Document No. IITGN-WB-EQ4C-V1.0
Proposed Changes in IS 13920 Code
IITGN - World Bank Project on Seismic Codes

Proposed Changes in Ductile Design and Detailing of Reinforced Concrete Structures Subjected to Seismic Forces – Code of Practice (IS 13920 : 2016) Proposed Modifications and Commentary

by

Svetlana Brzev[1]
Sudhir K. Jain[1]
Durgesh C Rai[2]

with assistance from

Parthesh Sunilbhai Oza[1]
Parul Srivastava[2]

1) *Indian Institute of Technology Gandhinagar*
2) *Indian Institute of Technology Kanpur*

November 2019

- Revisions in this document have been shown with strikethrough font when the content has been removed, or underlined when new content has been added.
- This document has been developed under the World Bank-sponsored Project on Improving Seismic Resilience of Built Environment in India at the Indian Institute of Technology Gandhinagar.
- This report presents the proposed modifications for the code on Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces (IS 13920: 2016). Proposed revisions in the code have been shown with strikethrough font when the content has been removed, or underlined when new content has been added.
- The views and opinions expressed are those of the authors and not necessarily of the World Bank, IIT Gandhinagar, IIT Kanpur, or the Bureau of Indian Standards.
- Comments and feedback may please be forwarded to:

Prof. Sudhir K. Jain, IIT Gandhinagar, Palaj, Gujarat 382355

email: *skjain@iitgn.ac.in*, *skjain.iitk@gmail.com*

Acknowledgements

- We gratefully acknowledge the World Bank for sponsoring this project on Improving Seismic Resilience of Built Environment in India. Special thanks are due to Keiko Sakoda, Thomas Moullier, Anup Karanth and Deepak Singh at the World Bank.
- The authors are also grateful to the World Bank's reviewer, Prof. Andreas J. Kappos, United Kingdom for providing comprehensive set of recommendations that contributed to enhancing the quality of this document.
- The authors acknowledge input and feedback received from internal reviewers Nilesh Shah and Hemal Mistry and other Indian colleagues, including Anal Shah, Hiten Shah, Vipul Ahuja, A K Jain, etc.
- We sincerely appreciate the participation of following colleagues in the roundtable workshop held at IIT Gandhinagar on April 8, 2019 to review and discuss seismic design codes, IS 1893 (Part 1) and IS 13920:

Ahuja, Vipul Consulting Engineer, New Delhi
Bhowmick, Alok Consulting Engr., New Delhi
Brzev, Svetlana IIT Gandhinagar
Goswami, Rupen IIT Madras
Ingle, R. K. VNIT Nagpur
Jain, Sudhir K. IIT Gandhinagar
Jaiswal, O. R. VNIT Nagpur
Karanth, Anup The World Bank, Delhi
Kaushik, Hemant B. IIT Guwahati
Khandelwal, Praveen NTPC Delhi
Kochak, Narayan Consulting Engineer, Pune
Kumar, Arun S Bureau of Indian Standards, Delhi
Kumar, Hemant Consulting Engineer, Delhi
Kumar, Manish IIT Gandhinagar
Murty, C. V. R. IIT Madras
Mistry, Hemal Consulting Engineer, Surat
Pathak, Jayanta Assam Engineering College, Guwahati
Perez-Gavilan, Juan Jose Universidad Nacional Autónoma de México
Rai, Durgesh C. IIT Kanpur
Shah, Anal Consulting Engineer, Ahmedabad
Shah, Bhavin Consulting Engineer, Ahmedabad
Shah, Nilesh Consulting Engineer, Surat
Sharma, Rajeev Consulting Engineer, Delhi
Sheth, Alpa Consulting Engineer, Mumbai
Singh, Deepak The World Bank, Delhi
Singh, Yogendra IIT Roorkee
Singhal, Vaibhav IIT Patna
Tandon, Mahesh Consulting Engineer, Delhi

- A seminar-cum-workshop was organized at IIT Gandhinagar, wherein more than 180 academicians, practising engineers and students participated from across the country to publicly discuss the proposed modifications in seismic codes IS 1893 (Part 1) and IS 13920, in addition to discussing the codal compliance in seismic design of a few real-life buildings. We sincerely appreciate their time and effort which proved extremely helpful in revising the report.

Foreword

Challenges associated with the implementation of IS 13920 : 2016 standard prompted a need for numerous proposed revisions which are contained in this document.

a) The following key changes have been made:

1) It is proposed to revise the hook extension length in transverse beam/column reinforcement to 6 times diameter (but not less than 65 mm) **(3.5)**. It is currently prescribed to use 8 times diameter but not < 75 mm. The proposed length is considered to be safe and more feasible for field applications.
2) It is proposed to increase the minimum grade of concrete to M25 - previously M20 grade was permitted for some applications **(5.2)**. Also, maximum concrete grade (M70) has been prescribed, but higher grades are permitted under certain conditions.
3) It is proposed to revise provision pertaining to the actual 0.2 percent proof strength of steel **(5.3.1)**: this strength should not exceed the characteristic 0.2 percent proof strength by more than 125 MPa (instead of the current 20% limit).
4) It is proposed to increase the ratio of the ultimate strength to 0.2 percent proof strength for steel from 1.15 to 1.25 **(5.3.3)**. This is in line with properties of commercially available steel in India and requirements of international codes.
5) It is proposed to revise one of the requirements regarding the lap splices in beams **(6.2.6.1(c)3)** because the current requirement is not clear and is difficult to implement in practice.
6) Signs in the equations for determining required shear capacity for beam sway to left are incorrect and had to be revised **(6.3.3)**.
7) Ignoring contribution of concrete towards shear strength of beams is overly conservative for certain cases **(6.3.4)**. Hence, contribution of concrete towards the shear strength of beams in plastic hinge conditions has been considered under certain conditions.
8) It is proposed to revise the requirements pertaining to maximum permitted spacing of links in beams **(6.3.5)**. The spacing requirement b) has been relaxed - from 6 to 8 times the diameter of the smallest longitudinal bar. Requirement c) has also been relaxed – from 100 to 250 mm.
9) The provision regarding the shortest cross-sectional dimension of a column have been revised **(7.1.1)**. According to the requirement a) the minimum dimension shall not be less than $20d_b$ for rectangular columns and $30d_b$ for circular columns, where d_b is the largest longitudinal reinforcing bar diameter. Requirement b) 300 mm has remained unchanged.
10) It is proposed to change the multiplier in summation of bending moments at beam-column joints from 1.4 to 1.2 **(7.2.1)**. The value of 1.4 was first introduced in the 2016 version of IS 13920, and it has caused significant implementation challenges. The implications of this clause are significant increase in the size of columns and amount of longitudinal reinforcement. Since the purpose of this clause is to ensure that beams yield before the columns, it is believed that a multiplier of 1.2 provides a reasonable margin in flexural overstrength.
11) Strong column-weak beam can be relaxed for certain reinforced concrete moment resisting frames **(7.2.1.3)**.The 2016 edition requires strong column-weak beam design for special moment resisting frames (SMRFs) which may be hard to implement for buildings in moderate seismic regions and may not be justified **(1.1.1)**. Hence, Intermediate moment resisting frames have been introduced for application in low-to-moderate seismic regions **(9)**.
12) The requirement for lap splices in columns has been modified to have “lap splices shall be provided only in the centre of clear column height” **(7.3.2.1)**.
13) The cross-tie hooks in columns which engage both links (hoops) and peripheral longitudinal bars project into the concrete cover and may corrode easily **(7.4.2)**. Cross-ties and links need to engage only the peripheral longitudinal bars - Figure 10 has been corrected accordingly.
14) Figure 15 shows various joint dimensions for calculation of joint shear but it is not sufficiently clear (**8.1.1 – previously 9.1.1**). An isometric figure of the beam-column joint is now included which provides better illustration of beam and column dimensions required for joint shear calculation. Also, the joint shear strength equations have been corrected by deleting area term.

15) It is proposed to avoid welded splices in beams (**6.2.6.3**) and columns (**7.3.2.3**) of Special moment resisting frames, as well as Special structural walls (**10.8.3.3**). The provision which permitted welded splices was first introduced in the 2016 revision of IS 13920. International codes (e.g. ACI 318-14) permit the use of welded splices in RC structural members, but the provisions are more detailed than the current IS 13920, and specify minimum size of reinforcement for welded splices (19 mm diameter) and steel tensile strength, among others. Unless more specific provisions are included in IS 13920 it is recommended to avoid welded splices due to possibility of a brittle failure at the splice location.

16) Provisions related to Special shear (structural) walls (**10**) have been modified and expanded. These modifications were required to make the provisions more comprehensive and in line with international state-of-the-art seismic design and construction practice. Major revisions are summarized below:
    a. The term "shear wall" has been replaced by "structural wall" in line with the international codes and IS 1893 (Part 1).
    b. The minimum wall thickness requirement has been revised to include slenderness ratio limit, which is relevant for preventing instability (buckling) within the wall's compression zone (**10.1.2**).
    c. Dimensional limits for walls have been revised: minimum length-to-thickness ratio of 6.0 has been proposed (earlier the value was equal to 4.0); this is in line with international codes (**10.1.3**).
    d. Classification of walls has been simplified to include only squat walls and flexural walls (intermediate walls were removed). This is in line with most international codes (**10.1.4**).
    e. Minimum distributed reinforcement requirements were revised to align with international codes and simplify current requirements (**10.1.6**).
    f. A new clause defining plastic hinge region has been introduced (**10.1.11**). Previously there was no specific clause but the length of plastic hinge region was defined by 10.8.2.
    g. Design for shear has been revised to consider an increase in design shear force due to flexural overstrength and the effect of higher vibration modes in tall buildings (**10.2**).
    h. A ductility check has been introduced to prove that a Special structural wall has an ability to deform in an inelastic manner within the plastic hinge zone (**10.3.3**).
    i. Provisions for boundary elements in Special structural walls have been revised and expanded to facilitate design of these elements, by recommending the minimum dimensional requirements and design procedure (**10.4**). It is proposed to provide boundary elements in all Special structural walls.
    j. Provisions for coupled structural walls have been significantly expanded and clarified (**10.5**).
    k. Additional requirements related to design of squat structural walls have been introduced (**10.9**). It has been recognized that seismic response of squat walls is different from flexural walls, hence special provisions are required to address their design and detailing.

17) Provisions related to gravity columns have been significantly clarified and expanded (**12 – previously 11**). The clause title has been revised from "Gravity Columns" to "Gravity Load-Resisting Frames, Walls, Joints, and Flat Slabs" in recognition of the fact that several different structural elements which are not a part of the lateral force resisting system are subjected to increased displacement demands. Provisions for detailing of gravity load-resisting structural elements at different seismic demand levels have been proposed.

b) The following new provisions have been added:

1) Intermediate Moment Resisting Frames (IMRFs) have been introduced as an alternative to Special Moment Resisting Frames (SMRFs) which require significant effort in design, construction and quality control (**9**). Application of IMRFs is limited to zones III and IV (**1.1.1**). The detailing provisions are less stringent than for SMRFs but the R factor is lower (according to proposed change in IS 1893).

2) Intermediate structural walls (ISWs) have been introduced as an alternative to special structural

walls which require significant effort in design, construction and quality control (11). Application of ISWs is limited to zones III and IV (1.1.1). The detailing provisions are less stringent than for special structural walls but the R factor is lower (according to a proposed change in IS 1893).

3) Two-way slabs without beams (flat slabs/plates) have been permitted as a lateral force resisting system in zone III. A set of design and detailing provisions for these slabs has been introduced (13).

4) Seismic design provisions for the foundations of RC buildings designed according to IS 13920 have been introduced. Foundations are critical elements of a seismic load path in buildings and need to be addressed by IS 13920 (14).

c) The following provisions require substantial deliberations and are proposed for consideration in future code revisions:

1) The code has been drafted keeping in view the requirements of buildings, and may not be suitable for certain other type of structures. Hence, in title of the code "structure" may be replaced by "buildings."
2) Clauses in the code may need to be sequenced better, so as to provide requirements on "demand" in one place, and followed by provisions for the "capacity".
3) Provisions related to the design of precast and prestressed RC structures in seismic zones **(1.1.2)** may need to be added.
4) The current provisions for the anchorage of longitudinal beam bars at the external beam-column joint require significant modifications so as to align these with international practices **(6.2.5)**.
5) Provisions for shear strength of concrete in a beam-column joint may need to consider both diagonal compression and diagonal tension failure **(8.1.1)**.
6) A ductility check has been proposed for RC structural walls **(10.3.3)**. However similar checks need to be considered for other ductile RC structural elements, such as beams and columns covered by 6 and 7, respectively.
7) Soil-structure interaction needs to be considered in seismic design of foundations, specifically with regards to foundation movements **(14.2)**.

~~w~~While the common methods of design and construction have been covered in this standard for RC structural systems with moment resisting frames and RC ~~structural systems with~~ structural walls that participate in resisting earthquake force, design and construction of other lateral ~~load~~force resisting structural systems made of reinforced concrete but not covered by this standard, may be permitted by the approving agency or a Committee constituted by the agency only on production of satisfactory evidence from experiments on prototype sub-assemblages and structures, and non-linear analyses demonstrating their adequacy to resist earthquake shaking expected in the region where the structures are expected to be built. Such non-linear analyses shall demonstrate that the collapse mechanism of the proposed structure is desirable and that the lateral deformation capacity of the structure is sufficient to resist the ground deformation imposed in the region where the structure is located. The Committee of the approving agency shall comprise of competent engineers with the necessary experience and shall have the authority to review the data submitted, ask for additional data, tests and to frame special rules for such structural systems not covered under this Code.

The Committee responsible for the formulation of this standard has taken into consideration the views of manufacturers, users, engineers, architects, builders and technologists, and has related the standard to the practices followed in the country in this field. Also, due weightage has been given to the need for international coordination among standards prevailing in different seismic regions of the world.

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

ACI 318-14 and ACI 318-19, 'Building code requirements for structural concrete and commentary', issued by the American Concrete Institute.

~~ACI 318-11, 'Building code requirements for structural concrete and commentary', issued by the American Concrete Institute.~~

CSA A23.3-14, 'Design of concrete structures and explanatory notes', issued by the Canadian Standards Association.

IBC 2012 'International Building Code', published by International Code Council, Inc.

EN 1998-1:2005 (E) Eurocode 8: 'Design of structures for earthquake resistance — Part 1: General rules,

seismic actions and rules for buildings', issued by European Committee for Standardization, Brussels, edition 2005-06-01.

prEN 1998-1-2:2019.2 (E) Eurocode 8: 'Design of structures for earthquake resistance — Part 1-2: Rules for new buildings', official 2nd draft issued by European Committee for Standardization, Brussels.

NZS 3101(Part 1): 2006 'Concrete structures standard (including Amendments 1, 2, and 3)', issued by Standards ~~Council,~~ New Zealand.

The composition of the Committee responsible for the formulation of this standard is given at Annex B of IS 13920 : 2016.

For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS 2 : 1960 'Rules for rounding off numerical values (revised)'. The number of significant places retained in the rounded off value should be same as that of the specified value in this standard.

1.1 –

This standard covers the requirements for designing and detailing of members of reinforced concrete (RC) structures, mostly buildings, designed to resist lateral effects of earthquake shaking so as to give them adequate toughness and ductility to resist severe earthquake shocks without collapse. ~~Even though the general concepts adopted in this standard for structures are also applicable for RC bridge systems, provisions of this standard shall be taken only as a guide for RC bridge piers and wells of large cross-sections, but are not sufficient.~~ This standard addresses lateral ~~load~~force resisting structural systems of RC ~~structures~~ buildings composed of,

a) RC moment resisting frames,
b) RC moment resisting frames with unreinforced masonry infill walls,
c) RC moment resisting frames with RC structural walls, and
d) RC structural walls.

1.1.1 –

Provisions of this standard shall be adopted in all lateral ~~load~~force resisting systems of RC structures located in ~~S~~seismic ~~Z~~zones III, IV or V. The standard is optional in Seismic Zone II. Sections 9 and 11 are options for RC structures in seismic zones III and IV. However, sections 9 and 11 are not applicable to RC structures located in seismic zone V.

1.1.2 –

The provisions for RC structures given herein apply specifically to ~~monolithic~~ RC construction and the lateral force ~~load~~-resisting systems which are identified in IS1893 (Part 1): 2016 (Table 9). ~~Note that m~~ Monolithic RC flat slab structures ~~must~~ are recognized as a primary lateral force resisting system in seismic zones II and III according to Cl.7.2.6 of revised IS 1893 (Part 1): 2016. ~~capable of providing similar level of performance as envisioned in this standard.~~ Their structural components must be designed and detailed as per 13 and checked for drift compatibility as per 12. ~~Specialist literature must be referred to for design and construction of such structures. The adequacy of such designs shall be demonstrated by adequate, appropriate experimentation and nonlinear dynamic structural analyses.~~

Also, precast and/or pre-stressed concrete members may be used, only if they are designed to provide similar level of ductility as ~~that of~~ monolithic RC structures with the same lateral force ~~load~~-resisting system identified in IS 1893 (Part 1): 2016~~ during or after an earthquake~~. Provisions of pertinent international seismic design codes and specialist literature should be referred to for design and construction of such structures. ~~Likewise,~~

1.1.3 –

All RC frames, RC walls and their elements in a structure need not be designed to resist lateral loads and the designer can judiciously identify the lateral ~~load~~ force resisting system based on relative stiffness and location in the building and design those members for full lateral force. RC monolithic members assumed not to participate in the lateral force resisting system (*see* **3.6**~~7~~) shall be permitted, provided that their contribution in resisting lateral load is not considered and their effect on the seismic response of the system is accounted for. Consequence of failure of structural and non-structural members which are not part of the lateral force resisting system (also known as "gravity load-resisting system") shall also be considered in design.

3.2 – Boundary Elements

These are portions along the ends of a structural wall that are ~~strengthened~~ reinforced by concentrated longitudinal reinforcement and confined by transverse reinforcement. Their ~~y may have the same~~ thickness may preferably be the same as that of the wall web, however in some cases larger thickness may be required.

3.3 – Columns

These are members (generally vertical) of moment resisting frames ~~with~~ designed for axial, flexural and shear~~ing~~ actions.

3.4 – ~~Cover~~ Concrete Cover

It is that portion of an RC ~~concrete~~ structural member which is not confined by transverse reinforcement.

3.5 – Transverse Reinforcement

It is a continuous bar having a 135° hook with an extension of ~~8~~ 6 times diameter (but not < 65~~75~~ mm) at one end and a hook not less than 90° with an extension of ~~8~~ 6 times diameter (but not < 65~~75~~ mm) at the other end. The hooks shall engage peripheral longitudinal bars. In general, the 90° hooks of two successive cross-ties engaging the same longitudinal bars shall be alternated end for end. Transverse reinforcement (also called hoops) in columns and beams is typically ~~called~~ in the form of links, spirals and ~~stirrups and that in beams is called~~ cross-ties.

3.5.1[1] - Link

A link (or hoop) is a single steel bar bent into a closed loop having a 135° hook with an extension of 6 times its diameter, ~~(~~ but not ~~<~~ less than 65 mm~~)~~, at each end ~~, which is embedded in the confined core of the section, and placed normal to the longitudinal axis of the RC beam or column~~. The extension shall be embedded in confined concrete core and shall be placed normal to the longitudinal axis of the RC beam or column.

3.5.2 - Spiral

Spiral is a continuous helical bar wrapping around the longitudinal bars in a column.

3.5.3 - Cross-tie

A cross-tie is a single steel bar bent at one end with 135° hook having an extension of 6 times its diameter but not less than 65 mm; and at the other end with either a 135° hook (type 1) or a 90° hook (type 2) with an extension of 6 times its diameter. A cross-tie shall engage the peripheral longitudinal bars with hook.

3.6 – Gravity Load Resisting Frame ~~Columns in Buildings~~

It is a frame consisting of slabs and/or beams supported by columns and/or walls which are not considered to be a part of the lateral force resisting system.
~~column, which is not part of the lateral load resisting system and designed only for force actions (that is, axial force, shear force and bending moments) due to gravity loads. But, it should be able to resist the gravity loads at lateral displacement imposed by the earthquake forces.~~

3.8.1 – 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~~7.6~~.

3.8.2 – Intermediate Moment Resisting Frame (IMRF)

It is a moment-resisting frame with lower ductility and relaxed detailing provisions compared to SMRF and intended for application in lower seismic zones, and which meets the requirements of 9.

[1] Previously 3.9

~~3.8.2~~ 3.8.3 – Ordinary Moment Resisting Frame (OMRF)

It is a moment resisting frame not meeting special seismic detailing requirements for ductile behaviour set by IS 13920.

~~3.9 – Link~~

~~It is a single steel bar bent into a closed loop having a 135° hook with an extension of 8 6 times diameter (but not < 65 75 mm) at each end, which is embedded in the confined core of the section, and placed normal to the longitudinal axis of the RC beam or column.~~

3.9~~10~~ – Structural~~hear~~ Wall (also Called Shear~~tructural~~ Wall)

It is a vertically oriented planar element that is primarily designed to resist lateral force effects (axial force, shear force and bending moment) in its own plane.

~~3.11~~ 3.9.1 – Special Structural ~~Shear~~ Wall (SSW)

It is a structural wall meeting special detailing requirements for ductile behaviour specified in 10.

3.9.2 – Intermediate Structural ~~Shear~~ Wall (ISW)

It is a structural wall with lower ductility and relaxed detailing requirements compared to Special Structural Wall and intended for application in lower seismic zones, which meets the requirements of 11.

3.9.3 – Ordinary ShearStructural Wall (OSW)

It is a structural wall not meeting special seismic detailing requirements for ductile behaviour set by IS 13920.

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 millimetres, loads in Newtons and stresses in MPa, 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 or wall
A_h	Horizontal reinforcement area within spacing S_v
A_k	Area of concrete core of column
A_s	Area of longitudinal reinforcement in a boundary element
A_{sd}	Reinforcement along each diagonal of coupling beam
A_{sh}	Area of cross-section of bar forming spiral or link
A_{st}	Area of uniformly distributed vertical reinforcement in a structural wall along its length
A_{stw}	Area of uniformly distributed vertical reinforcement within the web of a structural wall
A_v	Vertical reinforcement at a joint
b_b	Width of beam
b_w	Width of wide beam, length of a boundary element in a structural wall
B_c ,b_c	Width of column
B_c', D_c'	Spacing of longitudinal bars supported by respectively shorter and longer dimension of rectangular confining link
b_j	Effective width of a beam-column joint
D	Overall depth of beam
D_c	Overall depth of rectangular column (diameter of circular column)
D_k	~~d~~Diameter of column core measured to the outside of spiral or link
d	Effective depth of member
d_b	Diameter of longitudinal bar
d_w	Effective depth of wall section, width of boundary element
E_s	Elastic modulus of steel
f_{ck}	Characteristic compressive strength of concrete cube
f_y	Yield stress of steel reinforcing bars, or 0.2 percent proof strength of reinforcing steel
h	Longer dimension of rectangular confining link measured to its outer face
h_c	Depth of column
h_j	Effective depth of a joint

h_{st}	~~Clear s~~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/ longer cross-section dimension of wall
L_s	Clear span of a couplings beam
M_u	Design moment of resistance of ~~entire~~ RC beam, column or wall section
M_b	Design moment of resistance for beam (Figure 8)
M_c	Design moment of resistance for column (Figure 8)
M_u^{Ah}	Hogging design moment of resistance of beam at end A
M_u^{As}	Sagging design moment of resistance of beam at end A
M_u^{Bh}	Hogging design moment of resistance of beam at end B
M_u^{Bs}	Sagging design moment of resistance of beam at end B
~~M_u^{BL}~~ M_{bl}	Design moment of resistance of beam framing into ~~column~~ joint from the left
~~M_u^{BR}~~ M_{br}	Design moment of resistance of beam framing into ~~column~~ joint from the right
M_{ct}	Design moment of resistance framing into the joint from top
M_{cb}	Design moment of resistance framing into the joint from bottom
M_{uw}	Design moment of resistance of web of RC structural wall alone
P_u	Factored axial load
S_o	Spacing of hoops in columns of intermediate moment resisting frames
S_v	Spacing of links along the longitudinal direction of ~~beam or~~ column
t_w	Thickness of web of RC structural wall
V_{jdX}, V_{jdY}	Design shear demands on joints in X and Y directions respectively
$V_{u,a}^{D+L}$	Factored shear force demand at end A of beam due to dead and live loads
$V_{u,b}^{D+L}$	Factored shear force demand at end B of beam due to dead and live loads
~~V_j~~	~~Design shear resistance at a joint~~
V_u	Factored shear force
V_{us}	Design shear resistance offered at a section by steel links

w_j	Effective depth of a beam-column joint
x_u , x_u^*	Depth of neutral axis from extreme compression fibre
~~A~~α	Inclination of diagonal reinforcement in coupling beam
Δ_u	Elastic lateral displacement due to factored seismic force V_u
~~P~~ρ	Area of longitudinal reinforcement as a fraction of gross area of cross-section in a RC beam, column or structural wall
ρ_w	Area of longitudinal reinforcement as a fraction of gross area of cross-section within the web of a structural wall
ρ_c	Area of longitudinal reinforcement on the compression face of a beam as a fraction of gross area of cross -section
ρ_{max}	Maximum area of longitudinal reinforcement permitted on ~~the tension~~ any face of a beam as a fraction of gross area of cross-section
ρ_{min}	Minimum area of longitudinal reinforcement to be ensured on ~~the tension~~ any face of a beam as a fraction of gross area of cross-section
τ_c	Design shear strength of concrete
$\tau_{c, max}$	Maximum nominal shear stress permitted at a section of RC beam, column or struct ral wall
τ_{jc}	Nominal shear strength of concrete in beam-column joint
τ_{jdX}, τ_{jdY}	Design shear stress demand in beam-column joint for X and Y directions respectively
τ_{uv}	Factored shear stress at critical section of a flat slab for seismic load combination
τ_v	Nominal shear stress at a section of RC beam, column or structural wall

5.1 –

The design and construction of RC ~~reinforced concrete~~ buildings shall be governed by provisions of IS 456, except as modified by the provisions of this standard for those elements participating in lateral force resistance.

5.2 –

Minimum grade of concrete shall be ~~M20 (f_{ck} = 20 MPa), but~~ M25, ~~for buildings, more than 15 m in height in Seismic Zones III, IV and V; and~~ but not less than that required by IS 456 based on exposure conditions.

Maximum grade of concrete shall be M70. When concrete grade is higher than M70 a parabolic stress-strain diagram should be used with the maximum compression strain of 0.002.

5.3 –

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

5.3.1 –

Steel reinforcement~~s~~ used for construction of structural members in special moment resisting frames and special structural walls shall conform to IS 1786, which identifies grade Fe 415S ~~or less (conforming to IS 1786~~ and grade Fe 500S to be acceptable for seismic design applications. ~~and Fe 550, that is; high strength deformed steel bars produced by thermo-mechanical treatment process having elongation more than 14.5 percent, and conforming to IS 1786,~~.

Steel grades 415, 500, and 600 can be used for reinforcement in structural members of intermediate moment resisting frames, intermediate structural walls, and gravity load-resisting frames, walls, and flat slabs.

5.3.2 –

The actual 0.2 percent proof strength of steel bars based on tensile test must not exceed their characteristic 0.2 percent proof strength by more than 125 MPa~~20 percent~~.

5.3.3 –

The ratio of the actual ultimate strength to the actual 0.2 percent proof strength shall be at least 1.15, but preferably more than 1.25.

5.4 –

In RC frame buildings, lintel beams ~~shall~~ should preferably not be integrated into the columns to avoid short column effect. However, ~~W~~when lintel beams need to be integrated into the frame, they ~~shall~~ should be included in the ~~analytical~~ numerical model for structural analysis. Similarly, plinth beams (where provided), and staircase beams and slabs framing into columns ~~shall~~ should be included in the ~~analytical~~ numerical model for structural analysis.

5.5 –

RC ~~regular~~ moment-resisting frame buildings ~~shall~~ should have planar frames oriented along the two principal plan directions of buildings. Irregularities listed in IS 1893 (Part 1) ~~shall~~ should be avoided. Buildings with any of the listed irregularities perform poorly during earthquake shaking~~; in addition, buildings with floating columns and set-back columns also perform poorly~~. When irregularities appear in a building as listed in Tables 5 or 6 of IS 1893 (Part 1), the guidance given therein for the respective irregularity in Tables 5 or 6 should~~shall~~ be followed.

6 – Beams of Special Moment Resisting Frames

6.1 – General

Requirements of this section shall apply to beams that form a part of the lateral force resisting system ~~resisting earthquake-induced effects~~ and are proportioned primarily to resist flexure and shear. Beams, in which the factored axial compressive stress exceeds 0.08 fck, shall be designed as columns per requirements of IS456, in addition to the requirements of 7.

6.1.1 –

Beams ~~shall~~should preferably have a width-to-depth ratio of more than 0.3.

6.1.2 –

Beams shall not have width less than ~~200~~230 mm.

6.1.3 –

Beams ~~shall~~should not have depth D more than 1/4th of clear span. ~~This may not apply to the floor beam of frame staging of elevated RC water tanks.~~

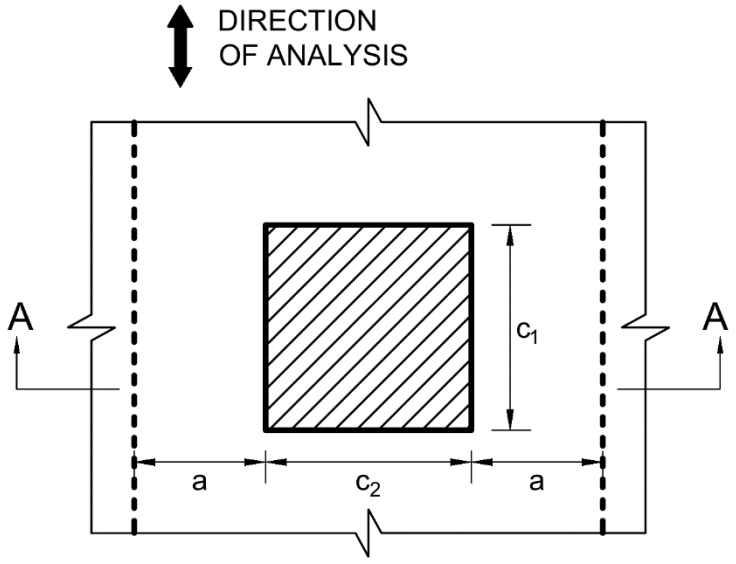
6.1.4 –

Width of beam b_w ~~shall~~should not exceed the width of supporting member c_2 plus a distance on either side of supporting member equal to the smaller of ~~(a) and (b)~~

a) Width of supporting member, c_2, and
b) 0.75 times breadth of supporting member, c_1 (*see* Fig. 1A and Fig. 1B)

For circular columns c_2 shall be replaced by $0.8D_c$, where D_c is column diameter.

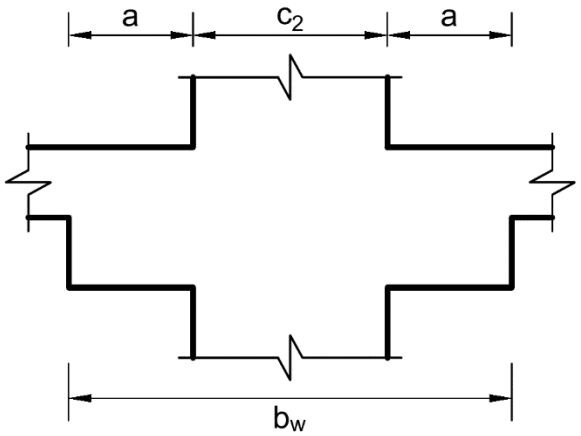
~~Transverse reinforcement for~~ When the beam width ~~of a beam that~~ exceeds ~~width of~~ the column width c_2, transverse reinforcement ~~shall be provided~~ as shown in Fig. 1~~B~~C shall be provided throughout the beam span, including within the beam column joint.



a - NOT GREATER THAN THE SMALLER OF c_2 AND $0.75c_1$

PLAN

1A Plan View of a Beam Column Joint Showing Effective ~~Breadth and~~ Width of Wide Beam ~~Joint~~



SECTION A - A

. **1B Maximum Effective Width of Wide Beam ~~and Required Transverse Reinforcement~~**

FIG.1 WIDE BEAMS ~~BEAM-COLUMN JUNCTIONS~~ (new drawing)

6.2.1 –

The longitudinal reinforcement in beams shall be as given below:

a) Beams shall have at least two 12 mm diameter bars each at the top and bottom faces

b) Minimum longitudinal steel ratio ρ_{min} required on any face at any section is:

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

Where f_{ck} and f_y are expressed in MPa.

6.2.3 –

The amount of ~~L~~longitudinal steel on bottom face of a beam framing into a column (at the face of the column) shall be at least ~~half~~ ½ the steel on its top face at the same section. At exterior joints, the anchorage length ~~calculation~~ shall be determined by consider~~ing~~ the~~is~~ bottom steel ~~to be~~ as tension steel.

6.2.4 –

Amount of ~~L~~longitudinal steel in beams at any section on top or bottom face shall be at least 1/4th of longitudinal steel provided at the top face of the beam at the face of the column~~;~~. W~~w~~hen the top longitudinal steel in the beam at the two supporting column faces is different, the larger of the two amounts shall be considered for the design.

6.2.5 –

At an exterior joint, top and the bottom bars of beams shall be provided with anchorage length, X, beyond the inner face of the column, but not extending below the beam soffit level, equal to development length of the bar in tension, L_d, plus 10 times bar diameter ($10d_b$) minus the allowance for 90° bend~~.~~ (*see* Fig. 2).

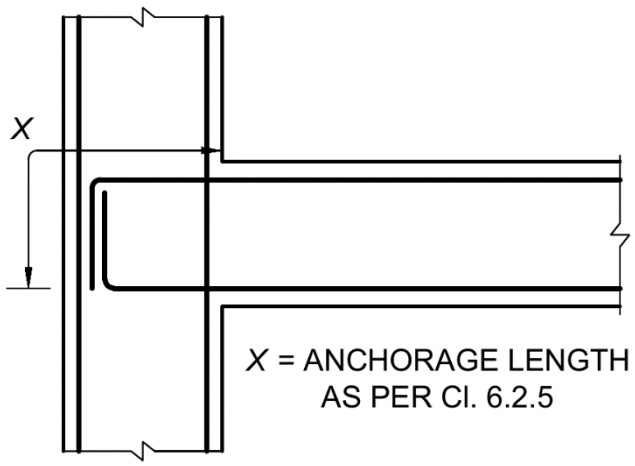


FIG. 2 ANCHORAGE OF LONGITUDINAL BEAM BARS AT EXTERIOR BEAM-COLUMN JOINT

6.2.6.1 − Lap Splices

When adopted, closed links shall be provided over the entire length over which the longitudinal bars are spliced. Further,

a) the spacing of these links shall not exceed 150 mm (*see* Fig.3).

b) the lap length shall not be less than the bar development length in tension of the largest longitudinal reinforcement bar in tension.

c) lap splices shall not be provided,
   1) within a joint~~.~~
   2) within a distance of 2d from joint face of the column; and
   3) within a distance of 2d from critical sections where flexural yielding is likely to occur as a result of inelastic lateral displacements. ~~within a quarter length of the beam adjoining the location where flexural yielding may generally occur under the effect of earthquake forces effects.~~

d) not more than 50 percent of area of steel bars on either top or bottom face shall be spliced at any one section.

.

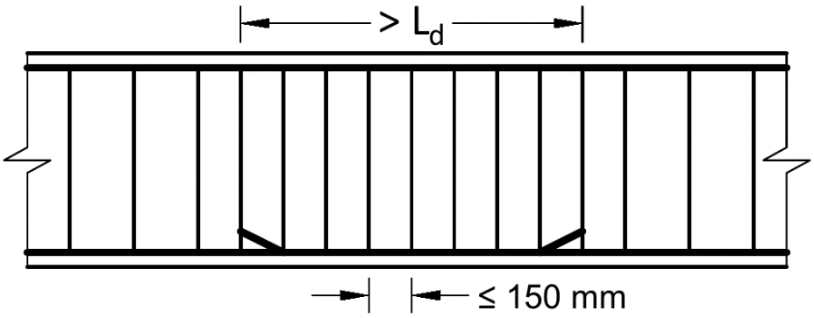


FIG. 3 ~~LAP LENGTH~~ CLOSED LINKS AT LOCATION OF SPLICING OF LONGITUDINAL BARS IN BEAM

6.2.6.2 – Mechanical couplers

Mechanical couplers (conforming to IS 16172) ~~shall~~ may be used when longitudinal steel bars have to be continued ~~-~~due to large beam spans ~~larger than~~ exceeding their manufactured lengths. Further,

a) only those mechanical couplers which are conforming to IS 16172 and capable of developing the specified tensile strength in spliced bar shall be permitted. ~~At any section, not more than 50 percent of bars shall be coupled, and the next location of couplers shall be at least 300 mm away; and~~

b) the spacing between adjacent longitudinal bars shall be also determined by considering ~~based also on the~~ outer size of the coupler to allow easy flow of concrete.

6.2.6.3 – Welded splices

Welded splices shall not be used for longitudinal reinforcement in beams. ~~for a distance equal to two times the depth of the member from the member face or in any location where yielding of reinforcement is likely to take place. At any location, not more than 50 percent of area of steel bars shall be spliced at any one section~~.

Welding of links, ties, ~~inserts~~ or other similar elements to ~~vertical~~ longitudinal beam reinforcement bars ~~required as per design~~ is not permitted~~, in any seismic zone~~.

6.3.1 –

Only vertical links shall be used in beams (*see* Fig. 4A); inclined links shall not be used. ~~And~~
a) ~~I~~In normal practice, a link is made of a single bent bar. But, it may be made of two bars also, namely a U-link with a 135° hook with an extension of 6 ~~8~~ times diameter (but not less than 65~~75~~ mm) at each end, embedded in the core concrete, and a cross-tie (*see* Fig. 4B).

~~c)~~b) ~~t~~The hooks of the links and cross-ties shall engage around peripheral longitudinal bars. Consecutive cross-ties engaging the same longitudinal bars shall have their 90° hooks at opposite sides of the beam. When the longitudinal reinforcement bars are secured by cross-ties in beams that have a slab on one side, the 90° hooks of the cross-ties shall be placed on that side.

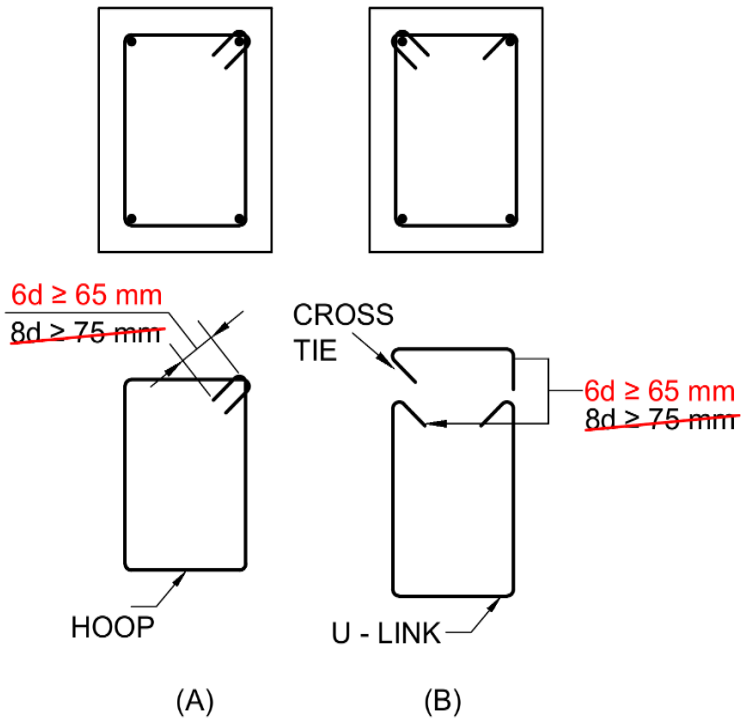


FIG.4 DETAILS OF TRANSVERSE REINFORCEMENT IN BEAMS (revised drawing content shown in red)

6.3.3 –

Shear force capacity of a beam shall be more than larger of,

a) factored shear force as per linear structural analysis; and

b) factored gravity shear force, plus equilibrium shear force when plastic hinges are formed at both ends of the beam (*see* Fig. 5) given by

i) for sway to right:

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

ii) for sway to left:

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

where M_u^{As}, M_u^{Ah}, M_u^{Bs} *and* M_u^{Bh} are sagging and hogging moments of resistance of the beam section at ends A and B, respectively. These shall be calculated as per IS 456. L_{AB} is clear span of beam. $V_{u,a}^{D+L}$ and $V_{u,b}^{D+L}$ are the factored shear~~s~~ forces at ends A and B, respectively, due to vertical loads acting on the span. ~~t~~The partial safety factor for dead and live loads shall be 1.2, and the beam shall be considered to be simply supported for this estimation.

The design shear force demand at end A of the beam shall be larger of the two values of $V_{u,a}$ computed above. Similarly, the design shear force demand at end B shall be larger of the two values of $V_{u,b}$ computed above.

6.3.4 –

In the calculation of design shear force capacity of RC beams, contributions of the ~~following shall not be considered~~ ~~a)~~ bent up bars and ~~b)~~ inclined links shall not be considered.~~, and c)~~ Further, concrete contribution, ~~in the RC~~ shall not be considered when earthquake-induced shear force calculated in accordance with 6.3.3 represents at least one-half of the maximum required shear capacity within the applicable beam length.

6.3.5 – Close Spacing of Links

Spacing of links over a length of 2d at either end of a beam shall not exceed,

a) d/4;
b) ~~6~~ 8 times the diameter of the smallest longitudinal bar; and
c) ~~100~~ 250 mm (*see* Fig. 6).

However, the spacing of links need not be less than 100 mm.

6.3.5.3 –

Construction joint shall not be provided in the regions of beam having closely spaced transverse reinforcement~~s~~.

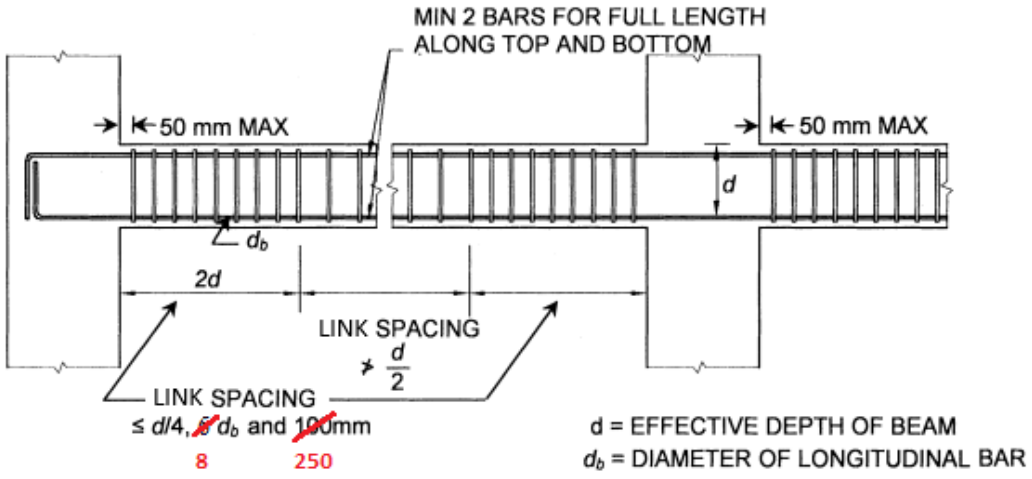


FIG.6 ~~DETAILS~~ DISTRIBUTION OF TRANSVERSE REINFORCEMENT IN BEAMS

7 – Columns and Inclined Members <u>of Special Moment Resisting Frames</u>

7.1 – Geometry

Requirements of this section shall apply to columns and inclined members <u>in special moment resisting frames that form a part of the lateral force resisting system and are proportioned primarily to resist axial forces, flexure, and shear. Inclined members with the factored axial compressive stress due to gravity and earthquake effects of less than or equal to 0.08 f_{ck} shall be designed according to 6.</u> The factored axial compressive stress considering all load combinations relating to seismic loads shall be limited to 0.40 f_{ck} in all such members~~, except in those~~ covered in <u>7</u> ~~10~~.

For columns of shapes other than rectangular and circular (such as 'T', 'X' and '+' shaped), which form part of the lateral ~~load~~ <u>force</u> resisting system, appropriate designs and detailing shall be carried out using specialist literature.

7.1.1 –

The ~~minimum~~ <u>shortest cross-sectional dimension</u> of a column, <u>measured on a straight line passing through the geometric centroid, shall</u> not be less than <u>the larger of</u>,

a) $20d_b$ <u>for rectangular columns and $30d_b$ for circular columns</u>, where d_b is diameter of the largest ~~diameter~~ longitudinal reinforcing ~~ement~~ bar <u>(either single bar or individual bundled bar)</u> in the beam passing through or anchoring into the column at the joint, <u>and</u>~~or~~
b) 300 mm (see Fig. 7).

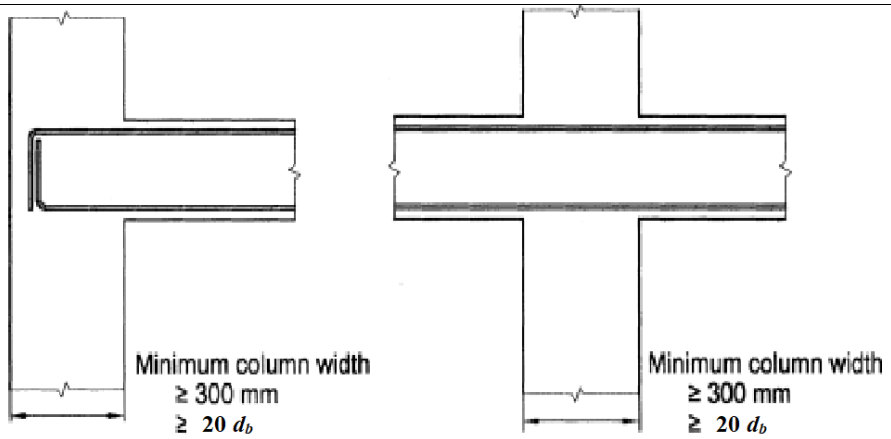


FIG. 7 MINIMUM SIZE OF RC COLUMN~~S~~ BASED ON DIAMETER OF LARGEST LONGITUDINAL REINFORCEMENT BAR IN BEAMS FRAMING INTO IT

7.1.2 –

The cross-section aspect ratio (that is, ratio of smaller dimension to larger dimension of the cross-section of a column or inclined member) shall not be less than 0.4. Vertical members of RC buildings whose cross-sectional aspect ratio is less than 0.4 shall be designed as per requirements of <u>10.1.3</u> ~~9~~.

7.2.1 –

<u>The following requirements shall be followed to avoid yielding of columns which are part of the lateral force-resisting system in special RC frames</u>:

a) At each beam-column joint of a moment-resisting frame, the sum of ~~nominal~~ design moments of ~~design~~ resistance for ~~of~~ columns framing ~~meeting~~ at the ~~at~~ joint ~~(with nominal strength calculated for the factored axial load in the direction of the lateral force under consideration so as to give least column nominal design strength) along each principal plane~~ shall be at least 1.2 ~~1.4~~ times the sum of ~~nominal~~ design moments of resistance ~~design strength of~~ for beams framing ~~meeting~~ at the ~~at~~ same joint ~~in the same plane~~ (see Fig. 8).

b) The design moment of resistance of a column shall be ~~estimated~~ calculated for the factored axial force~~s,~~ consistent with the direction of the lateral forces considered, resulting in the lowest flexural strength ~~arising in all load combination using the design P-M interaction diagram~~.

The moments of resistance shall be summed such that the column moments oppose the beam moments. The equation shall be satisfied for beam moments acting in both principal directions in the vertical plane of the frame considered. The effect of biaxial moments acting in the columns should also be considered.

~~In the event of~~ When a beam-column joint does not ~~conforming to~~ meet the above requirements, the lateral strength and stiffness of the columns framing into the joint shall be ignored~~,~~ and the columns shall be ~~considered to be~~ designed as gravity columns ~~only and shall~~ (not ~~be considered~~ ~~as~~ part of the lateral force~~load~~ resisting system).

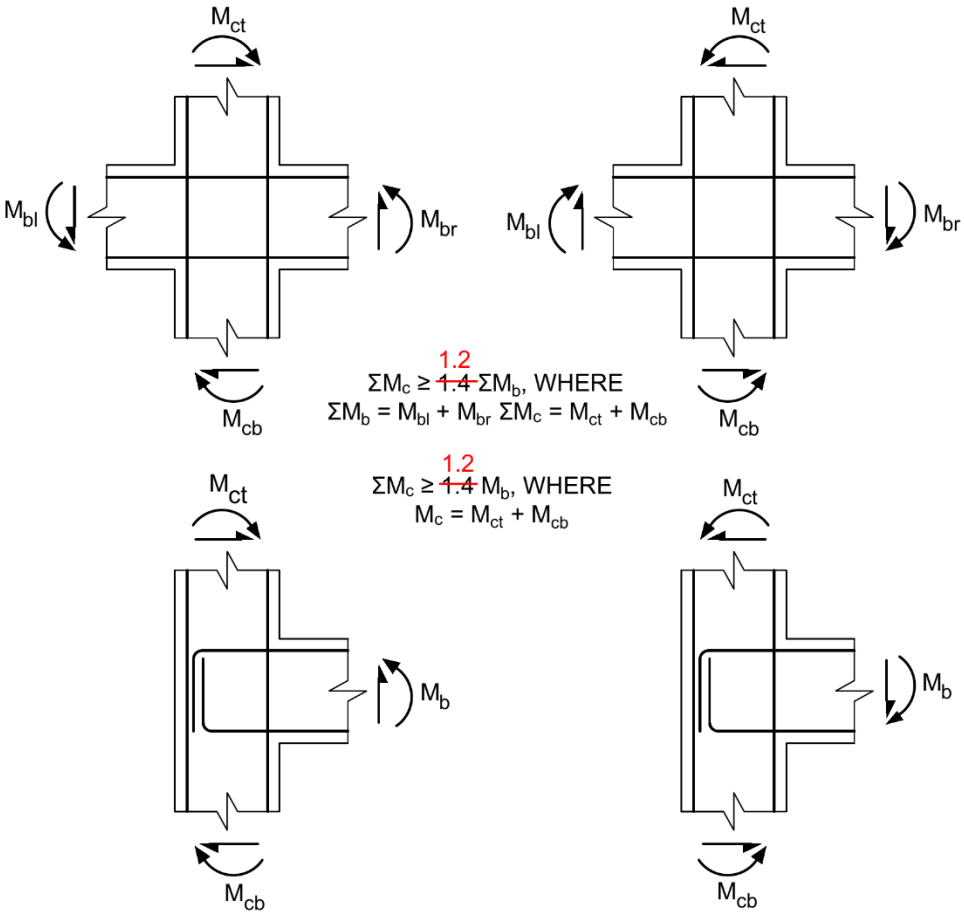


FIG. 8 STRONG COLUMN – WEAK BEAM REQUIREMENT (revised drawing content shown in red)

7.2.1.1 −

The design moments of resistance of a beam shall be estimated based on the principles of mechanics and the ~~limiting strain states of the~~ limit states design method enunciated in IS 456. The design

moment of resistance of a column shall be estimated for factored axial forces arising in all load combination using the design *P-M* interaction diagram.

7.2.1.2 –

This check shall be performed at each joint for both positive and negative directions of shaking in the plane under consideration. Further, in this check, design moments of resistance in beam(s) ~~meeting~~framing at a joint shall be considered in the same direction, and similarly the design moments of resistance of column(s) at the same joint shall be considered to be in the direction opposite to that of the moments in the beams.

7.2.1.3 –

This check shall be waived for

a) single-storey buildings,
b) top storey of the buildings up to 6 storeys tall, and
c) top two storeys of buildings more than 6 storeys tall. ~~roof level only~~, ~~in buildings more than 4 storeys tall.~~

~~The provisions of 7.2 are not applicable for flat slab structures.~~

7.3.1 – Minimum and Maximum Bar Diameter

Longitudinal reinforcement in columns shall meet the following minimum requirements:
a) Circular columns shall have minimum of 6 bars with the minimum 12 mm bar diameter.

b) Maximum diameter of longitudinal reinforcement should be such that 1.25 times bar development length is less than half the clear height of column.

7.3.2.1 − Lap Splices

When adopted, closed links shall be provided over the entire length over which the longitudinal bars are spliced. Further,

a) the spacing of these links shall not exceed 100 mm.

b) the lap length shall not be less than the development length of the largest longitudinal reinforcement bar in tension.

c) lap splices ~~shall~~ should be provided only in the central ~~central~~ half of clear column height. However, if that is not possible these splices may be provided at the column base, but an increased lap splice length of at least 1.3 times the development length L_d of the largest longitudinal reinforcement bar in tension should be used. ~~and not~~
~~1) within a joint, or~~
~~2) within a distance of 2d from face of the beam~~.

d) not more than 50 percent of area of steel bars shall be spliced at any one section/floor, but it shall be permitted to splice all bars at a section, provided that splice length is at least 1.3 ~~1.4~~ times the development length L_d of the largest longitudinal reinforcement bar in tension.

e) lap splices shall not be used for bars of diameter larger than 32 mm for which mechanical splicing shall be adopted.

7.3.2.2 − Mechanical Couplers

Only ~~M~~mechanical couplers conforming to IS 16172 shall be used. Further,
(a) only those mechanical splices capable of developing the specified tensile strength of spliced bar shall be permitted. and ~~At any section, not more than 50 percent of bars shall be coupled. The next section of couplers shall be at least 300 mm away.~~

(b) the spacing between adjacent longitudinal bars shall take into account the outer size of the coupler to allow easy flow of concrete.

7.3.2.3 − Welded Splices

Welded splices shall not be used in columns. ~~for a distance equal to two times the depth of the member from the member face or in any location where yielding of reinforcement is likely to take place. At any section, not more than 50 percent of area of steel bars shall be spliced at any one section. But, welding of links, ties, inserts or other similar elements to vertical reinforcement bars required as per design is not permitted, in any seismic zone~~

7.3.3 –

A column that extends more than 100 mm beyond the confined core owing to architectural requirement (*see* Fig. 9) shall be detailed in the following manner:

a) When the contribution of this area is considered in the estimate of column strength ~~f columns~~, it shall have at least the minimum longitudinal and transverse reinforcement given in this standard

b) When the contribution of this area is not considered in the estimate of column strength ~~of columns~~, it shall have at least the minimum longitudinal and transverse reinforcement given in IS 456.

7.4.1 -

Transverse reinforcement shall consist of closed loop,

a) spiral or circular links in circular columns, and
b) rectangular links in rectangular columns.

In either case, the closed link shall have 135° hook ends with an extension of 6 ~~8~~ times its diameter (but not less than 65 mm) at each end, which are embedded in the confined core of the column (*see* Fig. 10~~A~~).

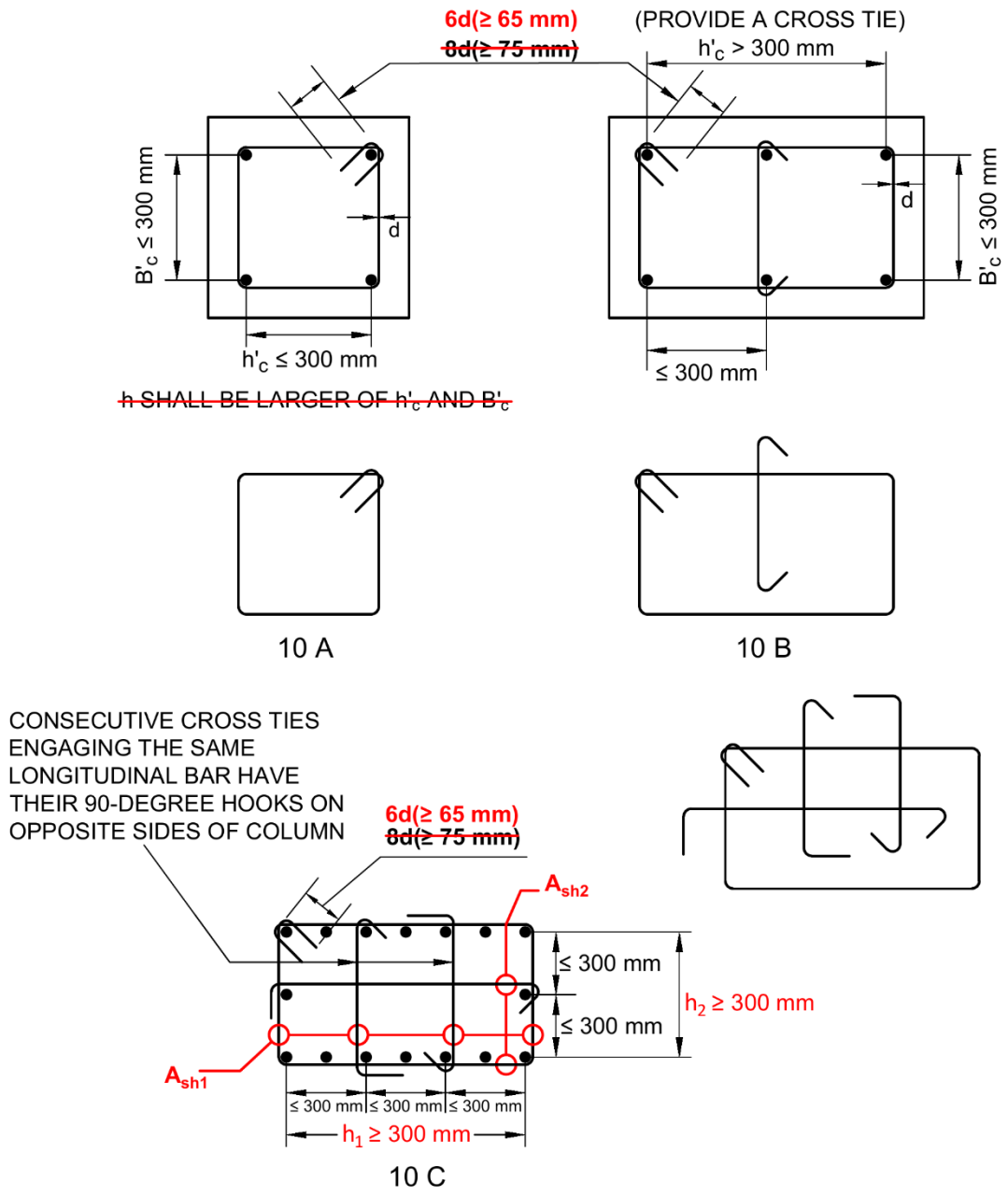


FIG. 10 DETAIL OF TRANSVERSE REINFORCEMENT IN COLUMNS <u>(revised drawing content shown in red)</u>

7.4.2 –

When rectangular links are used,

a) the minimum diameter ~~permitted~~ of transverse reinforcement bars ~~is~~ <u>shall be</u> 8 mm, when diameter of longitudinal bar is less than or equal to 32 mm, and 10 mm, when diameter of longitudinal bar is more than 32 mm;

b) the maximum spacing of ~~parallel legs~~ <u>longitudinal bars supported by corners of a rectangular link~~s~~ and/or cross-tie</u> shall be 300 mm centre to centre;

c) a cross-tie shall be provided~~, if the length of any side~~ when the spacing of the longitudinal bars supported by the corners of a link ~~(*h*'$_c$ or *B*'$_c$)~~ exceeds 300 mm (*see* Fig. 10B); the cross-tie shall be placed perpendicular to the~~is~~ link ~~whose length~~ that supports the corner longitudinal bars with spacing that exceeds 300 mm. Alternatively, a pair of overlapping ~~links~~ cross-ties may be provided within the column (*see* Fig. 10C). In either case, the hook ends of the links and cross-ties shall engage around peripheral longitudinal bars. Consecutive cross-ties engaging the same longitudinal bars shall have their 90° hooks on opposite sides of the column. ~~Cross-ties of the same or smaller bar size as the hoops shall be permitted;~~

d) the maximum spacing of links shall be lesser of one-half the least lateral dimension of the column and 150 mm.~~, except where special confining reinforcement is provided as per **7.6**~~.

~~e) Construction joints shall not be provided in regions of columns with closely spaced transverse reinforcement.~~

7.5 – Design Shear Force in Columns

The design shear force for columns shall be the larger of:

a) factored shear force demand as per linear structural analysis~~;~~, and

b) factored equilibrium shear force demand when plastic hinges are formed at both ends of the beams given by:

1) For sway to right larger of the following two values:

$$V_u = 1.4\left[\frac{M_u^{As} + M_u^{Bh}}{h_{st}}\right]$$

and

$$V_u = 1.4\left[\frac{M_{u1C}^{+} + M_{u2C}^{-}}{h_{st}}\right]$$

2) For sway to left larger of the following two values:

$$V_u = 1.4\left[\frac{M_u^{Ah} + M_u^{Bs}}{h_{st}}\right]$$

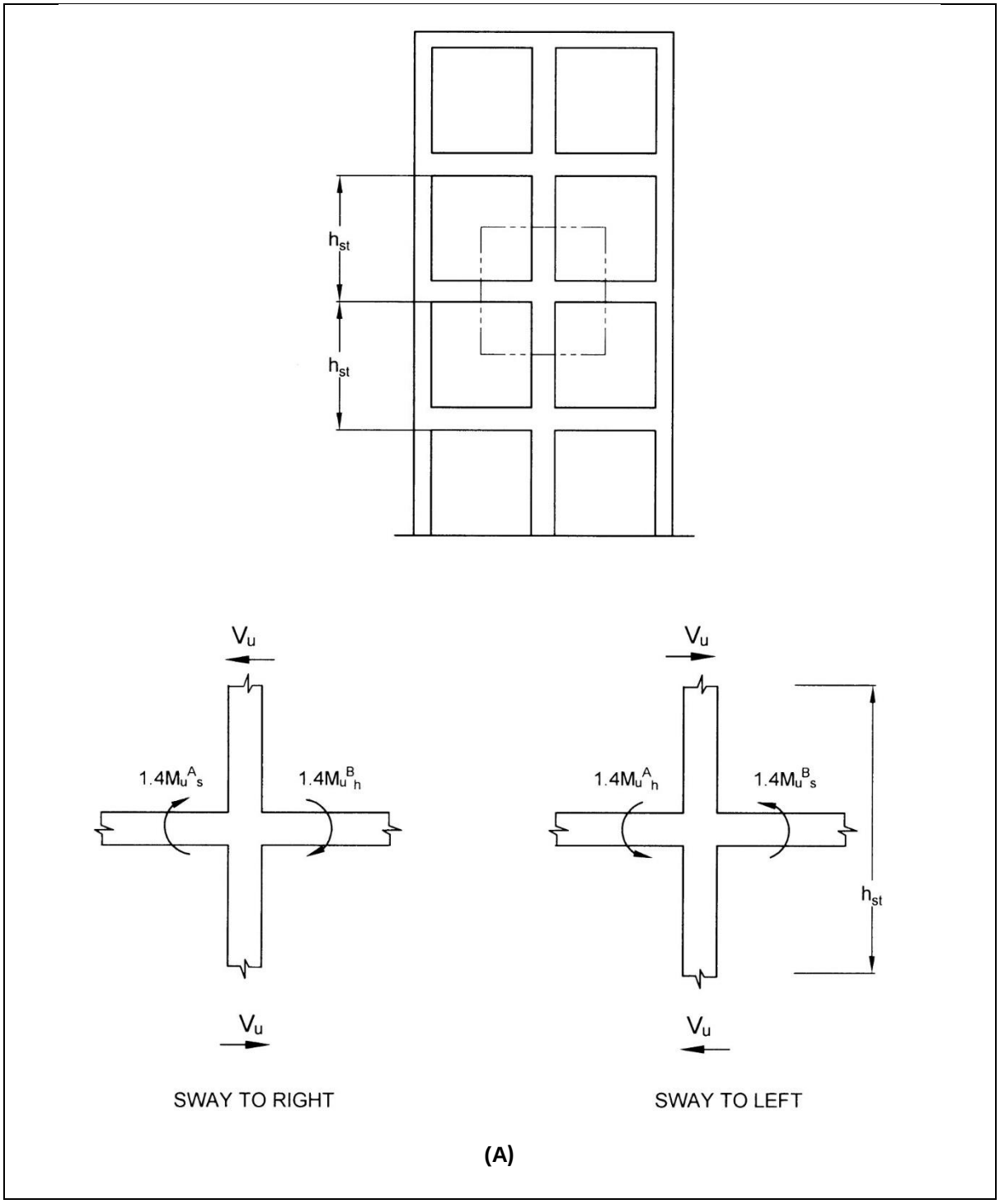
and

$$V_u = 1.4\left[\frac{M_{u1C}^{-} + M_{u2C}^{+}}{h_{st}}\right]$$

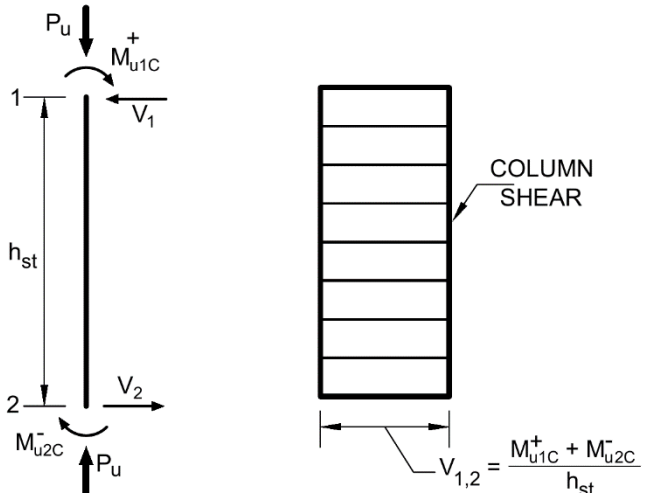
Where h_{st} is the storey height and $M_u^{As}, M_u^{Ah}, M_u^{Bs}$ *and* M_u^{Bh} are design sagging and hogging moments of resistance of beams framing into the column on opposite faces *A* and *B*, respectively, with one hogging moment and the other sagging (*see* Fig. 11A).~~;~~ M_{u1C}^{+}, M_{u1C}^{-}, M_{u2C}^{+} and M_{u2C}^{-} are design positive and negative moments of resistance of column associated with range of P_u (factored axial force) at column ends 1 & 2 (see Fig. 11B).

~~and~~.

The design moments of resistance ~~of~~ for beam and column sections shall be calculated as per IS 456.


h_{st}
h_{st}
V_u
$1.4M_u^A{}_s$
$1.4M_u^B{}_h$
V_u
SWAY TO RIGHT
V_u
$1.4M_u^A{}_h$
$1.4M_u^B{}_s$
h_{st}
V_u
SWAY TO LEFT

(A)



(B) – new drawing

FIG. 11 EQUILIBRIUM DESIGN SHEAR FORCE DEMAND ON COLUMN WHEN PLASTIC HINGES ARE FORMED (A) AT BEAM ENDS AND (B) AT COLUMN ENDS

7.5.1 –

The ~~calculation of~~ design shear force capacity of RC columns shall be calculated as per IS 456.

7.6 – Special Confining Reinforcement

The requirements of this section shall be applicable to ~~be met with in beams and~~ columns, unless a larger amount of transverse reinforcement is required from shear strength considerations given in ~~6.3.3 for beams and~~ 7.5 ~~for columns~~

7.6.1 –

~~Flexural yielding is likely during strong earthquake shaking in columns when the shaking intensity exceeds the expected intensity of earthquake shaking (*see* Fig. 12).~~
~~This s~~Special confining reinforcement shall **(***see* Fig. 12**)**,

a) be provided over a length l_o from the face of the joint, towards ~~mid-span, of beams and~~ mid-height~~s~~ of columns, on either side of the joint where 'l_o' shall not be less than the greatest of ~~is not less than~~
   1) larger cross-sectional ~~lateral~~ dimension of the column ~~member~~ at the joint face or at the section where yielding is likely to occur~~s~~,
   2) 1/6 of clear span of the ~~member~~ column, or
   3) 450 mm.

b) have a spacing not more than
   1) ¼ of minimum dimension of the column;
   2) 6 times the diameter of the smallest longitudinal reinforcement bar, and
   3) 100 mm ~~link~~.

c) have area A_{sh} of cross-section of the bar forming links or spiral of at least:
   1) in circular links or spirals:

$$A_{sh} = Maximum\ of \begin{bmatrix} 0.09 s_v\ D_k \dfrac{f_{ck}}{f_y}\left(\dfrac{A_g}{A_k} - 1\right) \\ 0.024 s_v\ D_k \dfrac{f_{ck}}{f_y} \end{bmatrix}$$

where

s_v = pitch of spiral or spacing of links,

D_k =diameter of core of circular column measured to outside of spiral/link,

f_{ck} =characteristic compressive strength of concrete cube,

f_y = 0.2 percent proof strength of transverse steel reinforcement bars,

A_g =gross area of column cross-section, and

A_k =area of concrete core of column $= \dfrac{\pi}{4} D_k^{\ 2}$

2) in rectangular links the following equations need to be satisfied in both cross-sectional directions of the column core:

$$A_{sh} = Maximum\ of \begin{bmatrix} 0.18 s_v\ h \dfrac{f_{ck}}{f_y}\left(\dfrac{A_g}{A_k} - 1\right) \\ 0.05 s_v\ h \dfrac{f_{ck}}{f_y} \end{bmatrix}$$ where

h = cross-sectional ~~longer~~ dimension of column core measured to the outside edges of the rectangular link composing area A_{sh}; ~~measured to its outer face~~, ~~which does not exceed 300 mm;~~ this dimension is perpendicular to the legs of the links that constitute A_{sh} in each direction of cross-section (see Fig. 10~~B~~C), and

A_k = area of confined concrete core in rectangular link measured to its outer dimensions.

The *h* value shall not exceed 300 mm. In case of larger column dimensions *h* ~~of the link~~ could be reduced by introducing cross-ties (*see* Fig. 10B~~C~~). In such cases, A_k shall be measured as overall core area, regardless of link arrangement. Hooks of cross-ties shall engage peripheral longitudinal bars.

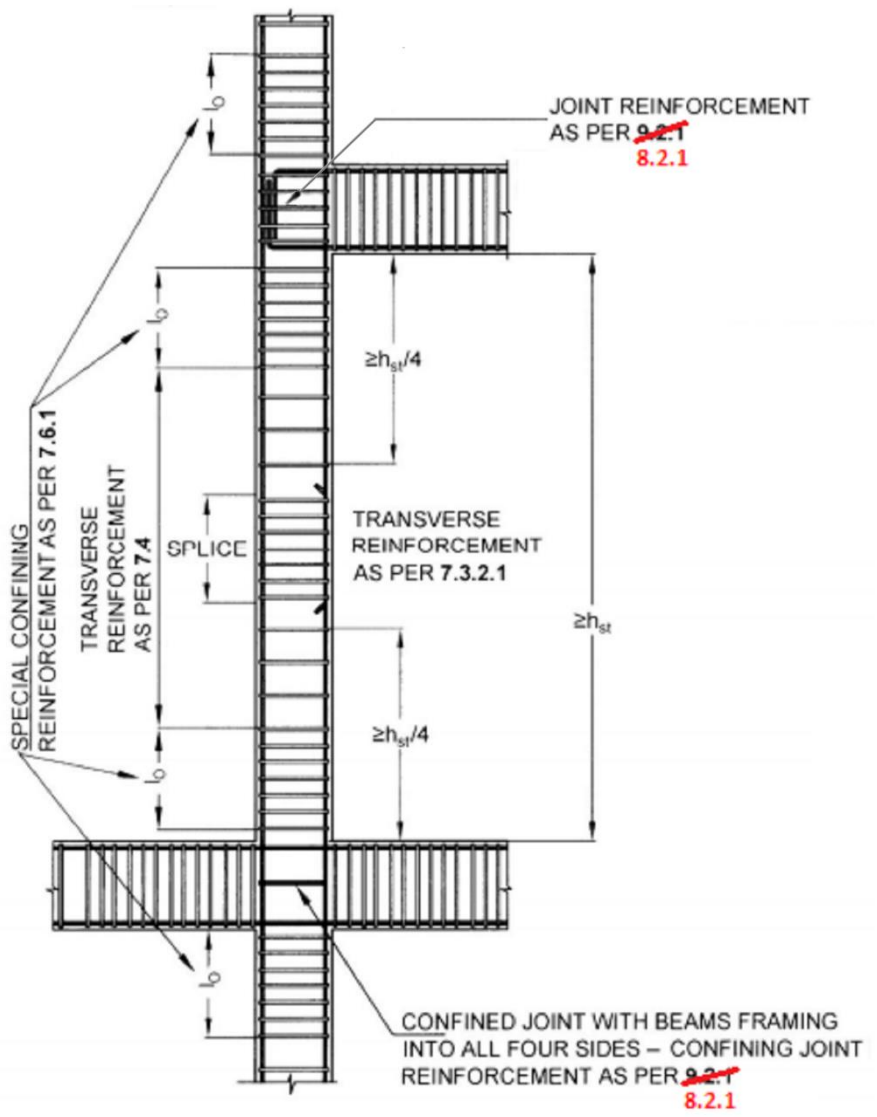


FIG. 12 COLUMN AND JOINT DETAILING

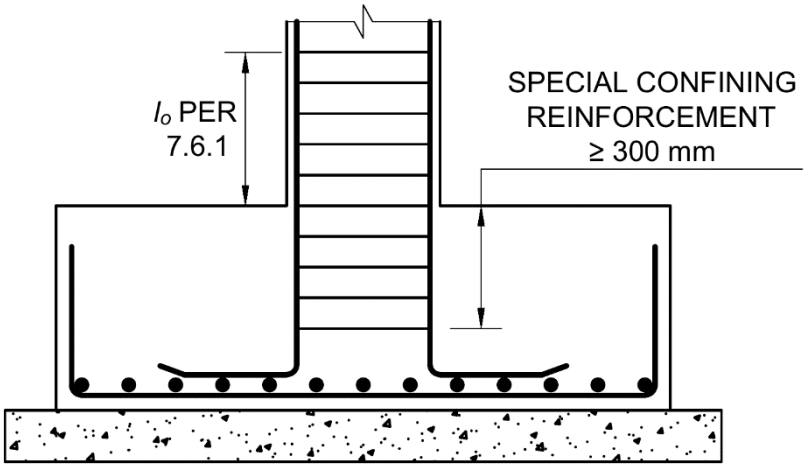


FIG. 13 – PROVISION OF SPECIAL CONFINING REINFORCEMENT IN FOOTING (transverse reinforcement spacing was revised)

7.6.3 –

The point of contra-flexure in the columns in bottom and top storeys tend to shift upwards due to larger joint stiffness at the lower end. Hence, special confining reinforcement shall be provided over the height $2l_o$ (instead of height l_o) at lower ends of these storeys.

~~When the calculated point of contra-flexure, under the effect of gravity and earthquake effects, is not within the middle half of the member clear height, special confining reinforcement shall be provided over the full height of the column.~~

~~9~~8 – Beam-Column Joints of Special Moment Resisting Frames

~~9~~8.1 – Design of Beam-Column Joints for ~~Distortional~~ Shear

~~9~~8.1.1 – *Shear Strength of* Concrete *in a Joint*

The nominal shear strength τ_{jc} of concrete in a beam-column joint shall be taken as

$$\tau_{jc} = \begin{Bmatrix} 1.5\ \sqrt{f_{ck}} & \text{for joints confined by beams on all four faces} \\ 1.2\ \sqrt{f_{ck}} & \text{for joints confined by beams on three faces} \\ 1.0\ \sqrt{f_{ck}} & \text{for other joints} \end{Bmatrix}$$

~~(Aej was removed from the above equations)~~

Shear strength of joint is equal to the product of τ_{jc} and A_{ej} where

A_{ej} is effective shear area of joint ~~equal to given by~~ $b_j \times w_j$, in which b_j is the effective ~~breadth of~~ joint width perpendicular to the direction of shear force and w_j ~~is~~ the effective ~~width~~ depth of joint along the direction of shear force.

The effective ~~width of~~ joint width b_j (*see* Fig. 15) shall be determined as follows:

~~$b_j = \min\left[b_b; b_c + 0.5h_c\right]$ *if* $b_c < b_b$~~

$b_j = b_c$ *if* $b_c < b_b$

and

$b_j = \min[b_b+2x;\ b_c+h_c]$, if $b_c \geq b_b$ where

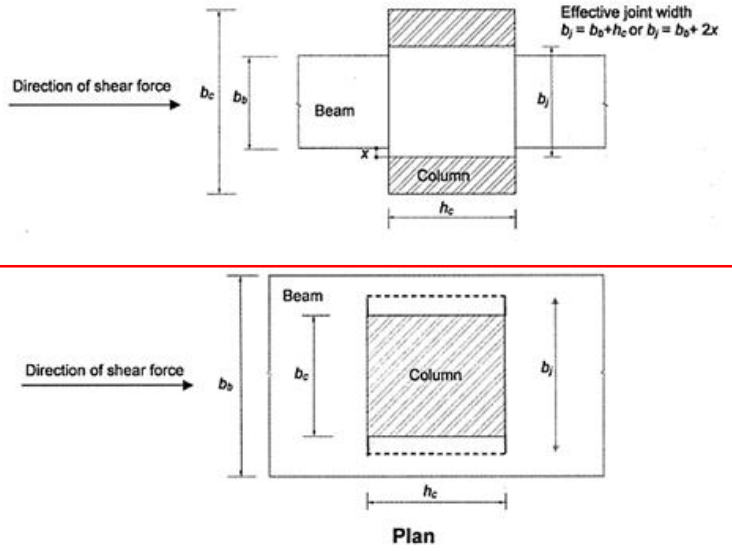
b_b = width of beam ~~and~~

b_c = width of column

h_c = depth of column in considered direction of forces generating shear

x= perpendicular distance between the beam face and the nearest column face.
~~And~~ Effective joint depth shall be determined as follows
$w_j = h_c$ but not less than half the depth of beam framing into joint.
When beams and column framing into a joint have different grade, f_{ck} should be assigned as lesser of the two values.
A joint face is considered to be confined by a beam if the beam width is at least 3/4th of the effective joint width. Extensions of beams by at least one overall beam depth beyond the joint face are considered adequate for confining that joint face.

For calculation of b_j and w_j in circular columns b_c and h_c shall be determined from an equivalent square.



~~FIGURE 15– PLAN VIEW OF A BEAM COLUMN JOINT SHOWING EFFECTIVE BREADTH AND WIDTH OF JOINT~~

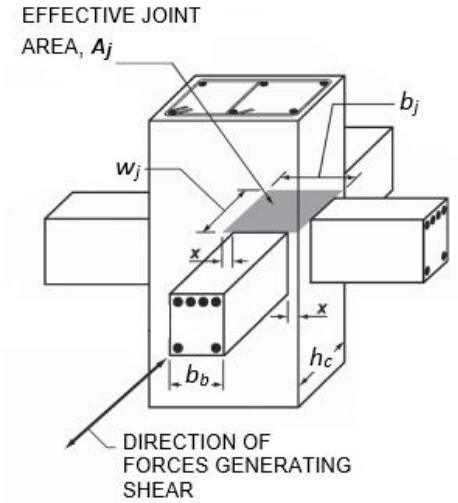


FIG. 15 – AN ISOMETRIC VIEW OF A BEAM-COLUMN JOINT SHOWING ~~EFFECTIVE~~ WIDTH AND DEPTH OF JOINT (source: ACI 318-14 Fig. R18.8.4) (new drawing)

~~9~~8.1.2 – *Design Shear Stress Demand on a Joint*

a) Design shear stress demand acting horizontally along each of the two principal plan directions of the joint shall be estimated from earthquake shaking considered along each of

these directions, using

$\tau_{jdX} = \frac{V_{jdX}}{b_j w_j}$ for shaking along plan direction X of earthquake shaking, and

$\tau_{jdY} = \frac{V_{jdY}}{b_j w_j}$ for shaking along plan direction Y of earthquake shaking.

It shall be ensured that the ~~joint~~ shear capacity of joint concrete estimated using ~~9~~8.1.1 exceeds both τ_{jdX} and τ_{jdY}.

b) Design shear force demands V_{jdX} and V_{jdY} acting horizontally on the joint in principal plan directions X and Y shall be estimated considering that the longitudinal beam bars in tension reach a stress of 1.25 f_y (when over strength plastic moment hinges are formed at beam ends).

~~8.1.3 – *Width of Beam Column Joint*~~

~~When beam reinforcement extends through beam column joint, the minimum width of the column and the shear wall parallel to beam shall be 20 times the diameter of the largest longitudinal beam bar~~.

~~9~~8.2 – Transverse Joint Reinforcement

~~9~~8.2.1 – *Confining Reinforcement in Joints*

a) When all four vertical faces of the joint have~~are having~~ beams framing into them covering at least 75 percent of the width on each face,
   a) At least half the special confining reinforcement required as per 7.6 at the two ends of columns, shall be provided through the joint within the depth of the shallowest beam framing into it; and
   b) Spacing of these transverse links shall not exceed 150 mm.

b) When all four vertical faces of the joint are not having beams framing into them, or when all four vertical faces have beams framing into them but do not cover at least 75 percent of the width on any face,
   1) Special confining reinforcement required as per 7.6 at the two ends of columns shall be provided through the joint within the depth of the shallowest beam framing into it, and
   2) spacing of these transverse links shall not exceed 150 mm.

~~9~~8.2.2 –

In the exterior and corner joints, all 135° hooks of cross-ties should be provided along the outer face of columns.

8.2.3 –

Maximum diameter of beam bar passing through a joint shall not be more than column dimension in line with the beam divided by 20**.**

9 – Intermediate Moment Resisting Frames[2]

9.1– Beams

9.1.1–

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. The design force shall be larger than the two values obtained in part i) and ii), as follows:

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 sagging and hogging moments of resistance of the beam section at ends A and B, respectively. These moments shall be calculated as per IS 456. L_{AB} is clear span of the beam, $V_{u,a}^{D+L}$ and $V_{u,b}^{D+L}$ are the factored shear forces at ends A and B, respectively, due to vertical loads acting on the span. Partial safety factor for dead and live loads shall be 1.2, and the beam shall be considered as simply supported.

9.1.2–

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

9.1.3–

The amount of positive 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.

9.1.4–

The spacing of links over a length of 2d at either end of a beam shall not exceed

(a) d/4,

(b) 8 times the diameter of the smallest longitudinal bar, and

(c) 250 mm.

However, the spacing of links need not be less than 100 mm.

The first link shall be at a distance not exceeding 50 mm from the joint face. Elsewhere, the beam

[2] This section is new – it is based on Section 10 of IITK-GSDMA-EQ11-V2.0 Proposed Draft Provisions and Commentary on Ductile Detailing of RC Structures Subjected to Seismic Forces.

shall have vertical links at a spacing not exceeding d/2.

9.2 – Columns

9.2.1–

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

a) calculated factored shear force as per analysis, and

b) factored equilibrium shear force demand when plastic hinges are formed at both ends of the beams given by:

1) For sway to right:

$$V_u = \left[\frac{M_u^{As} + M_u^{Bh}}{h_{st}}\right] \geq \left[\frac{M_{u1C}^{+} + M_{u2C}^{-}}{h_{st}}\right]$$

2) For sway to left:

$$V_u = \left[\frac{M_u^{Ah} + M_u^{Bs}}{h_{st}}\right] \geq \left[\frac{M_{u1C}^{-} + M_{u2C}^{+}}{h_{st}}\right]$$

where M_u^{As}, M_u^{Ah}, M_u^{Bs} and M_u^{Bh} are design sagging and hogging moments of resistance of beams framing into the column on opposite faces A and B, respectively, with one hogging moment and the other sagging (see Fig. 11). M_{u1C}^{+}, M_{u1C}^{-}, M_{u2C}^{+} and M_{u2C}^{-} are design positive and negative moments of resistance of column associated with range of P_u (factored axial force) at column ends 1 & 2 (see Fig. 11a). The design moments of resistance of beam and column sections shall be calculated as per IS 456.

9.2.2–

The links shall be provided over a length l_o from the joint face at each end of the column storey height. Spacing of the links, s_v, shall not exceed the smallest of

(a) one-half the least lateral dimension of column,

(b) Minimum of 8 times the diameter of smallest longitudinal bar, and

(c) 200 mm.

The length l_o shall not be less than the largest of (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.
The first link shall be provided at a distance not exceeding $s_v/2$ from the joint face.

9.2.3–

Spacing of transverse reinforcement outside the length l_o shall be as per IS 456:2000 column design provisions

9.3– Beam-Column Joints

Joints shall have shear reinforcement not less than that required by clause 9.2.2 for the columns, and the reinforcement shall be provided in the joint within the depth of the shallowest beam framing into the column.

10 – Special Structural ~~Shear~~ Walls

10.1.1 –

The requirements of this section apply to the special ~~shear~~structural walls ~~,~~ that are part of the lateral force resisting system of earthquake-resistant RC building.

10.1.2 –

The minimum wall thickness ~~of special shear walls~~ shall not be less than,

a) larger of 150 mm and h_{st}/ 20 for solid structural walls (without significant openings); and

b) 300 mm for buildings with coupled structural~~shear~~ walls ~~in any seismic zone~~.

The ~~minimum~~ wall thickness ~~provided~~ must also conform to the fire resistance requirements for walls based on occupancy as laid down in IS 456.

When a wall is designed for out-of-plane seismic loads and a perpendicular beam is framing into it, the wall thickness shall be not less than $20d_b$, where d_b is maximum longitudinal bar diameter passing through the wall.

10.1.3 –

The minimum ratio of horizontal wall length ~~wall~~ to thickness shall be ~~4~~ 6.

Walls with length-to-thickness ratio values ranging from 2.5 to 6.0 shall also be designed as special structural walls when their $h_w / L_w < 2.0$, otherwise they shall be treated as RC columns in SMRFs and designed according to 7.

10.1.4 –

Special structural~~shear~~ walls shall be classified as squat~~, intermediate~~ or ~~slender~~flexural depending on the overall height h_w to length L_w ratio, as follows

a) Squat walls: h_w / L_w ~~<1~~ ≤ 2, and
~~Intermediate walls: 1 ≤ h_w / L_w ≤ 2, and~~
b) Flexural (also known as ~~S~~slender) walls: $h_w / L_w > 2$.

10.1.5 –

In the design of flanged wall sections, only that part of the flange shall be considered which extends beyond the face of the web of the structural wall ~~at least~~ for a distance equal to smaller of

a) actual width available;
b) half the distance to the adjacent ~~shear~~ wall web, and
c) 1/10th of the total wall height.

10.1.6 –

~~Special Shear walls shall be provided with uniformly spaced reinforcement in its cross-section along vertical and horizontal. At Least a minimum area of reinforcement bars as indicated in Table 1 shall be provided along vertical and horizontal directions.~~ Distributed reinforcement shall be uniformly spaced in horizontal and vertical directions within the wall. The reinforcement ratio for each direction shall not be less than 0.0025 (based on gross cross-sectional area).

~~**Table 1 Minimum Reinforcement in RC Shear Walls**~~

~~**(Clause 10.1.6)**~~

~~**Sl. NO.**~~	~~**Type of wall**~~	~~**Reinforcement Details**~~
~~i)~~	~~Squat walls~~	$(\rho_h)_{\min} = 0.0025$ $(\rho_v)_{\min} = 0.0025 + 0.5\left(1 - \frac{h_w}{L_w}\right)(\rho_h - 0.0025)$ $(\rho_{v,net}) = (\rho_{v,web}) + \left(\frac{t_w}{L_w}\right)\left[0.02 - 2.5(\rho_{v,web})\right]$ $(\rho_v)_{provided} < (\rho_h)_{provided}$
~~ii)~~	~~Intermediate walls~~	~~$(\rho_h)_{\min} = 0.0025$~~ ~~$(\rho_{v,be})_{\min} = 0.0080$~~ ~~$(\rho_{v,web})_{\min} = 0.0025$~~ ~~$(\rho_{v,net}) = 0.0025 + 0.01375\left(\frac{t_w}{L_w}\right)$~~
~~iii)~~	~~Slender walls~~	~~$(\rho_{v,be})_{\min} = 0.0080$~~ ~~$(\rho_{v,web})_{\min} = 0.0025$~~ ~~$(\rho_{v,net}) = 0.0025 + 0.01375\left(\frac{t_w}{L_w}\right)$~~

10.1.7 –

Reinforcement bars shall be provided in two curtains within the wall's cross-section ~~of the wall~~, with each curtain having bars running along vertical and horizontal directions, when one or more of the following conditions are met

a) factored shear stress demand in the wall exceeds $0.25\sqrt{f_{ck}}$ MPa~~, ; or~~

b) $h_w/L_w \geq 2$ (flexural walls), or

c) wall thickness is 200 mm or higher.

~~When steel is provided in two layers, all vertical steel bars shall be contained within the horizontal steel bars; the horizontal bars shall form a closed core concrete area with closed loops and cross-ties.~~

10.1.8 –

The largest diameter of longitudinal steel bars used in any part of a wall shall not exceed 1/10th of the thickness of that part. The vertical reinforcing bars shall have at least 10 mm diameter, while horizontal reinforcing bars shall have at least 8 mm diameter.

10.1.9 –

The maximum spacing of vertical or horizontal distributed wall reinforcement shall not exceed the smaller of,

a) 1/5th horizontal length L_w of wall~~;~~, and

~~b) 3 times thickness tw of web of wall; and~~

b~~c~~) 450 mm.

The maximum spacing of horizontal and vertical distributed reinforcement within the plastic hinge region of the wall should not exceed 300 mm.

10.1.10 –

Special structural ~~shear~~ walls ~~shall~~ should be founded on properly designed foundations and shall not be discontinued to rest on beams, columns or inclined members. When flexural walls are discontinued at the base they shall be detailed as per 7.6.5 (Fig. 14).

10.1.11 – Plastic hinge region[3]

Special structural walls with $h_w/L_w > 2$ shall be designed for flexural ductility resulting from yielding of the vertical reinforcement in plastic hinge regions. A section where the vertical reinforcement will first yield as a result of lateral displacements is called critical section. For buildings having foundation below the ground (grade) level, plastic hinge region shall extend above the critical section. For buildings having basements below ground (grade) level, plastic hinge region shall extend above and below the critical section.

Length of the plastic hinge region shall not be less than the greater of

a) $0.5L_w + 0.1h_w$, and

b) $M_u/4V_u$

where Lw is the overall length of the longest shear wall or coupled wall in the direction under consideration, and hw is the overall wall height. Mu and Vu are design moment and shear for a load combination including seismic load having largest Mu/4Vu ratio

10.2.1 –

Nominal shear stress ~~demand~~ τ_v ~~on~~ in a wall shall be estimated as;

$$\tau_v = \frac{\alpha \cdot V_u}{t_w d_w},$$

where V_u is factored shear force, t_w is thickness of the web, and d_w is effective depth of wall section (along the length of the wall), which may be taken as 0.8 L_w for rectangular sections.

[3] This is a new clause

Factored shear force V_u shall be obtained from lateral load analysis. Factored shear force shall be increased by factor α to account for flexural overstrength and effect of higher vibration modes by taking one of the following values:

a) α= 1.0 when $h_w / L_w < 2.0$,
b) α= 1.4 when $h_w / L_w \geq 2.0$ but $h_w \leq 60$ m, and
c) α= 2.1 when $h_w > 60$ m.

10.2.2 – Design shear strength of concrete

Design shear strength of concrete τ_c shall be calculated as per Table 19 of IS 456. Reinforcement within 0.4 Lw portion of wall length shall be considered as tension reinforcement for the τ_c calculation, unless depth of neutral axis is calculated by strain compatibility.

10.2.3 – Design of horizontal shear reinforcement

When nominal shear stress demand τ_V on a wall is,

a) more than maximum design shear strength $\tau_{c,\max}$ of concrete (given in Table 20 of IS 456), the wall section shall be re-designed;
b) less than maximum design shear strength $\tau_{c,\max}$ of concrete and more than design shear strength τ_c, ~~design~~ distributed horizontal shear reinforcement ~~shall be provided~~ of area A_h ~~given by~~ shall be determined as follows:

$$A_h = \frac{V_{us}}{0.87 f_y \left(\frac{d}{s_v}\right)} = \frac{V_u - \tau_c t_w d_w}{0.87 f_y \left(\frac{d}{s_v}\right)}$$

which shall not be less than the minimum area of horizontal distributed reinforcement ~~steel~~ as per ~~10.1.5~~ 10.1.6; and

c) less than design shear strength τ_c of concrete, ~~horizontal shear reinforcement shall be~~ the minimum area of horizontal ~~steel~~ reinforcement shall be provided as per ~~10.1.5~~ 10.1.6.

10.3 – Design for Ductile Flexural Behaviour ~~Axial Force and Bending Moment~~

10.3.1. – Design for axial load and bending

A wall shall be proportioned such that the design moment of resistance is greater than the factored bending moment at the section under consideration. Design moment of resistance M_u of the wall section subjected to combined bending moment due to in-plane seismic loading and compressive axial load shall be estimated in accordance with requirements of ~~l~~Limit ~~s~~States ~~d~~Design method given in IS 456~~, using the principles of mechanics involving equilibrium equations, strain compatibility conditions and constitutive laws~~. The moment of resistance of a ~~slender~~ flexural rectangular structural wall section may be estimated using expressions given in Annex A. ~~Expressions given in Annex A are not applicable for structural walls with boundary elements.~~

The factored flexural resistance at any point below the plastic hinge region (e.g. underground parking levels) shall be 1.4 times the factored bending moment at the bottom of the plastic hinge region determined by the analysis. Portion of the wall immediately below the plastic hinge region should contain a minimum 20% additional vertical reinforcement compared to the section at the bottom of the plastic hinge region.

~~10.3.2 –~~

~~The cracked flexural strength of a wall section should be greater than its uncracked flexural~~

~~strength.~~

10.3.2 –

When a wall is subjected to the effect of axial load and combined in-plane and out-of-plane bending due to seismic loads it is recommended to use similar approach as for design of RC columns.

~~**10.3.3 –**~~

~~In structural walls that do not have boundary elements, at least a minimum of 4 bars of 12 mm diameter arranged in 2 layers, shall be concentrated as vertical reinforcement at the ends of the wall over a length not exceeding twice the thickness of RC wall~~

10.3.3 – Ductility check[4]

To ensure that a special structural wall has adequate ductility, the ratio of neutral axis distance, x, and the wall length, L_w, that is, x/L_w, shall not be greater than 0.4 at any wall section which is subjected to combined axial load and bending according to 10.3.1.

A ductility check shall be performed to prove that inelastic rotational capacity of the wall, θ_{ic}, is greater than its inelastic rotational demand, θ_{id}, that is,

$\theta_{ic} > \theta_{id}$

The inelastic rotational demand at the base of a wall, θ_{id}, shall be taken as

$$\theta_{id} = \frac{\left(\Delta_u R - \Delta_u \gamma_w\right)}{\left(h_w - \frac{L_w}{2}\right)} \geq 0.003$$

where

Δ_u = elastic lateral displacement due to factored seismic force,

$\Delta_u \times R$ = design displacement,

γ_w = 1.4 flexural overstrength factor,

$\Delta_u \gamma_w$ = elastic portion of the lateral displacement, and

L_w= the length of the longest wall (within the hinge region) in the direction considered.

The inelastic rotational capacity of a wall, θ_{ic}, shall be taken as

$$\theta_{ic} = \left(\frac{\varepsilon_{cu} L_w}{2x} - 0.002\right) \leq 0.025$$

where ε_{cu} = 0.0035 maximum concrete compression strain

10.4 – Boundary Elements[5]

~~Boundary elements are portions along the wall edges that are strengthened by longitudinal and transverse reinforcement even if they have the same thickness as that of the wall web. It is advantageous to provide boundary elements with dimension greater than thickness of the wall web.~~

10.4.1 – Dimensions of boundary elements

~~Boundary elements shall be provided along the vertical boundaries of walls, when the extreme fibre compressive stress in the wall exceeds 0.2 f_{ck}. Boundary elements may be discontinued at elevations where extreme fiber compressive stress becomes less than 0.15 f_{ck}. Extreme fibre compressive stress shall be estimated using a linearly elastic model and gross section properties.~~

Boundary elements are portions along the wall edges that are strengthened by concentrated vertical reinforcement and transverse links, and shall be provided at both ends of every special structural

[4] This is a new clause

[5] This clause was significantly revised and expanded

wall.

A boundary element may have the same width (d_w) as the wall web, provided that their reinforcement can be accommodated within the wall end zones. However, it is recommended to provide column-like boundary elements (known as barbell sections) when boundary elements resist large seismically induced bending moments.

A boundary element shall meet the following dimensional limits:

a) In walls with rectangular section the length of a boundary element (b_w) shall be at least $0.2L_w$ (unless a more rigorous procedure is followed), and

b) In walls with barbell sections, the width of a barbell section (d_w) shall be at least $h_{st}/10$ but not less than the wall thickness (t_w).

10.4.2 – Design of boundary elements

10.4.2.1 –

A boundary element is an integral part of the wall cross-section and shall have adequate ~~axial~~ flexural ~~load-carrying~~ capacity~~, assuming short column action, so as to enable it~~ to ~~carry axial compression arising from~~ sustain ~~a~~ combined effect of factored gravity axial force~~load~~ and ~~the an additional compressive load induced by the seismic~~ earthquake-induced bending moment ~~force~~. The moment of resistance for a structural wall section with boundary elements may be estimated using expressions given in Annex A.

10.4.2.~~1~~2 –

The load factor for gravity load shall be taken as 0.8, if gravity load gives higher axial compressive strength of the boundary element.

10.4.3 – Vertical reinforcement in boundary elements

At least 4 vertical reinforcement bars arranged in 2 layers shall be provided in each boundary element. The vertical reinforcement ratio in ~~the~~ a boundary element~~s~~ shall not be less than 0.8 percent, and not greater than 6 percent (relative to its cross-sectional area).~~; the practical upper limit would be~~ It is recommended to limit the reinforcement ratio to ~~not to exceed~~ 4 percent in order to avoid congestion.

Bar diameter of vertical reinforcing bars in boundary elements shall be at least 12 mm. If the boundary element terminates on a footing, mat, or pile cap, vertical reinforcement shall extend into these foundation elements by at least the development length.

10.4.4 – Transverse links in boundary elements

10.4.4.1-

Boundary elements~~, where required, as per 10.4.1,~~ shall be provided with ~~special confining reinforcement~~ transverse links throughout their height, having area A_{sh} of cross-section of the bar forming links of at least ~~given by.~~

$$A_{sh} = 0.05\, s_v\, h\, \frac{f_{ck}}{f_y}$$

and have a spacing s_v.

10.4.4.2-

Spacing of transverse links in a boundary element within the plastic hinge region shall not exceed the lesser of~~,~~

a) 1/2 ~~1/3~~ of minimum ~~member~~ dimension of the boundary element; or
b) 6 times diameter of the smallest longitudinal reinforcement bar.

~~c) 100 mm but may be relaxed to 150 mm, if maximum distance between cross-ties/parallel legs of links or ties is limited to 200 mm,~~

but the spacing of links need not ~~to~~ be taken neither less than 100 mm nor more than 150 mm.

10.4.4.3-

Spacing of transverse links in a boundary element outside the plastic hinge region shall not exceed the lesser of~~.~~

a) minimum dimension of the boundary element; or
b) 8 times diameter of the smallest longitudinal reinforcement bar within the boundary element ($8d_b$).

The spacing need not be taken neither less than 150 mm nor more than 200 mm.

10.4.4.4-

Bar diameter of links in boundary elements shall be at least 8 mm.

The transverse links shall be arranged such that the longitudinal bars around the perimeter of the boundary element are laterally supported, similar to RC columns in SMRFs (Clause 7.4). Cross-ties shall have hooks at both ends.
If the boundary element terminates on a footing, mat, or pile cap, transverse links shall extend by at least 300 mm into these foundation elements.

10.5 – Coupled~~ing Beams Shear~~Structural Walls

10.5.1 –

Coupled structural walls are ~~C~~coplanar special structural walls ~~may be~~ connected by means of coupling beams which span across doorways or other openings

10.5.2 – Coupling beams

10.5.2.1 –

A coupling beam with the ratio of clear span L_s and overall depth D (L_s/D) of 4.0 or higher shall be designed as a beam of a special moment resisting frame according to Section 6, with the wall boundary element being treated as a column.

Special diagonal reinforcement is required for coupling beams with L_s/D ratio of less than 2.0 and earthquake induced shear stress τ_{ve} ~~in coupling beam~~ which exceeds the following value

$$\tau_{ve} > 0.1\sqrt{f_{ck}}\left(\frac{L_s}{D}\right),$$

~~where L_s is clear span of coupling beam and D overall depth,.~~ Shear stress τ_{ve} is to be determined based on the factored shear force at the end of a coupling beam V_u and its cross-section with the width t_w and depth D (Fig. 16).

In such a case, the entire earthquake-induced shear, bending moment, and axial compression shall be resisted by two intersecting groups of diagonally placed bars symmetrical about the midspan. ~~diagonal reinforcement alone~~ The reinforcement shall be designed and detailed as follows: ~~Further,~~

a)The required area of ~~this diagonal~~ reinforcement along each diagonal shall be estimated as follows:

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

Where~~, V_u is factored shear force on the coupling beam and~~ α is the angle ~~made by~~ between the diagonal reinforcement and ~~with~~ the horizontal axis.

b)The diagonal reinforcement (area A_{sd}) shall comprise of at least 4 bars placed in two reinforcement curtains. The bar size shall not be less than 12 ~~of 8~~ mm diameter ~~shall be provided along each diagonal. All longitudinal bars along each diagonal shall be enclosed by special c~~ Confinement~~ing~~ in a coupling beam shall be achieved by one of the following alternative arrangements shown in Fig. 16:

i) ~~t~~Diagonal reinforcement bars shall be enclosed by special transverse confining reinforcement (links) within the length L_s. The requirement contained in~~as per~~ 7.6 shall be followed regarding

the amount and spacing of the confining reinforcement. The transverse reinforcement shall have spacing measured parallel to the diagonal bars,~~at a spacing not exceeding 100 mm.~~

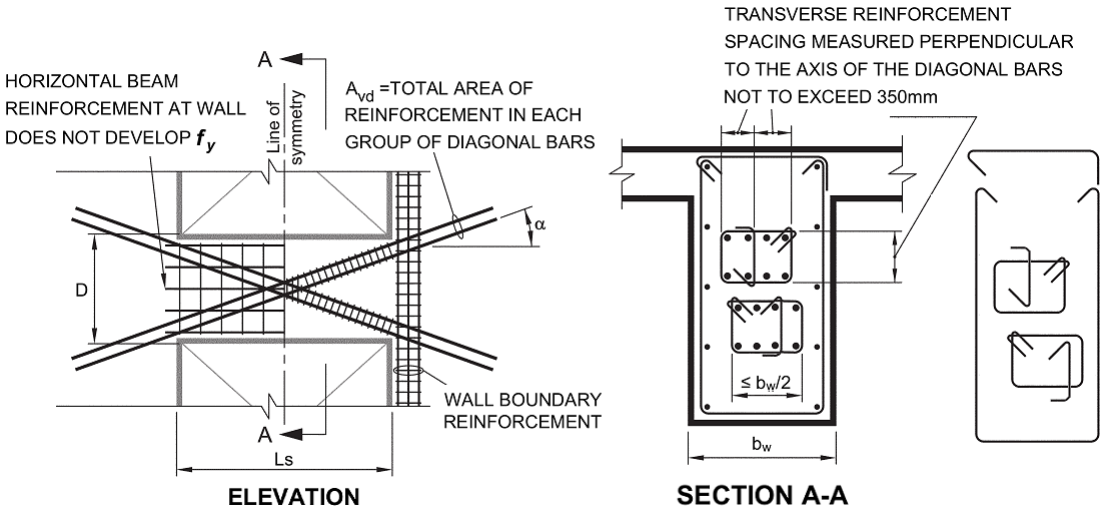
or

ii) The vertical transverse reinforcement shall be provided over the entire beam length. Longitudinal spacing of transverse reinforcement shall be as per 6. Horizontal beam reinforcement shall be provided in two curtains with the minimum reinforcement ratio of 0.0025. The entire coupling beam section shall be confined, and there is no need for confinement of diagonal reinforcement.

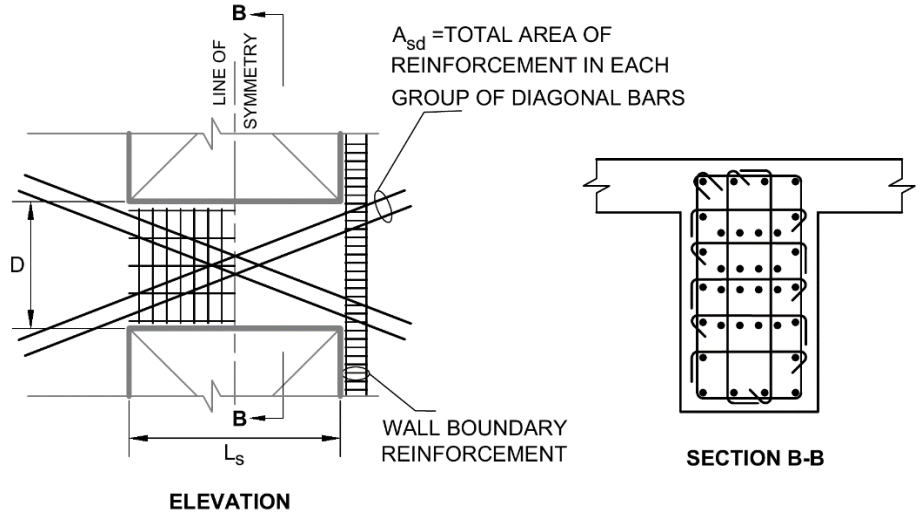
For the purpose of calculating area A_k for diagonal reinforcement in A_{sh} equation (7.6.1), concrete cover shall be assumed on all four sides of each group of diagonal bars.

a) The positive/negative bending moment at the ends of coupling beams is as follows

$$M_u = V_u \cdot L_s / 2$$



i) Confinement of individual diagonals



ii) Confinement of the entire beam section (new drawing)

FIG. 16 COUPLING BEAMS WITH DIAGONAL REINFORCEMENT (source: ACI 318-14 Fig. R18.10.7)

10.5.2.2-

Design of coupling beams with L_s/D ratio ranging from 2.0 to 4.0 shall be performed a) by considering a diagonal reinforcement arrangement according to 10.5.2.1, or

b) by treating the coupling beam as a beam of a SMRF according to 6, with boundary elements treated as columns (Only for coupling beam design purposes).

10.5.3 –

The diagonal and longitudinal reinforcement of a coupling beam shall be anchored into the adjacent walls with an anchorage length of ~~1.5~~ 1.25 times the development length in tension L_d (see Fig. 16).

10.6.2 –

Additional steel reinforcement shall be provided along all four edges of openings in walls. Further,

a) the area of the~~se~~ vertical and horizontal steel should be equal to that of the respective interrupted bars, provided half on either side of the wall in each direction.
b) these vertical bars should extend for full height of the storey in which this opening is present.
c) the horizontal bars should be provided with development length in tension beyond the edge of the opening.

10.7 – Sliding friction resistance check at construction joints

Vertical reinforcement across a horizontal construction joint shall have area, A_{st} given by:

$$\frac{A_{st}}{A_g} \geq \frac{1}{0.87 f_y}\left(\frac{\tau_v}{\mu} - \frac{P_u}{A_g}\right)$$

where τ_v is factored shear stress at the joint, P_u is axial force due to unfactored dead load ~~(positive for compression)~~, ~~and~~ A_g is gross cross-sectional area of wall at the joint location, and μ is the coefficient of friction.

The coefficient of friction, μ, value shall be determined as follows:
a) 0.6 for concrete placed against hardened concrete that is clean, free of laitance, and not intentionally roughened, or
b) 1.0 for concrete placed against hardened concrete that is clean, free of laitance, and intentionally roughened to a full amplitude of approximately 6 mm.

10.8.1 – Anchorage of horizontal reinforcement

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

Horizontal reinforcement shall be anchored at wall ends with a U-bar provided around concentrated vertical bars. Horizontal lap splices shall be placed away from the end of the wall by the minimum five times wall thickness ($5t_w$).

In plastic hinge regions of special structural walls the horizontal reinforcement shall be anchored with straight bar embedment, hook, or mechanical anchorage to develop yield strength f_y within the confined core of the boundary element.

Where the boundary element has sufficient length to develop the horizontal web reinforcement, it shall be permitted to terminate the horizontal web reinforcement with a standard hook.

10.8.2 – Splicing of vertical reinforcement

~~In slender walls (H/L_w > 2), splicing of vertical flexural reinforcement should be avoided, as far as possible, in regions where flexural yielding may take place, which extends for a distance larger of~~

~~a) L_w above the base of the wall; and~~

~~b) 1/6th of the wall height;~~

~~but not larger than $2L_w$.~~

a) Within ~~In~~ the plastic hinge region of ~~slender~~ flexural walls ($H/L_w > 2$), not more than 50% of ~~splicing of vertical flexural~~ reinforcement shall be spliced at the same location, with the lap splice length of $1.25L_d$. When this is not possible all reinforcing bars could be spliced at the same location, but the splice length should be increased to $1.50L_d$.

b) Outside the plastic hinge region it is allowed to splice 100% of vertical wall reinforcement at the same location, with the splice length equal to L_d.

10.8.3 – Splices and cross-ties for vertical reinforcement

10.8.3.1 – ~~Lap splices~~

Distributed vertical reinforcement in the wall within a plastic hinge region shall be confined with cross-ties of minimum 8 mm diameter at maximum 150 mm spacing.

Confinement of distributed vertical reinforcement outside the plastic hinge region of a special structural wall is optional.

~~When adopted, closed links shall be provided over the entire length over which the longitudinal bars are spliced. Further,~~

~~a) the spacing of these links shall not exceed 150 mm.~~
~~b) the lap length shall not be less than the development length of the largest longitudinal reinforcement bar in tension.~~

~~c) lap splices shall be provided only in the central half of clear wall height, and not,~~

~~1) within a joint; or~~

~~2) within a distance of 2d from a location where yielding of reinforcement is likely to take place.~~

~~d) not more than 50 percent of area of steel bars shall be spliced at any one section~~

10.8.3.2 –

Mechanical couplers (conforming to IS 16172) may ~~shall~~ be used, provided they are~~. Further, only those mechanical splices conforming to the above standard and~~ capable of developing the specified tensile strength of spliced bars ~~shall be permitted~~ at centre of clear wall height between the floors. ~~within a distance equal to two times the depth of the member from the beam-column joint or in any location where yielding of reinforcement is likely to take place.~~

10.8.3.3 – Welded splices

Welded reinforcement splices are not permitted in special structural walls. ~~avoided as far as possible. In no case shall they be used for a distance equal to two times the depth of the member from the member face or in any location where yielding of reinforcement is likely to take place. At any location, not more than 50 percent of area of steel bars shall be spliced at any one section.~~

~~Welding of links, ties, inserts or other similar elements to vertical reinforcement bars required as per design is not permitted, in any seismic zone~~

~~10.8.4 –~~

~~In buildings located in Seismic Zones II and III, closed loop transverse links shall be provided around lapped spliced bars larger than 16 mm in diameter. The minimum diameter of such links shall be 1/4th of diameter of spliced bar but not less than 8 mm at spacing not exceeding 150 mm centres.~~

10.9 Additional requirements for squat structural walls[6]

The requirements of this clause apply to squat structural walls with hw/Lw of less than 2.0. Unless otherwise noted, other requirements of 10 are applicable to design of squat structural walls.

[6] This is a new clause

10.9.1 Minimum amount of distributed reinforcement

Horizontal and vertical distributed reinforcement shall be provided in such a manner that the reinforcement ratio is not less than 0.0030 (based on gross cross-sectional area) in each direction.

It is required to provide two curtains of distributed reinforcement.

10.9.2 Longitudinal reinforcement for resisting overturning moment

The vertical tension force required to resist overturning may be provided by a combination of concentrated and distributed reinforcement. The approach for calculating moment resistance may be the same as for flexural walls prescribed in 10.3.1.

10.9.3 Distributed reinforcement for shear resistance

The required amount of distributed horizontal reinforcement for shear resistance, ρ_h, shall be determined as follows:

$$\rho_h = \frac{\tau_v}{0.87 \phi_s f_y}$$

Where shear stress τ_v is determined from 10.2.1 corresponding to the factored shear force V_u.

The amount of distributed vertical reinforcement required to resist shear shall be determined as a function of the ratio of distributed horizontal reinforcement as follows

$$\rho_v = \rho_h - \frac{P_u}{0.87 f_y A_g}$$

where A_g is gross cross-sectional wall area (corresponding to the thickness t_w and length L_w).

When $h_w/L_w < 1.0$ the distributed vertical reinforcement determined for shear resistance shall be in addition to the distributed vertical reinforcement that contributes to resisting the overturning moment (see 10.9.2).

11 – Intermediate Structural Walls[7]

11.1 – Distributed and Concentrated Reinforcement

Distributed horizontal and vertical reinforcement shall be provided according to 10.1.6, 10.1.7, 10.1.8, and 10.1.9.

Concentrated reinforcement in the form of minimum four vertical reinforcing bars with 12 mm diameter (or larger) shall be provided at each end of the wall. The depth of a concentrated reinforcement region shall not be less than 2 times the wall thickness and its width shall be equal to the wall thickness. The reinforcement shall be confined by means of 8 mm diameter (or larger) transverse links at the maximum spacing of 200 mm.

11.2 – Design for shear force

Nominal shear stress τ_V in a wall shall be estimated as;

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

where V_u is the factored shear force, t_w is the web thickness, and d_w is effective depth of the wall section (along the length of the wall), which may be taken as 0.8 L_w for rectangular sections.

[7] This is a new clause

Design shear strength of concrete τ_c shall be calculated as per Table 19 of IS 456. Reinforcement within 0.4 L_w portion of wall length shall be considered as tension reinforcement for the τ_c calculation, unless depth of neutral axis is calculated by strain compatibility.

11.3 – Design for axial load and bending

Design moment of resistance Mu of the wall section subjected to combined bending moment and compressive axial load shall be estimated in accordance with the requirements of Limit States Design method given in IS 456. The moment of resistance of a rectangular structural wall section with uniformly distributed vertical reinforcement may be estimated using expressions given in Annex A.

~~11~~12 –Gravity-load resisting frames, walls, and flat slabs~~Columns in Buildings~~

12.1 – Applied loads

Gravity-load resisting structural members shall be designed and detailed ~~according to~~ for bending moments and shear forces induced when the structure is subjected to 'R/I' times the design lateral displacement Δ_u. Internal forces in the members should be obtained either from ~~the~~ unfactored equivalent static analysis ~~design seismic loads~~ or unfactored dynamic analysis without scaling given by IS 1893 (Part 1).

12.2 ~~11.2~~ – Gravity-Load Resisting Frames

The gravity-load resisting frame members are deemed to be safe ~~provisions in **11.1.1** and **11.1.2** shall be satisfied,~~ when induced bending moments and horizontal shear forces under the ~~said~~ lateral displacement as per **12.1**, combined with factored gravity bending moment and shear force do not exceed the design moment of resistance and design lateral shear capacity of the columns and beams.

12.3~~11.1~~ – Seismic design of columns in gravity-load resisting frames

~~Gravity columns shall satisfy **7.3.2**, **7.4.1** and **7.4.2**. But, spacing of links along the full column height shall not exceed 6 times diameter of smallest longitudinal bar or 150 mm.~~

The seismic demand requirements for columns that are part of gravity-load resisting frames depend on the inelastic flexural deformation demand on the member. When seismic demands on the gravity-load resisting frame are calculated using a linear elastic analysis, the design requirements depend on how much induced bending moment due to the seismic deformation demands from applied loads per 12.1 exceeds the moment of resistance M_u. The columns shall meet the specified detailing requirements, which depend on the magnitude of induced bending moment and axial compression stress, as follows:

Induced bending moment depending on the level of factored axial compressive stress due to factored load combinations including seismic loads		Column detailing requirements
$\leq 0.16 f_{ck}$	$\geq 0.32 f_{ck}$	
$<1.5 M_u$	$<1.0 M_u$	IS456 column detailing requirements

≥1.5M_u but <3.0M_u	≥1.0M_u but <2.0M_u	Columns of Intermediate moment resisting frames (IMRFs) (9.2.2 and 9.2.3)
≥3.0M_u but <5.0M_u	≥2.0M_u but <3.0M_u	Columns of special moment resisting frames (SMRFs) (7.3, 7.4, and 7.6)
Note: Linear interpolation shall be used for intermediate levels of axial compression. Columns in gravity load-resisting frames which are part of an IMRF or SMRF system shall also be designed for shear as per 9.2.1(b) or 7.5(b).		

~~11~~2.1.2 –

~~Gravity columns with factored gravity axial stress exceeding $0.4f_{ck}$ shall satisfy **11.1.1** and shall have transverse reinforcement at least one half of special confining reinforcement required by **7.6**.~~

11~~2~~.42 – Seismic design of beams in gravity-load resisting frames

~~When induced bending moments and shear forces under said lateral displacement combined with factored gravity bending moment and shear force exceed design moment and shear strength of the frame, **11.2.1** and **11.2.2** shall be satisfied.~~

Seismic demand requirements for beams that are part of gravity-load resisting frames depend on the inelastic flexural deformation demand on the member. When seismic demands on the gravity-load resisting frame are calculated using a linear elastic analysis, the design requirements depend on how much induced bending moment due to the seismic deformation demands from applied loads per 12.1 exceeds the moment of resistance M_u. The beams shall meet the specified detailing requirements depending on the magnitude of induced bending moment, as follows:

Induced bending moment	Beam detailing requirements
<1.0M_u	IS456 beam detailing requirements
≥1.0M_u but <3.0M_u	Beams of Intermediate moment resisting frames (IMRFs) (9.1.4)
≥3.0M_u but <5.0M_u	Beams of special moment resisting frames (SMRFs) (6.3.1, 6.3.2 and 6.3.5)
Beams in gravity load-resisting frames which are part of an IMRF or SMRF system shall also be designed for shear as per 6.3.3(b) or 9.1.1(b).	

~~11~~2.2.1 –

~~Mechanical and welded splices shall satisfy **7.3.2.2** and **7.3.2.3.**~~

~~11~~2.2.2 –

~~Gravity columns shall satisfy **7.4** and **7.6**.~~

12.5 –

When an induced bending moment due to seismic deformation demands determined from linear elastic analysis is greater than 5.0 times the factored moment of resistance, the design shall be modified either to reduce the induced bending moment or increase the moment of resistance of the member. This provision applies to columns and beams from 12.3 and 12.4.

12.6- Seismic design of gravity load-resisting walls

The walls shall meet the following detailing requirements, which depend on the magnitude of induced bending moment and axial compression stress:

Induced bending moment depending on the level of factored axial compressive stress due to factored load combinations including seismic loads		Wall detailing requirements
≤0.16f_{ck}	≥0.32f_{ck}	
<0.7M_u	<0.5M_u	Bi-axial bending of wall with single layer of reinforcement
≥0.7M_u but <1.0M_u	≥0.5M_u but <0.7M_u	Bi-axial bending of wall with double layer of reinforcement
≥1.0M_u but <1.2M_u	≥0.70M_u but <0.80M_u	In-plane bending of wall with double layer of reinforcement and vertical concentrated reinforcement as per 10.4.3 (a) with transverse links as that of column in SMRF.
Linear interpolation shall be used for intermediate levels of axial compression. Bending moments in the wall shall be determined considering its gross cross-sectional properties.		

12.7- Slab-column connections of two-way slab without beams (flat slab / flat plate)

Slab-column connections shall preferably not require punching shear reinforcement, and that can be achieved when interstorey drift ratio is limited as under

(a) Interstorey drift ratio is less than or equal to 0.001, or

(b) Interstorey drift ratio is less than or equal to $0.2[0.035 - 0.05(\tau_{uv}/k_s\tau_c)]$.

Where τ_{uv} is factored shear stress at critical section *d/2* from face of support, for seismic load combination only,

$k_s\tau_c$ is concrete shear capacity as per clause 31.6.3 of IS:456 - 2000, and

d = effective depth of slab/drop panel.

13 - Two-way Slabs without Beams (Flat Slabs/Flat Plates)[8]

13.1-

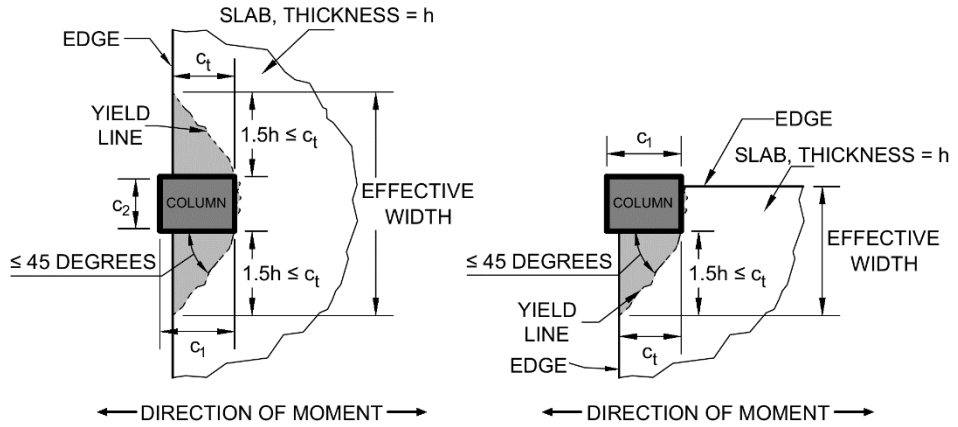
Factored slab moments at the supports shall be calculated for load combinations including seismic loads. Reinforcement for resisting unbalanced moments shall be placed within the column strip defined per IS:456 - 2000.

13.2-

Reinforcement placed within the effective slab width shall be designed to resist α times unbalanced moment, where α is per clause 31.3.3 of IS:456 - 2000.

Effective slab width shall not extend beyond the column face by a distance greater than c_t measured perpendicular to the slab span for exterior and corner connections, and 2c_t for interior connections (see Fig. 17). Note that c_t is distance from the interior face of the column to the slab edge measured parallel to c_1 - but not exceeding c_1, where c_1 is dimension of a rectangular column measured in the direction of the span for which moments are being determined.

[8] This is a new clause



(A) EDGE CONNECTION (B) CORNER CONNECTION

FIG. 17 EFFECTIVE WIDTH FOR REINFORCEMENT PLACEMENT IN EDGE AND CORNER CONNECTIONS (source: ACI 318-14 Fig. R18.4.5.1) (new drawing)

13.3-

At least one-half of the reinforcement in the column strip at the support shall be placed within the effective slab width given in 13.2

13.4-

At least one-fourth of the top reinforcement at the support in the column strip shall be continuous throughout the span.

13.5-

Continuous bottom reinforcement in the column strip shall be at least one-third of the top reinforcement at the support in the column strip.

13.6-

At least one-half of all bottom middle strip reinforcement and all bottom column strip reinforcement at midspan shall be continuous and shall develop full yield strength in tension (equal to development length L_d) at the face of support.

13.7-

At discontinuous edges of the slab, all top and bottom reinforcement at the support shall develop full yield strength in tension (equal to development length L_d) at the face of support.

13.8-

At the critical column sections defined in clause 31.6.1 of IS:456 - 2000, two-way shear caused by factored gravity loads without moment transfer shall not exceed $0.4V_c$, where V_c shall be calculated in accordance with clause 31.6.3 of IS:456 - 2000. This requirement need not be satisfied if the slab satisfies12.7.

14 – Foundations[9]

Foundations of RC frames and structural walls designed in accordance with IS13920 shall be checked for seismic safety according to this clause. Other requirements for foundation design shall be applicable as well, as prescribed by pertinent codes and standards.

14.1 – Required foundation capacity for seismic design

The foundations shall have an overturning moment of resistance and a factored shear resistance of walls/columns and factored sliding shear resistance of footings, not less than 125% of the factored

[9] This is a new clause

moment of resistance for the wall/column section at the base.

14.2 – Foundation movements

Increased displacements due to foundation movements shall be accounted for in the design of lateral force resisting system and the members of gravity-load resisting frame/wall.

Annex A

~~(Clause 10.3.1)~~

Moment of resistance of rectangular ~~shear~~ structural wall section

A-1 (Clause 10.3.1)

The moment of resistance M_u of a ~~slender~~ flexural rectangular structural wall section with uniformly distributed vertical reinforcement may be estimated as :

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

$$\frac{M_u}{f_{ck} t_w L_w^2} = \varphi\left[\left(1+\frac{\lambda}{\varphi}\right)\left(\frac{1}{2}-0.416\frac{x_u}{L_w}\right)-\left(\frac{x_u}{L_w}\right)^2\left(0.168+\frac{\beta^2}{3}\right)\right]$$ where

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

~~$$\frac{x_u^*}{L_w} = \left(\frac{0.0035}{0.0035+(0.002+0.87 f_y/E_s)}\right);$$~~

$$\frac{x^*_u}{L_w} = \frac{0.0035}{0.0035+\left(\frac{0.87 f_y}{E_s}\right)}$$

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

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

ρ = vertical reinforcement ratio = $A_{st}/(t_w L_w)$

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

~~$\beta = (0.002+0.87 f_y/E_s)/(0.0035)$~~

$$\beta = \frac{0.87 f_y}{0.0035 E_s}$$

E_s = elastic modulus of steel, and

P_u = factored compressive axial force on wall

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

$$\frac{M_u}{f_{ck} t_w L_w^2} = \alpha_1\left(\frac{x_u}{L_w}\right)-\alpha_2\left(\frac{x_u}{L_w}\right)^2-\alpha_3-\frac{\lambda}{2}$$

where

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

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

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

x_u/L_w to be used in this expression, shall be obtained by solving the equation:

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

where

$$\alpha_4 = \left(\frac{\varphi}{\beta}-\lambda\right),$$ and

$\alpha_5 = \left(\frac{\varphi}{2\beta}\right)$.

A-2[9] (Clause 10.4.2)

The moment of resistance M_u of a rectangular wall section with distributed vertical reinforcement within the wall web (area A_{st}) and concentrated reinforcement (area A_s) in boundary elements (cross-section $t_w \times a$) can be estimated based on the following procedure. The procedure is applicable for flexural tension failure ($x_u/L_w < x_u^*/L_w$).

First, the depth of neutral axis is to be determined as follows:

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

where

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

$$\lambda = \left(\frac{P_u}{f_{ck} t_w L_w}\right)$$

$$\beta = \frac{0.87 f_y}{0.0035 E_s}$$

Note that reinforcement ratio for distributed vertical reinforcement A_{stw} within the wall web is

$$\rho_w = A_{st} / \left[t_w (L_w - 2a)\right]$$

Internal forces in steel and concrete are as follows

$$T_1 = C_2 = 0.435 f_y \cdot t_w \cdot \rho_w \cdot \beta \cdot x_u$$

$$T_2 = 0.87 f_y \cdot t_w \cdot \rho_w \left[L_w - a - x_u(1+\beta)\right]$$

$$C_1 = 0.87 f_y \cdot t_w \cdot \rho_w \left[x_u(1-\beta) - a\right]$$

$$C_c = 0.36 f_{ck} \cdot t_w \cdot x_u$$

$T_3 = C_3 = 0.87 f_y \cdot A_s$ (assuming that steel yields both in tension and compression)

Next, the moment of resistance M_u can be determined by summing the moments of internal forces about the bottom fibre, as follows:

$$M_u = C_c \cdot d_{Cc} + C_1 \cdot d_{C1} + C_2 \cdot d_{C2} + C_3 \cdot d_{C3} - T_1 \cdot d_{T1} - T_2 \cdot d_{T2} - P_u \cdot d_{Pu}$$

where

$$d_{Cc} = L_w - 0.416 x_u$$

$$d_{C1} = L_w - 0.5\left[x_u(1-\beta) + a\right]$$

$$d_{C2} = L_w - x_u(1 - 2\beta/3)$$

$$d_{C3} = L_w - a$$

$$d_{T1} = L_w - x_u(1 + 2\beta/3)$$

$$d_{T2} = 0.5\left[L_w + a - x_u(1+\beta)\right]$$

$$d_{Pu} = 0.5 L_w$$

Note that steel resultant forces in boundary elements (T_3 and C_3) form a couple moment (at lever arm $d_{C3} = L_w - a$), hence only one of these forces (C_3) appears in the equation for moment of resistance M_u.

[9] This is a new clause