateral loads and for deflection control if positioned properly. Due to large cross section, compressive stress ratio is usually lower than that in columns.

C 9.1.2 –

The minimum thickness is specified as 150 mm to avoid unusually thin section. Very thin sections are susceptible to lateral instability in zones where inelastic cyclic loading may have to be sustained.

The requirement of minimum thickness in case of coupled shear walls is introduced in view of constructional difficulties in providing diagonal bars with the other steel in the coupling beam (Figure C6).

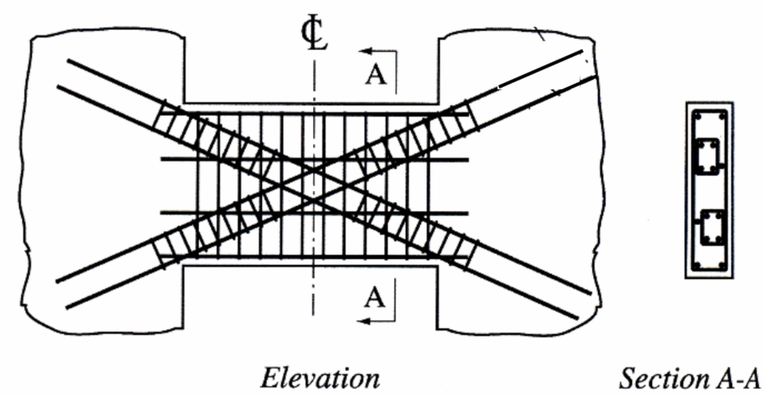


Figure C6: Schematic of reinforcement in coupling beams [from ACI318M, 2002]

C9.1.3 –

The effective width of a flanged wall section is shown in Fig. C7. This flange width criterion is on similar lines as flange width criteria for T beam.

CODE

COMMENTARY

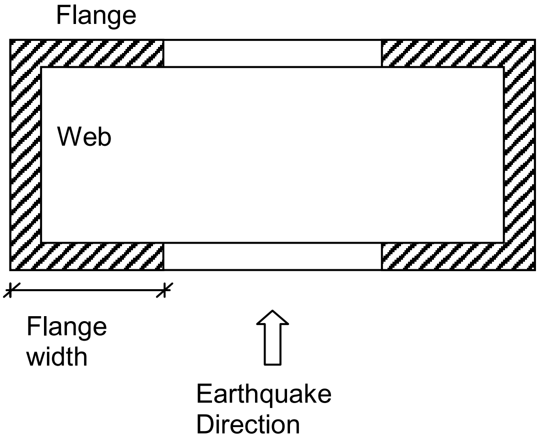


Figure C7 – Effective width of a flanged wall section

9.1.4 –

Shear walls shall be provided with reinforcement in the longitudinal and transverse directions in the plane of the wall. The minimum reinforcement ratio shall be 0.0025 of the gross area in each direction. This reinforcement shall be distributed uniformly across the cross section of the wall.

C 9.1.4 –

Distribution of a minimum reinforcement uniformly across the height and width of the wall helps to control the width of inclined cracks that are caused due to shear.

9.1.5 –

If the factored shear stress in the wall exceeds $0.25\sqrt{f_{ck}}$ or if the wall thickness exceeds 200 mm, reinforcement shall be provided in two curtains, each having bars running in the longitudinal and transverse directions in the plane of the wall.

C9.1.5 –

The use of two curtains of reinforcement will reduce fragmentation and premature deterioration of the concrete under cyclic loading into the inelastic range.

9.1.6 –

The diameter of the bars to be used in any part of the wall shall not exceed 1/10th of the thickness of that part.

C9.1.6 –

This is to prevent the use of very large diameter bars in thin wall sections.

9.1.7 –

The maximum spacing of reinforcement in either direction shall not exceed the smaller of $l_w/5$, $3t_w$, and 450 mm; where l_w is the horizontal length of the wall, and t_w is the thickness of the wall web.

C 9.1.7 –

This clause is similar to spacing requirements of slabs.

CODE

9.2 – Shear Strength

9.2.1 –

The nominal shear stress, τ_v shall be calculated as:

$$\tau_v = \frac{V_u}{t_w d_w}$$

where

V_u = factored shear force,

t_w = thickness of the web, and

d_w = effective depth of wall section. This may ~~by~~ be taken as 0.8 l_w, for rectangular sections.

9.2.2 –

The design shear strength of concrete, τc, shall be calculated as per Table ~~13~~19 of IS 456 : ~~1978~~ 2000.

9.2.3 –

The nominal shear stress in the wall, τ_v, shall not exceed τc, *max*, as per Table ~~14~~20 of IS 456: ~~1978~~2000.

9.2.4 –

When τ_v is less than τ_c shear reinforcement shall be provided in accordance with **9.1.4, 9.1.5** and **9.1.7.**

9.2.5 –

When τ_v is greater than τ_c, the area of horizontal shear reinforcement, A_h, to be provided within a vertical spacing, S_v is given by

$$V_{us} = \frac{0.87 f_y A_h d_w}{S_v}$$

where $V_{us} = (V_u - \tau_c\, t_w\, d_w)$, is the shear force to be resisted by the horizontal reinforcement. However, the amount of horizontal reinforcement provided shall not be less than the minimum, as per **9.1.4.**

COMMENTARY

C9.2 – Shear Strength

C9.2.1 –

Shear strength provisions are very similar to those of beams. The vertical reinforcement that is provided in the wall shall be considered for calculation of the design shear stress as per Table 19 of IS 456: 2000. The increase in shear strength due to axial compression may also be considered as per clause 40.2.2 of IS 456: 2000. However, for this, only 80% of the factored axial compressive force should be considered as effective. This is to consider possible effect of vertical acceleration.

Effective depth of wall is taken as 0.8 times the actual depth of the wall, and flanges are not taken into account for shear capacity.

CODE

COMMENTARY

9.2.6 –

The vertical reinforcement, that is uniformly distributed in the wall shall not be less than the horizontal reinforcement calculated as per **9.2.5.**

C9.2.6 –

This provision is particularly important for squat walls. When the height-to-width ratio is about 1.0, vertical and horizontal reinforcement are equally effective in resisting the shear force. However, for walls with height-to-width ratio less than 1.0, a major part of the shear force is resisted by the vertical reinforcement. Hence, adequate vertical reinforcement should be provided for such walls.

9.3 – Flexural Strength

9.3.1 –

The moment of resistance, M_{uv}, of the wall section may be calculated as for columns subjected to combined bending and axial load as per IS 456: ~~1978~~2000. The moment of resistance of slender rectangular shear wall section with uniformly distributed vertical reinforcement is given in Annex A.

9.3.2 –

Cracked flexural strength of the wall section should be greater than its uncracked flexural strength.

C9.3 – Flexural Strength

C9.3.1 –

The equations in Annex A are derived assuming a rectangular wall section of depth l_w and thickness t_w that is subjected to combined uni-axial bending and axial compression. The vertical reinforcement is represented by an equivalent steel plate along the length of the section. The stress strain curve assumed for concrete is as per IS 456:2000 whereas that for steel is assumed to be bi-linear. Two equations are given for calculating the flexural strength of the section. Their use depends on whether the section fails in flexural tension or in flexural compression. Complete derivation of these equations is available in :

> Medhekar, M.S., and Jain, S.K., "Seismic Behaviour, Design, and Detailing of RC Shear Walls, Part I: Behaviour and Strength," *The Indian Concrete Journal*, Vol. 67, No. 7, July 1993, 311-318.

C9.3.2 –

This provision governs those wall sections which, for architectural or other reasons, are much larger in cross section than required from strength consideration alone. Consider a wall section that is subjected to a gradual increase in moment. Initially, it is uncracked and behaves like a plane concrete section. When the cracking moment is reached, concrete in the extreme fibre ruptures in tension. A further increase in moment causes the reinforcement to take all tension on the section. Thus, the cracked flexural strength of the section should be greater than the uncracked flexural strength to prevent a brittle failure involving sudden fracture of the tension reinforcement.

CODE

9.3.3 –

In walls that do not have boundary elements, vertical reinforcement shall be concentrated at the ends of the wall. Each concentration shall consist of a minimum of 4 bars of 12 mm diameter arranged in at least 2 layers.

9.4 – Boundary Elements

Boundary elements are portions along the wall edges that are strengthened by longitudinal and transverse reinforcement. Though they may have the same thickness as that of the wall web it is advantageous to provide them with greater thickness.

9.4.1 –

Where the extreme fibre compressive stress in the wall due to factored gravity loads plus factored earthquake force exceeds $0.2f_{ck}$, boundary elements shall be provided along the vertical boundaries of walls. The boundary elements may be discontinued where the calculated compressive stress becomes less than $0.15f_{ck}$. The compressive stress shall be calculated using a linearly elastic model and gross section properties.

9.4.2 –

A boundary element shall have adequate axial load carrying capacity, assuming short column action, so as to enable it to carry an axial compression equal to the sum of factored gravity load on it and the additional compressive load induced by the seismic force. The later may be calculated as:

$$\frac{M_u - M_{uv}}{C_w}$$

M_u= factored design moment on the entire wall section,

M_{uv}= moment of resistance provided by distributed vertical reinforcement across the wall section, and

C_w= center to center distance between the boundary elements along the two vertical edges of the wall.

COMMENTARY

C9.3.3 –

Concentrated vertical reinforcement near the edges of the wall is more effective in resisting bending moment.

C9.4 – Boundary Elements

Wall sections having stiff and well confined boundary elements develop substantial flexural strength, are less susceptible to lateral buckling, and have better shear strength and ductility in comparison to plane rectangular walls not having stiff and well-confined boundary elements.

C9.4.1 –

During a severe earthquake, the ends of a wall are subjected to high compressive and tensile stresses. Hence, the concrete needs to be well confined so as to sustain the load reversals without a large degradation in strength.

C9.4.2 –

The boundary element is assumed to be effective in resisting the design moment due to earthquake induced forces along with the web of the wall. The axial compression that is required to be developed in the boundary element for this purpose is given by $(M_u\text{-}M_{uv})/C_w$. Thus, the boundary element should be designed as a short column for an axial load equal to the sum of the above axial compression and the gravity load on it.

The factored gravity load on a boundary element may be taken as the fraction of the gravity load in proportion to its cross-sectional area, unless more accurate calculations are carried out.

CODE

COMMENTARY

9.4.3 –

If the gravity load adds to the strength of the wall, its load factor shall be taken as 0.8.

C9.4.3 –

Moderate axial compression results in higher moment capacity of the wall. Hence, the beneficial effect of axial compression by gravity loads should not be fully relied upon in design due to the possible reduction in its magnitude by vertical acceleration. For example, consider a boundary element in which the unfactored axial compression due to gravity and earthquake loading is 400 kN and 50 kN, respectively. Also assume that the bending moment in the wall due to the seismic force exerts an additional axial force of 800 kN in the boundary element. The boundary element should be designed for the factored compression force of 1.2×[400+50+800] = 1500 kN. Also, the factored design tension force will be (0.8×400 – 1.2×50 – 1.2×800) $= -700\text{kN}$. Note that a load factor of 0.8 has been used for the gravity load as the gravity load adds to the strength of the wall by reducing design tension in the boundary element.

9.4.4 –

The percentage of vertical reinforcement in the boundary elements shall not be less than 0.8 percent, nor greater than 6 percent. In order to avoid congestion, the practical upper limit would be 4 percent.

9.4.5 –

Boundary elements, where required, as per **9.4.1,** shall be provided throughout their height, with special confining reinforcement, as per **7.4**. The first equation as per 7.4.8 need not be satisfied for boundary elements of shear walls.

C9.4.5 –

During a severe earthquake, boundary elements may be subjected to stress reversals. Hence, they have to be confined adequately to sustain the cyclic loading without a large degradation in strength.

The term $\left[\frac{A_g}{A_k}-1\right]$ in the first equation of clause 7.8 tends to increase as column size (A_g) reduces. Therefore, for very small sections, like boundary element of a shear wall, this equation may give unrealistically large amount of confinement reinforcement.

9.4.6 –

Boundary elements need not be provided, if the entire wall section is provided with special confining reinforcement, as per **7.4**.

CODE

9.5 – Coupled Shear Walls

9.5.1 –

Coupled shear walls shall be connected by ductile coupling beams~~. If~~, if the earthquake induced shear stress in the coupling beam exceeds~~.~~

$$\frac{0.1\, l_s \sqrt{f_{ck}}}{D}$$

or when l_s/D is less than or equal to 3, where l_s is the clear span of the coupling beam and D is its overall depth, the entire earthquake induced shear and flexure shall, preferably, be resisted by diagonal reinforcement.

9.5.2 –

The area of reinforcement to be provided along each diagonal in a diagonally reinforced coupling beam shall be:

$$A_{sd} = \frac{V_u}{1.74\, f_y \sin\alpha}$$

where V_u is the factored shear force, and α is the angle made by the diagonal reinforcement with the horizontal. At least 4 bars of 8 mm diameter shall be provided along each diagonal. The reinforcement along each diagonal shall be enclosed by special confining reinforcement, as per **7.4.** The pitch of spiral or spacing of ties shall not exceed 100 mm.

COMMENTARY

C9.5 – Coupled Shear Walls

C9.5.1 –

Coupling beam must have large ductility as they are subjected to extensive inelastic deformations at their ends. In coupling beams of small span-to-depth ratio, diagonal reinforcement is much more effective in controlling shear deformations and in preventing sliding shear failure as compared to the conventional parallel reinforcement.

Experiments have shown that the efficacy of longitudinal reinforcing steel in coupling beams is reduced in walls when l_s/D is more than 3.

C9.5.2 –

The design of a diagonally reinforced coupling beam is based on the assumption that the shear force resolves itself into diagonal compression and tension forces (Fig. C8).

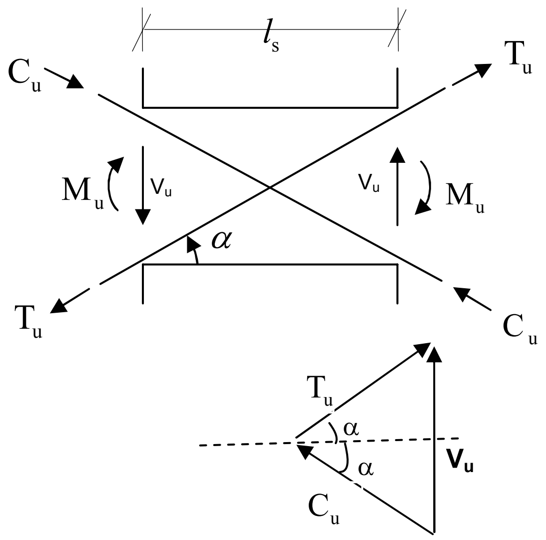


Figure C8 – Diagonally reinforced coupling beam

$$2M_u = V_u\, l_s$$

$$V_u = 2T_u \sin\alpha\,;$$

$$\Rightarrow M_u = \frac{V_u\, l_s}{2} = T_u\, l_s \sin\alpha\,;$$

$$\Rightarrow M_u = (D - 2d')T_u \cos\alpha$$

CODE / COMMENTARY

$$\Rightarrow A_s = \frac{V_u}{2 f_y \sin\alpha}$$

However, the code advises $A_{sd} = \dfrac{V_u}{1.74 f_y \sin\alpha}$, which is conservative.

9.5.3 –

The diagonal or horizontal bars of a coupling beam shall be anchored in the adjacent walls with an anchorage length of 1.5 times the development length in tension.

C9.5.3 –

The increase in development length is to consider the adverse effect of reversed cyclic loading on the anchorage of a group of bars.

9.6 – Opening in Walls

C9.6 – Opening in Walls

9.6.1 –

The shear strength of a wall with openings should be checked along critical planes that pass through openings.

C9.6.1 –

An opening in a shear wall causes high shear stresses in the region of the wall adjacent to it. Hence, it is necessary to check such regions for adequacy of horizontal shear reinforcement in order to prevent a diagonal tension failure due to shear.

9.6.2 –

Reinforcement shall be provided along the edges of openings in walls. The area of the vertical and horizontal bars should be such as to equal that of the respective interrupted bars. The vertical bars should extend for the full storey height. The horizontal bars should be provided with development length in tension beyond the sides of the opening.

9.7 – Discontinuous Walls

Columns supporting discontinuous walls shall be provided with special confining reinforcement, as per **7.4.4.**

C9.7 – Discontinuous Walls

Columns supporting discontinued shear walls may be subjected to significant axial compression and may have to undergo extensive inelastic deformation. Hence, they have to be adequately confined over their full height to ensure good ductility.

9.8 – Construction Joints

The vertical reinforcement ratio across a horizontal construction joint shall not be less

C9.8 – Construction Joints

Sliding tends to occur at construction joints during earthquake. This phenomenon becomes

CODE

than:

$$\frac{0.92}{f_y}\left(\tau_v - \frac{P_u}{A_g}\right)$$

where τ_v is the factored shear stress at the joint, P_u is the factored axial force (positive for compression), and A_g is the gross cross sectional area of the joint.

COMMENTARY

more common in short shear walls with low gravity loads. Therefore, the design shear force at the joint must be less than the shear force that can safely be transferred across the joint, V_j. This is calculated by shear friction concept and is given by

$$V_j = \mu\left(0.8P_u + 0.87 f_y A_v\right)$$

where, μ is the coefficient of friction at the joint (μ=1.0), and A_v is the area of vertical reinforcement available. To account for the possible effects of vertical acceleration, the axial load is taken as $0.8P_u$ instead of P_u itself.

9.9 – Development, Splice and Anchorage Requirement

9.9.1 –

Horizontal reinforcement shall be anchored near the edges of the wall or in the confined core of the boundary elements.

9.9.2 –

~~Splicing~~ Lap splicing of vertical flexural reinforcement should be avoided, as far as possible, in regions where yielding may take place. This zone of flexural yielding may be considered to extend for a distance of l_w above the base of the wall or one sixth of the wall height, whichever is more. However, this distance need not be greater than $2l_w$. Not more than one third of the~~this~~ vertical reinforcement shall be spliced at such a section. Splices in adjacent bars should be staggered by a minimum of 600 mm.

9.9.3 –

Lateral ties shall be provided around lapped spliced bars that are larger than 16 mm in diameter. The diameter of the tie shall not be less than one fourth that of the spliced bar nor less than 6 mm. The spacing of ties shall not exceed 150 mm center to center.

C9.9 – Development, Splice and Anchorage Requirement

C9.9.1 –

Horizontal reinforcement acts as web reinforcement for resisting the shear force. Hence, it should be well anchored.

C9.9.2 –

Splices are to be avoided in critical regions. Sometimes, a large number of vertical flexural reinforcement may have to be extended up several storeys. In such a case, some splicing in potential plastic hinge regions may be unavoidable. Such splices must be staggered so that not more than every third bar is spliced at the same level in a potential hinge region.

C9.9.3 –

Lateral ties help improve performance of lap splices even when concrete cover has spalled off.

CODE

9.9.4 – Welded and Mechanical Splices

Welded splices and mechanical connections shall confirm to ~~25.2.5.2~~ **26.2.5.2** of IS 456: ~~1978~~ 2000. However, not more than half the reinforcement shall be spliced at a section, where flexural yielding may take place.

COMMENTARY

C9.9.4 – Welded and Mechanical Splices

Welded splices and mechanical connectors behave better than lap splices under severe earthquake shaking. Therefore, every alternate bar may be spliced in potential plastic hinge region.

CODE

COMMENTARY

10. – Intermediate Moment Resisting Frame[1]

Requirements of this section are meant for intermediate moment resisting frames. These frames may be adopted in seismic zones II and III only.

C10. – Intermediate Moment Resisting Frame

Implementation of ductile detailing provisions requires substantially higher effort in design, construction and quality control. Therefore, this section gives somewhat lower ductility requirements for moment resisting frames of zone II and III in India which are termed as Intermediate Moment Resisting Frame (IMRF).

IMRF buildings are specified a lower value of R (higher seismic design force) than for special moment resisting frames (SMRF).

10.1 – Flexural Members

10.1.1 –

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 associated with development of moment capacity at both ends of the beam plus the factored gravity load on the span. This is given by:

i) for sway to right:

$$V_{u,a} = V_a^{D+L} - \frac{M_u^{As} + M_u^{Bh}}{L_{AB}} \text{ and}$$

$$V_{u,b} = V_b^{D+L} + \frac{M_u^{As} + M_u^{Bh}}{L_{AB}},$$

ii) for sway to left:

$$V_{u,a} = V_a^{D+L} + \frac{M_u^{Ah} + M_u^{Bs}}{L_{AB}} \text{ and}$$

$$V_{u,b} = V_b^{D+L} - \frac{M_u^{Ah} + M_u^{Bs}}{L_{AB}},$$

where M_u^{As}, M_u^{Ah} and M_u^{Bs}, M_u^{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: 2000. L_{AB} is clear span of

C10.1.1 –

The objective of this clause is to reduce the risk of shear failure in an earthquake. This is similar to clause 6.3.3 except that the multiplier 1.4 is absent in order to reduce conservatism in shear design.

[1] This section is new.

CODE

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 partial safety factor of 1.2 on loads. The design shear at end A shall be the larger of the two values of $V_{u,a}$ computed above. Similarly, the design shear at end B shall be the larger of the two values of $V_{u,b}$ computed above.

10.1.2 –

The positive steel at a joint face must be at least equal to one-third of the negative steel at that face.

10.1.3 –

The steel provided at each of the top and bottom face of the member at any section along its length shall be at least equal to one-fifth of the maximum negative moment steel provided at the face of either joint.

10.1.4 –

The spacing of hoops over a length of *2d* at either end of a beam shall not exceed (a) *d/4*, and (b) 8 times the diameter of the smallest longitudinal bar; however, it need not be less than 100 mm. The first hoop shall be at a distance not exceeding 50 mm from the joint face. Elsewhere, the beam shall have vertical hoops at a spacing not exceeding *d/2.*

10.2 – Column and Frame Members Subjected to Bending and Axial Load

10.2.1 –

The design shear force for columns shall he the maximum of:

a) calculated factored shear force as per analysis, and

b) a factored shear force given by

COMMENTARY

C10.1.2 –

Positive steel requirement at joint face of IMRF is lower than that in SMRF where it is one half of the negative steel (see also 6.2.3).

C10.1.3 –

In case of SMRF, the steel provided at each of the top and bottom face of the member at any section along its length shall be at least equal to one-fourth of the maximum negative moment steel provided at the face of either joint (see 6.2.4). This has been relaxed to one-fifth in case of IMRF.

C10.1.4 –

Requirement of spacing of hoops is same as that for SMRF (see 6.3.5) to have adequate confinement of the core concrete.

C10.2.1 –

This is similar to clause 7.3.4., except that the multiplier 1.4 in the equation is absent to reduce conservatism in column shear design.

CODE

$$V_u = \left[\frac{M_u^{bL} + M_u^{bR}}{h_{st}} \right]$$

where M_u^{bL} and M_u^{bR} are moment of resistance, of opposite sign, of beams framing into the column from opposite faces; and h_{st} is the storey height. The beam moment capacity is to be calculated as per IS 456:2000.

10.2.2 –

Spacing of the hoops shall not exceed (a) half the least lateral dimension of column, (b) 8 times the diameter of smallest longitudinal bar, and (c) 300 mm over a length l_0 from the joint face. The length l_0 shall not be less than (a) larger lateral dimension of the member at the section where yielding occurs, (b) 1/6 of clear span of the member, and (c) 450 mm.

10.2.3 –

Outside the length l_0, spacing of transverse reinforcement shall be as per IS 456:2000.

10.3 – Joints of Frames

Joints shall have the shear reinforcement not less than that required by clause 26.5.1.6 of IS 456 : 2000 within the column for a depth at least equal to that of the deepest framing element to the column.

COMMENTARY

C10.2.2 –

In comparison with SMRF, requirements on special transverse reinforcement in IMRF are considerably lower (see 7.4.6 and 7.4.1).

CODE

11. – Gravity Columns[2]

Gravity columns shall be detailed according to 11.1 and 11.2 depending on the magnitude of moments induced in the column when subjected to twice the lateral displacement under the factored design seismic loads.

COMMENTARY

C11. – Gravity Columns

IS 1893: 2002 allows dual systems (Fig. C9) wherein shear walls can be designed for total seismic loads, and frame can be designed only for gravity loads. Flat plate (Fig. C10a) and flat slab systems (Fig. C10b) with shear walls are good examples of dual systems. However, it was seen in 1994 Northridge earthquake in California that gravity frame could collapse during shaking, and hence, specific provisions are needed for gravity columns also.

The detailing requirements of the columns that are part of the lateral-force resisting system assume that the members may undergo deformations that exceed the yield limit of the member without significant loss of strength. But, the members that are not part of the lateral-force resisting system are not required to meet such detailing requirements. However, they should be able to carry the gravity loads even when undergoing earthquake induced displacement.

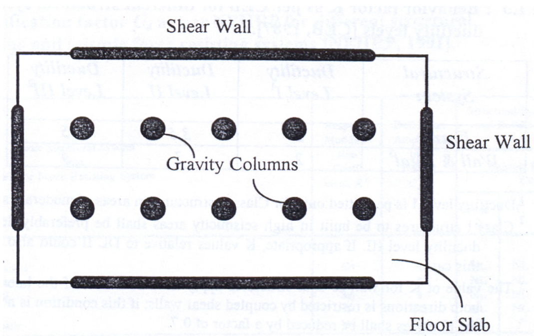


Figure C9 – Plan of building with shear walls and gravity columns (from Agarwal, 1996)

[2] This section is new.

CODE

COMMENTARY

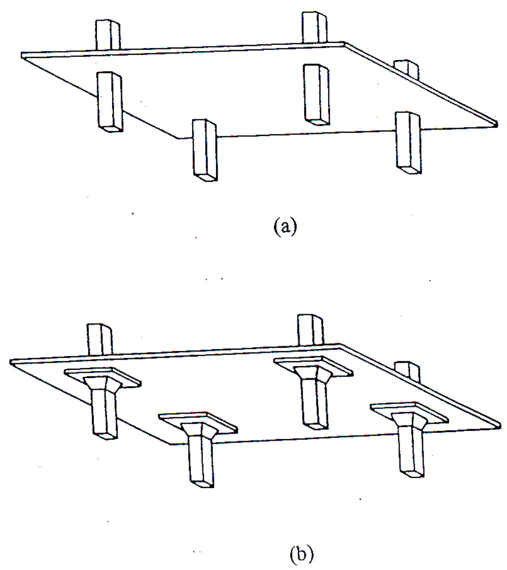


Figure C10 – Some structural systems in which gravity columns appear (a) Flat plate system, (b) Flat slab system (from Agarwal, 1996)

11.1 –

When the induced moments and shears under lateral displacement combined with factored gravity moment and shear do not exceed the design moment and shear strength of the column, clauses 11.1.1 and 11.1.2 shall be satisfied.

C11.1 –

This clause prescribes detailing requirements intended to provide a gravity column capable of taking gravity loads under moderate displacements. Clause 11.1.1 is for column with smaller gravity loads. In this case only nominal detailing is prescribed. These requirements include (i) 135° hook in ties (clause 7.3.1), and (ii) closely spaced ties near the floors (clause 7.3.2) and along entire length of the spliced region (clause 7.2.2). Special confining reinforcement should be provided for gravity column under discontinued stiff member (clause 7.4.4). Clause 11.1.2 is for gravity column with higher gravity load. Apart from nominal detailing that is prescribed for 11.1.1 this type of gravity columns require at least half the area of steel required for special confining reinforcement in column that resist lateral load (7.4.7 and 7.4.8).

11.1.1 –

Members shall satisfy 7.2.3, 7.3.1 and 7.3.2. Spacing of the hoops shall not exceed 6 diameter of the smallest longitudinal bar and 150mm, whichever is smaller, for the full column height.

C11.1.1 –

This is in line with requirements of ACI 318-02.

CODE

COMMENTARY

11.1.2 –

Member with factored gravity axial stress exceeding 0.35 f_{ck} shall satisfy 11.1.1 and amount of transverse reinforcement shall be at least one half of that required by 7.4.7 and 7.4.8.

11.2 –

When the induced moments and shear under lateral displacement combined with factored gravity moment and shear exceed the design moment and shear strength of the frame, or if induced moments are not calculated, clauses 11.2.1 and 11.2.2 shall be satisfied.

C11.2 –

This clause prescribes detailing requirements intended to provide a system capable of taking gravity loads under large displacements. This type of gravity column is required to be detailed so as to fulfill most of the requirements (i.e, transverse reinforcement in the column, special confining reinforcement, transverse reinforcement for joints) imposed on columns meant for taking seismic loads.

11.2.1 –

Materials should satisfy 5.2 and 5.3. Welded and mechanical splices should satisfy 6.2.8.

11.2.2 –

Members shall satisfy 7.3, 7.4 and 8.1.

CODE

COMMENTARY

Annex A

(Clause 9.3.1)

Moment of Resistance of rectangular shear wall section

A-1 The moment of resistance of a slender rectangular shear wall section with uniformly distributed vertical reinforcement may be estimated as follows:

(a) For $x_u/l_w < x_u^*/l_w$

$$\frac{M_{uv}}{f_{ck} l_w l_w^2} = \varphi\left[\left(1+\frac{\lambda}{\varphi}\right)\left(\frac{1}{2}-0.416\frac{x_u}{l_w}\right)-\left(\frac{x_u}{l_w}\right)^2\left(0.168+\frac{\beta^2}{3}\right)\right]$$

where

$$\frac{x_u}{l_w} = \left(\frac{\varphi+\lambda}{2\varphi+0.36}\right);$$

$$\frac{x_u^*}{l_w} = \left(\frac{0.0035}{0.0035+0.87\,f_y/E_s}\right);$$

$$\varphi = \left(\frac{0.87 f_y \rho}{f_{ck}}\right);$$

$$\lambda = \left(\frac{P_u}{f_{ck}\,t_w\,l_w}\right);$$

ρ = vertical reinforcement ration = $A_{st}/(t_w l_w)$

A_{st} = area of uniformly distributed vertical reinforcement,

β = 0.87 f_y/(0.0035 E_s)

E_s = elastic modulus of steel, and

P_u = axial compression on wall

(b) For $x_u^*/l_w < x_u/l_w < 1.0$

$$\frac{M_{uv}}{f_{ck}\,t_w\,l^2 w} = \alpha_1\left(\frac{x_u}{l_w}\right)-\alpha_2\left(\frac{x_u}{l_w}\right)^2-\alpha_3-\frac{\lambda}{2}$$

where

$$\alpha_1 = \left[0.36+\varphi\left(1-\frac{\beta}{2}-\frac{1}{2\beta}\right)\right]$$

$$\cancel{\alpha_2 = \left[0.15+\frac{\phi}{2}\left(1-\beta-\frac{\beta^2}{2}-\frac{1}{3\beta}\right)\right]}$$

CODE

COMMENTARY

$$\alpha_2 = \left[0.15 + \frac{\varphi}{2}\left(1 - \beta + \frac{\beta^2}{3} - \frac{1}{3\beta}\right)\right]; \text{ and}$$

$$\alpha_3 = \frac{\varphi}{6\beta}\left(\frac{1}{(x_u/l_w)} - 3\right)$$

the value of x_u/l_w to be used in this equation, should be calculated from the quadratic equation

$$\alpha_1 = \left(\frac{x_u}{l_w}\right)^2 + \alpha_4\left(\frac{x_u}{l_w}\right) - \alpha_5 = 0;$$

where

$$\alpha_4 = \left(\frac{\varphi}{\beta} - \lambda\right); \text{ and } \alpha_5 = \left(\frac{\varphi}{2\beta}\right).$$

These equations were derived, assuming a rectangular wall section of depth l_w and thickness t_w that is subjected to combined uni-axial bending and axial compression. The vertical reinforcement is represented by an equivalent steel plate along the length of the section. The stress-strain curve assumed for concrete is as per IS 456: ~~1978~~ 2000 whereas that for steel is assumed to be bi-linear. Two equations are given for calculating the flexural strength of the section. Their use depends on whether the section fails in flexural tension or in flexural compression.

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Acknowledgement

Authors gratefully acknowledge Dr. Ashok K Jain of IIT Roorkee and Dr. S K Deb of IIT Guwahati who reviewed an earlier version and provided many thoughtful suggestions. Review comments by GSDMA Review Committee, in particular those by Ms. Alpa Sheth, Seismic Advisor, GSDMA are gratefully acknowledged.