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

by

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with assistance from

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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 and commentary 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.

• Some of the commentary content in this document has been taken from IITK-GSDMA document Proposed Draft Provisions and Commentary on Ductile Detailing of RC Structures Subjected to Seismic Forces (EQ11- V.2.0 and EQ16-V1.0) (https://www.nicee.org/IITK-GSDMA Codes.php). However, the original commentary has been significantly revised and expanded to address current IS 13920: 2016 and proposed changes.

• 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.

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

IS 4326 : 1976 ‘Code of Practice for earthquake-resistant design and construction of buildings’ had provisions for addressing special features in the design and construction of earthquake-resistant RC buildings. It included then, some details for achieving ductility in reinforced concrete (RC) buildings. To keep abreast with the rapid developments and extensive research on earthquake-resistant design of RC structures, the technical committee decided to formulate separate provisions for earthquake-resistant design and detailing of RC structures, which resulted in the formulation of IS 13920 : 1993 ‘Code of Practice for Ductile Design and Detailing of Reinforced Concrete Structures Subjected to Seismic Forces’.

IS 13920 : 1993 incorporated some important provisions that were not covered in IS 4326 : 1976 for design of RC structures. The formulation of the standard addressed the following salient aspects:

a) Significant experience gained from performance of RC reinforced concrete structures that were designed and detailed as per IS 4326 : 1976 during past earthquakes. Many deficiencies were identified and corrected.

b) Provisions on design and detailing of beams and columns as given in IS 4326 : 1976 were revised with an aim to provide them with adequate stiffness, strength and ductility and to make them capable of undergoing extensive inelastic deformations and dissipating seismic energy in a stable manner.

c) Specifications were included on lower limits for strengths of material of earthquake-resistant RC structural
### CODE

- Geometric constraints were imposed on cross-sections of flexural members. Provisions were revised on minimum and maximum reinforcement limits. Requirements were made explicit for detailing of longitudinal reinforcement in beams at joint faces, splices and anchorage requirements. Provisions were included for calculating seismic design shear force, and detailing transverse reinforcement in beams.

- For members subjected to axial load and bending moment, constraints were imposed on cross-sectional aspect ratio and on absolute dimensions. Also, provisions are included for (1) location of lap splices, (2) calculation of seismic design for shear force of structural walls, and (3) special confining reinforcement in regions of columns that are expected to undergo cyclic inelastic deformations during a severe earthquake shaking.

- Specifications were included on a seismic design and detailing of reinforced concrete RC structural walls. These provisions assisted in (1) estimation of design shear force and bending moment demand on structural wall sections, (2) estimation of design moment capacity of wall sections, (3) detailing of reinforcement in the wall web, boundary elements, coupling beams, around openings, at construction joints, and (4) providing sufficient length for development, lap splicing and anchorage of longitudinal steel.

Following the earthquakes that occurred after the release of IS 13920 : 1993 (especially the 1997 Jabalpur, 2001 Bhuj, 2004 Sumatra, 2006 Sikkim, and 2011 Sikkim earthquakes), it was felt that this Code needed further improvement, hence IS 13920 : 2016 was issued. In this revision, the following changes were incorporated in the 2016 edition:

- The title is revised to reflect the ‘Design’ provisions that existed and new ones added, that determine the sizing, proportioning and reinforcement in RC systems.

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- d) Geometric constraints were imposed on cross-sections of flexural members. Provisions were revised on minimum and maximum reinforcement limits. Requirements were made explicit for detailing of longitudinal reinforcement in beams at joint faces, splices and anchorage requirements. Provisions were included for calculating seismic design shear force, and detailing transverse reinforcement in beams.

- e) For members subjected to axial load and bending moment, constraints were imposed on cross-sectional aspect ratio and on absolute dimensions. Also, provisions are included for (1) location of lap splices, (2) calculation of seismic design for shear force of structural walls, and (3) special confining reinforcement in regions of columns that are expected to undergo cyclic inelastic deformations during a severe earthquake shaking.

- f) Specifications were included on a seismic design and detailing of reinforced concrete RC structural walls. These provisions assisted in (1) estimation of design shear force and bending moment demand on structural wall sections, (2) estimation of design moment capacity of wall sections, (3) detailing of reinforcement in the wall web, boundary elements, coupling beams, around openings, at construction joints, and (4) providing sufficient length for development, lap splicing and anchorage of longitudinal steel.

Following the earthquakes that occurred after the release of IS 13920 : 1993 (especially the 1997 Jabalpur, 2001 Bhuj, 2004 Sumatra, 2006 Sikkim, and 2011 Sikkim earthquakes), it was felt that this Code needed further improvement, hence IS 13920 : 2016 was issued. In this revision, the following changes were incorporated in the 2016 edition:

- a) The title is revised to reflect the ‘Design’ provisions that existed and new ones added, that determine the sizing, proportioning and reinforcement in RC systems.
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<td>members meant to resist earthquake shaking. All RC frames, RC walls and their elements in a structure need not be designed to resist lateral loads and the designer may judiciously select effective lateral load resisting RC frames and walls and design those members for full design lateral force. All columns in frames not designed as lateral load resisting frames will be designed as gravity columns in line with the requirements of 44.12. Most provisions that existed earlier have been redrafted. Also, the sequence of sections is re-organized for greater clarity to designers and for removing ambiguities. All the figures have been redrawn which increases the clarity. Some new figures have been added.</td>
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<td></td>
<td>b) The following new provisions were added:</td>
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<td>1) Column-to-beam strength ratio provision has been added in keeping with the strong column — weak beam design philosophy for moment resisting frames;</td>
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<td>2) Shear design of beam-column joints;</td>
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<td></td>
<td>3) Design of slender RC structural walls is improved. The principle of superposition is dropped for estimating the design moment of resistance of structural walls with boundary elements. Instead, procedure is mentioned for estimating the same.</td>
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<td>c) Additional significant modifications incorporated are as under:</td>
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<td></td>
<td>1) The detail of anchorage of longitudinal beam bars at exterior beam-column joint has been revised (see 6.2.5).</td>
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<td></td>
<td>2) Clauses giving detail about mechanical couplers, welded splices in beam, column and shear wall have been added (see 6.2.6.2, 6.2.6.3, 7.3.2.2, 7.3.2.3, 10.8.3.2 and 10.8.3.3).</td>
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<td>3) The minimum diameter of a link has been changed to 8 mm in all cases (see 6.3.2).</td>
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<td>4) The factored axial compressive stress in columns consisting of all load combinations related to seismic loads is limited to 0.40 f_a has been added (see 7.1).</td>
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<td>5)</td>
<td>The minimum dimension of a column has been modified to 20 ( d_b ) or 300 mm (see 7.1.1).</td>
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<td>6)</td>
<td>The expression for area of cross-section ( A_{sb} ) for the bars forming circular or spiral and rectangular links or spiral have been modified [see 7.6.1(c)].</td>
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<tr>
<td>7)</td>
<td>Design of beam-column joints of moment resisting frames has been added and expressions for evaluating nominal shear strength is also given (see 8.1.1, previously 9.1.1).</td>
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<tr>
<td>8)</td>
<td>Expression for calculating special confining reinforcement in boundary element in shear wall is added (see 10.4.4).</td>
</tr>
<tr>
<td>9)</td>
<td>A figure showing reinforcement detail of coupled shear wall with diagonal reinforcement has been added (see 10.5.3).</td>
</tr>
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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
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<td>1.25 (5.3.3).</td>
<td>This is in line with properties of commercially available steel in India and requirements of international codes.</td>
</tr>
<tr>
<td>5)</td>
<td>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.</td>
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<tr>
<td>6)</td>
<td>Signs in the equations for determining required shear capacity for beam sway to left are incorrect and had to be revised (6.3.3).</td>
</tr>
<tr>
<td>7)</td>
<td>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.</td>
</tr>
<tr>
<td>8)</td>
<td>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.</td>
</tr>
<tr>
<td>9)</td>
<td>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 20dB for rectangular columns and 30dB for circular columns, where dB is the largest longitudinal reinforcing bar diameter. Requirement b) 300 mm has remained unchanged.</td>
</tr>
<tr>
<td>10)</td>
<td>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,</td>
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<td>11)</td>
<td>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).</td>
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<tr>
<td>12)</td>
<td>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).</td>
</tr>
<tr>
<td>13)</td>
<td>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.</td>
</tr>
<tr>
<td>14)</td>
<td>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.</td>
</tr>
<tr>
<td>15)</td>
<td>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).</td>
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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:

i. The term “shear wall” has been replaced by “structural wall” in line with the international codes and IS 1893 (Part 1).

ii. 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).

iii. 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).

iv. 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).

v. Minimum distributed reinforcement requirements were revised to align with international codes and simplify current requirements (10.1.6).

vi. 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.
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<td>vii.</td>
<td>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).</td>
</tr>
<tr>
<td>viii.</td>
<td>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).</td>
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<tr>
<td>ix.</td>
<td>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.</td>
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<tr>
<td>x.</td>
<td>Provisions for coupled structural walls have been significantly expanded and clarified (10.5).</td>
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<tr>
<td>xi.</td>
<td>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.</td>
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<td>17)</td>
<td>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.</td>
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<td>b)</td>
<td><strong>The following new provisions have been added:</strong></td>
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<td>1)</td>
<td>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).</td>
</tr>
<tr>
<td>2)</td>
<td>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).</td>
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<tr>
<td>3)</td>
<td>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).</td>
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<tr>
<td>4)</td>
<td>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).</td>
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<tr>
<td>c)</td>
<td><strong>The following provisions require substantial deliberations and are proposed for consideration in future code revisions:</strong></td>
</tr>
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<td>1)</td>
<td>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.”</td>
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<tr>
<td>2)</td>
<td>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.”</td>
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<td>3)</td>
<td>Provisions related to the design of precast and prestressed RC structures in seismic zones (1.1.2) may need to be added.</td>
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<tr>
<td>4)</td>
<td>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).</td>
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<tr>
<td>5)</td>
<td>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).</td>
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<tr>
<td>6)</td>
<td>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.</td>
</tr>
<tr>
<td>7)</td>
<td>Soil-structure interaction needs to be considered in seismic design of foundations, specifically with regards to foundation movements (14.2).</td>
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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 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.
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.
- 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,
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<td>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.</td>
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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 resisting structural systems of RC structures 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.

C1.1 – The code is targeted at buildings even though its title says “structures”. The standard is not applicable to design of bridge piers and large wells.

1.1.1 – Provisions of this standard shall be adopted in all lateral load resisting systems of RC structures located in Seismic 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.

C1.1.1 – Requirements of the original IS 13920 standard issued in 1993 were mandatory only for all structures in seismic zones IV and V, and for important buildings, industrial structures and buildings taller than 5 storeys in zone III. After several RC buildings in Ahmedabad (zone III) collapsed in the 2001 Bhuj earthquake, ductile requirements were made mandatory for all structures in seismic zones III, IV and V. However, ductile detailing requires substantially higher effort in design, construction, and quality control. Hence it is desirable to have the option for zone III to provide lower level of ductility through alternative lateral force resisting systems: Intermediate RC moment resisting frames and Intermediate RC structural walls. This is in line with seismic design provisions in other countries, e.g. Canada. National Building Code of Canada 2015 permits the use of lateral force load resisting...
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 monolithic RC flat slab structures must be 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.

<table>
<thead>
<tr>
<th>CODE</th>
<th>COMMENTARY</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>systems with lower ductility levels, but restricts building height, and seismic hazard level (seismic zone) for specific design applications.</td>
</tr>
<tr>
<td></td>
<td>Members where load combinations involving earthquake load do not govern the design, this code should still be followed, for instance, when wind loads are higher than seismic loads. This issue is clarified in Clause 6.3.1.1 of IS 1893 : 2016 (Part 1) which states, “Even when load combinations that do not contain earthquake effects, indicate larger demands than combinations including them, the provisions shall be adopted related to design, ductile detailing and construction relevant for earthquake conditions, which are given in this standard, IS 13920 and IS 800.”</td>
</tr>
<tr>
<td>1.1.2 –</td>
<td>C1.1.2 –</td>
</tr>
</tbody>
</table>
| Due to poor performance in past earthquakes, some international seismic and RC design codes (e.g. ACI 318-14 and CSA A23.3-14) restrict application of flat slabs (two-way slabs without beams) as lateral force load-resisting system to sites with low seismicity. Basic design and detailing requirements for flat slabs are presented in Clause 13. Most international codes treat bonded and unbonded post-tensioning systems separately for seismic applications, and restrict the applications of unbonded system in earthquake-prone areas by imposing heavy penalties. Indian code IS 1343:2012 does not contain stringent requirements for application of unbonded system in seismic zones, as related to design, durability, corrosion protection, and limited contribution of post-tensioning in design strength of section for seismic applications (only 20-25%). According to many international codes, bonded post-tensioning system can be used in low and moderate seismic zones without any penalty, provided that the RC members satisfy ductility requirements (refer to New Zealand code NZS 3101-2006 and Australian code AS3600). Until Indian code provisions are developed for precast concrete structures, provisions of international codes, e.g. Eurocode 8 (prEN 1998-1-2:2019.2) can be followed. The following references are also relevant for design of precast and prestressed concrete structures:

1. Negro,P. and Toniolo,G., Design Guidelines for Connections of Precast Structures under Seismic Actions, European... |
<table>
<thead>
<tr>
<th>CODE</th>
<th>COMMENTARY</th>
</tr>
</thead>
</table>

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 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 3.6.2) 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.
The following standards contain provisions which, through reference in this text, constitute provisions of this standard. At the time of publication, the editions indicated were valid. All standards are subject to revision, and parties to agreements based on this standard are encouraged to investigate the possibility of applying the most recent editions of the standards indicated below:

<table>
<thead>
<tr>
<th>IS No.</th>
<th>Title</th>
</tr>
</thead>
<tbody>
<tr>
<td>456 : 2000</td>
<td>Plain and reinforced Concrete (fourth revision)</td>
</tr>
<tr>
<td>1343 : 2012</td>
<td>Code of Practice for pre-stressed concrete (second revision)</td>
</tr>
<tr>
<td>1786 : 2008</td>
<td>High strength deformed steel bars and wires for concrete reinforcement (fourth revision, reaffirmed 2013)</td>
</tr>
<tr>
<td>IS 1893</td>
<td>Criteria for earthquake design of structures</td>
</tr>
<tr>
<td>(Part 1) : 2016</td>
<td>General provisions and buildings (sixth revision)</td>
</tr>
<tr>
<td>(Part 2) : 2014</td>
<td>Liquid retaining tanks — Elevated and ground supported</td>
</tr>
<tr>
<td>(Part 4) : 2015</td>
<td>Industrial structures including stack like structures (first revision)</td>
</tr>
<tr>
<td>4326 : 2013</td>
<td>Earthquake resistant design and construction of buildings — Code of Practice (third revision)</td>
</tr>
<tr>
<td>16172 : 2014</td>
<td>Reinforcement couplers for mechanical splices of bars in concrete— Specification</td>
</tr>
</tbody>
</table>

C2.1 –

The original code (IS 13920 : 1993) emerged from the following papers, which also provide the commentary on different specifications of this code:


<table>
<thead>
<tr>
<th>CODE</th>
<th>COMMENTARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>3 – Terminology</td>
<td>For the purpose of this standard, the following definitions shall apply.</td>
</tr>
<tr>
<td>3.1 – Beams</td>
<td>These are members (generally horizontal) of moment resisting frames with flexural and shearing actions.</td>
</tr>
<tr>
<td>3.2 – Boundary Elements</td>
<td>These are portions along the ends of a structural wall that are strengthened reinforced by concentrated longitudinal reinforcement and confined by transverse reinforcement. Their 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.</td>
</tr>
<tr>
<td>3.3 – Columns</td>
<td>These are members (generally vertical) of moment resisting frames with designed for axial, flexural and shearing actions.</td>
</tr>
<tr>
<td>3.4 – Cover Concrete Cover</td>
<td>It is that portion of an RC concrete structural member which is not confined by transverse reinforcement.</td>
</tr>
<tr>
<td>3.5 – Transverse Reinforcement</td>
<td>It is a continuous bar having a 135° hook with an extension of 8 6 times diameter (but not &lt; 6575 mm) at one end and a hook not less than 90° with an extension of 8 6 times diameter (but not &lt; 6575 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 in columns and beams is typically called in the form of</td>
</tr>
<tr>
<td>CODE</td>
<td>COMMENTARY</td>
</tr>
<tr>
<td>------</td>
<td>------------</td>
</tr>
<tr>
<td>links, spirals and stirrups and that in beams is called cross-ties.</td>
<td></td>
</tr>
<tr>
<td><strong>3.5.1</strong> - Link</td>
<td>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.</td>
</tr>
<tr>
<td><strong>3.5.2</strong> - Spiral</td>
<td>Spiral is a continuous helical bar wrapping around the longitudinal bars in a column.</td>
</tr>
<tr>
<td><strong>3.5.3</strong> - Cross-tie</td>
<td>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.</td>
</tr>
<tr>
<td><strong>3.6 – Gravity-Load Resisting Frame Columns in Buildings</strong></td>
<td>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. A 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.</td>
</tr>
</tbody>
</table>

1 Previously 3.9
<table>
<thead>
<tr>
<th>CODE</th>
<th>COMMENTARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.7 – Lateral Force Resisting System</td>
<td>It is that part of the structural system which participates in resisting forces induced by earthquake.</td>
</tr>
<tr>
<td>3.8 – Moment Resisting Frame</td>
<td>It is a three-dimensional structural system composed of interconnected members, without structural walls, so as to function as a complete self-contained unit with or without the aid of horizontal diaphragms or floor bracing systems, in which the members resist gravity and lateral forces primarily by flexural actions.</td>
</tr>
<tr>
<td>3.8.1 – Special Moment Resisting Frame (SMRF)</td>
<td>It is a moment-resisting frame specially detailed to provide ductile behaviour as per the requirements specified in 5, 6, 7 and 8.</td>
</tr>
<tr>
<td>3.8.2 – Intermediate Moment Resisting Frame (IMRF)</td>
<td>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.</td>
</tr>
<tr>
<td>3.8.3 – Ordinary Moment Resisting Frame (OMRF)</td>
<td>It is a moment resisting frame not meeting special seismic detailing requirements for ductile behaviour set by IS 13920.</td>
</tr>
<tr>
<td>3.9 – Link</td>
<td>It is a single steel bar bent into a closed loop having a 135° hook with an extension of 8 times diameter (but not &lt; 75 mm) at each end, which is embedded in the confined core of the section, and placed normal to the...</td>
</tr>
<tr>
<td>CODE</td>
<td>COMMENTARY</td>
</tr>
<tr>
<td>------</td>
<td>------------</td>
</tr>
<tr>
<td></td>
<td>Longitudinal axis of the RC beam or column.</td>
</tr>
<tr>
<td>3.9.10 – Structural Shear Wall (also called Shear Structural Wall)</td>
<td>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.</td>
</tr>
<tr>
<td>3.11 3.9.1 – Special Structural Shear Wall (SSW)</td>
<td>It is a structural wall meeting special detailing requirements for ductile behaviour specified in 10.</td>
</tr>
<tr>
<td>3.9.2 – Intermediate Structural Shear Wall (ISW)</td>
<td>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.</td>
</tr>
<tr>
<td>3.9.3 – Ordinary Shear Structural Wall (OSW)</td>
<td>It is a structural wall not meeting special seismic detailing requirements for ductile behaviour set by IS 13920.</td>
</tr>
</tbody>
</table>
### 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.

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Meaning</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_e$</td>
<td>Effective cross-sectional area of a joint</td>
</tr>
<tr>
<td>$A_{ej}$</td>
<td>Effective shear area of a joint</td>
</tr>
<tr>
<td>$A_g$</td>
<td>Gross cross-sectional area of column or wall</td>
</tr>
<tr>
<td>$A_h$</td>
<td>Horizontal reinforcement area within spacing $S_v$</td>
</tr>
<tr>
<td>$A_k$</td>
<td>Area of concrete core of column</td>
</tr>
<tr>
<td>$A_a$</td>
<td>Area of longitudinal reinforcement in a boundary element</td>
</tr>
<tr>
<td>$A_{ad}$</td>
<td>Reinforcement along each diagonal of coupling beam</td>
</tr>
<tr>
<td>$A_{ah}$</td>
<td>Area of cross-section of bar forming spiral or link</td>
</tr>
<tr>
<td>$A_{sd}$</td>
<td>Area of uniformly distributed vertical reinforcement in a structural wall along its length</td>
</tr>
<tr>
<td>$A_{sdw}$</td>
<td>Area of uniformly distributed vertical reinforcement within the web of a structural wall</td>
</tr>
<tr>
<td>$A_v$</td>
<td>Vertical reinforcement at a joint</td>
</tr>
<tr>
<td>$b_b$</td>
<td>Width of beam</td>
</tr>
<tr>
<td>$b_w$</td>
<td>Width of wide beam; length of a boundary element in a structural wall</td>
</tr>
<tr>
<td>$B_c, b_c$</td>
<td>Width of column</td>
</tr>
<tr>
<td>$B_{c1}, B_{c2}$</td>
<td>Spacing of longitudinal bars supported by respectively shorter and longer dimension of rectangular confining link</td>
</tr>
<tr>
<td>$b_j$</td>
<td>Effective width of a beam-column joint</td>
</tr>
<tr>
<td>CODE</td>
<td>COMMENTARY</td>
</tr>
<tr>
<td>------</td>
<td>------------</td>
</tr>
<tr>
<td>$D$</td>
<td>Overall depth of beam</td>
</tr>
<tr>
<td>$D_c$</td>
<td>Overall depth of rectangular column (diameter of circular column)</td>
</tr>
<tr>
<td>$D_h$</td>
<td>Diameter of column core measured to the outside of spiral or link</td>
</tr>
<tr>
<td>$d$</td>
<td>Effective depth of member</td>
</tr>
<tr>
<td>$d_b$</td>
<td>Diameter of longitudinal bar</td>
</tr>
<tr>
<td>$d_w$</td>
<td>Effective depth of wall section; width of boundary element</td>
</tr>
<tr>
<td>$E_s$</td>
<td>Elastic modulus of steel</td>
</tr>
<tr>
<td>$f_{ck}$</td>
<td>Characteristic compressive strength of concrete cube</td>
</tr>
<tr>
<td>$f_y$</td>
<td>Yield stress of steel reinforcing bars, or 0.2 percent proof strength of reinforcing steel</td>
</tr>
<tr>
<td>$h$</td>
<td>Longer dimension of rectangular confining link measured to its outer face</td>
</tr>
<tr>
<td>$h_c$</td>
<td>Depth of column</td>
</tr>
<tr>
<td>$h_j$</td>
<td>Effective depth of a joint</td>
</tr>
<tr>
<td>$h_{st}$</td>
<td>Clear Storey height</td>
</tr>
<tr>
<td>$L_{AB}$</td>
<td>Clear span of beam</td>
</tr>
<tr>
<td>$L_d$</td>
<td>Development length of bar in tension</td>
</tr>
<tr>
<td>$l_o$</td>
<td>Length of member over which special confining reinforcement is to be provided</td>
</tr>
<tr>
<td>$L_w$</td>
<td>Horizontal length of wall/longer cross-section dimension of wall</td>
</tr>
<tr>
<td>$L_s$</td>
<td>Clear span of a couplings beam</td>
</tr>
<tr>
<td>$M_o$</td>
<td>Design moment of resistance of entire RC beam, column or wall section</td>
</tr>
<tr>
<td>$M_{bo}$</td>
<td>Design moment of resistance for beam (Figure 8)</td>
</tr>
<tr>
<td>$M_c$</td>
<td>Design moment of resistance for column (Figure 8)</td>
</tr>
<tr>
<td>$M_u^{Ah}$</td>
<td>Hogging design moment of resistance of beam at end A</td>
</tr>
<tr>
<td>CODE</td>
<td>COMMENTARY</td>
</tr>
<tr>
<td>--------------</td>
<td>---------------------------------------------------------------------------</td>
</tr>
<tr>
<td>$M_{u}^{As}$</td>
<td>Sagging design moment of resistance of beam at end A</td>
</tr>
<tr>
<td>$M_{u}^{Bh}$</td>
<td>Hogging design moment of resistance of beam at end B</td>
</tr>
<tr>
<td>$M_{u}^{Bs}$</td>
<td>Sagging design moment of resistance of beam at end B</td>
</tr>
<tr>
<td>$M_{u}^{BL}$</td>
<td>Design moment of resistance of beam framing into column joint from the left</td>
</tr>
<tr>
<td>$M_{br}$</td>
<td>Design moment of resistance of beam framing into column joint from the right</td>
</tr>
<tr>
<td>$M_{ct}$</td>
<td>Design moment of resistance framing into the joint from top</td>
</tr>
<tr>
<td>$M_{cb}$</td>
<td>Design moment of resistance framing into the joint from bottom</td>
</tr>
<tr>
<td>$M_{uw}$</td>
<td>Design moment of resistance of web of RC structural wall alone</td>
</tr>
<tr>
<td>$P_{u}$</td>
<td>Factored axial load</td>
</tr>
<tr>
<td>$s_{v}$</td>
<td>Spacing of links along the longitudinal direction of beam or column</td>
</tr>
<tr>
<td>$t_{w}$</td>
<td>Thickness of web of RC structural wall</td>
</tr>
<tr>
<td>$V_{jx}$, $V_{jy}$</td>
<td>Design shear demands on joints in X and Y directions respectively</td>
</tr>
<tr>
<td>$V_{u}^{D+L}_{u,a}$</td>
<td>Factored shear force demand at end A of beam due to dead and live loads</td>
</tr>
<tr>
<td>$V_{u}^{D+L}_{u,b}$</td>
<td>Factored shear force demand at end B of beam due to dead and live loads</td>
</tr>
<tr>
<td>$V_j$</td>
<td>Design shear resistance at a joint</td>
</tr>
<tr>
<td>$V_{us}$</td>
<td>Factored shear force</td>
</tr>
<tr>
<td>$V_{cas}$</td>
<td>Design shear resistance offered at a section by steel links</td>
</tr>
<tr>
<td>$w_{f}$</td>
<td>Effective depth of a beam-column joint</td>
</tr>
<tr>
<td>CODE</td>
<td>COMMENTARY</td>
</tr>
<tr>
<td>-----------------</td>
<td>---------------------------------------------------------------------------</td>
</tr>
<tr>
<td>$x_u, x'_u$</td>
<td>Depth of neutral axis from extreme compression fibre</td>
</tr>
<tr>
<td>$A_d$</td>
<td>Inclination of diagonal reinforcement in coupling beam</td>
</tr>
<tr>
<td>$\Delta_u$</td>
<td>Elastic lateral displacement due to factored seismic force $V_u$</td>
</tr>
<tr>
<td>$\rho_L$</td>
<td>Area of longitudinal reinforcement as a fraction of gross area of cross-section in a RC beam, column or structural wall</td>
</tr>
<tr>
<td>$\rho_w$</td>
<td>Area of longitudinal reinforcement as a fraction of gross area of cross-section within the web of a structural wall</td>
</tr>
<tr>
<td>$\rho_c$</td>
<td>Area of longitudinal reinforcement on the compression face of a beam as a fraction of gross area of cross-section</td>
</tr>
<tr>
<td>$\rho_{max}$</td>
<td>Maximum area of longitudinal reinforcement permitted on the tension any face of a beam as a fraction of gross area of cross-section</td>
</tr>
<tr>
<td>$\rho_{min}$</td>
<td>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</td>
</tr>
<tr>
<td>$\tau_c$</td>
<td>Design shear strength of concrete</td>
</tr>
<tr>
<td>$\tau_{c, max}$</td>
<td>Maximum nominal shear stress permitted at a section of RC beam, column or structural wall</td>
</tr>
<tr>
<td>$\tau_e$</td>
<td>Nominal shear strength of concrete in beam-column joint</td>
</tr>
<tr>
<td>$\tau_{dX}, \tau_{dY}$</td>
<td>Design shear stress demand in beam-column joint for X and Y directions respectively</td>
</tr>
<tr>
<td>$\tau_{uv}$</td>
<td>Factored shear stress at critical section of a flat slab for seismic load combination</td>
</tr>
<tr>
<td>CODE</td>
<td>NOMINAL SHEAR STRESS AT A SECTION OF RC BEAM, COLUMN OR STRUCTURAL WALL</td>
</tr>
<tr>
<td>------</td>
<td>------------------------------------------------------------------------</td>
</tr>
<tr>
<td>$\tau_v$</td>
<td>Nominal shear stress at a section of RC beam, column or structural wall</td>
</tr>
</tbody>
</table>
CODE | COMMENTARY

| 5 – General Specification |  |

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.

C5.1 – Note that provisions of IS 456 are applicable to design of structures subjected to earthquake effects, but IS 13920 provisions are over and above those of IS 456.

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. Grade higher than M70 may be permitted provided that a parabolic stress-strain diagram is used with the maximum compression strain of 0.002.

C5.2 – 1978 version of IS 456 allowed M15 Grade concrete, but minimum grade of concrete as per IS 456 : 2000 is restricted to M20. It is proposed to increase the minimum grade of concrete from M20 to M25, because the latter grade has been widely used in Indian construction practice and ensures a durable structural performance. Note that grade M25 is the lowest grade for “standard concrete” according to IS 456 : 2000 Table 2.

Most international codes specify higher grade of concrete for seismic regions. For example, ACI 318-14 allows concrete with 17.2 MPa (2500 psi) cylinder compressive strength for ordinary constructions, but a minimum of 20.7 MPa (3000 psi) cylinder compressive strength for seismic constructions (the latter value corresponds to M25 concrete grade in India). Eurocode 8 requires a minimum grade C20 for concrete construction in seismic regions; this corresponds to cylinder compressive strength of minimum 20 MPa and the corresponding cube strength of 25 MPa.

IS 456 : 2000 stress-strain curves and stress block parameters are applicable to concrete with conventional strength characteristics, and are expected to give unconservative results for high-strength concrete (grade M70 and higher). Research studies on high-strength concrete specimens have shown that as the compression strength increases, the slope of both ascending and descending portions of the stress-strain curve becomes steeper and the failure is more explosive (Caldarone, 2009), see Fig. C.1. There are several empirical stress-strain equations for high-strength concrete, as discussed by Ayub, Shafiq and Nuruddin (2014). In the absence of IS 456 : 2000 equations it is proposed to keep a parabolic portion of the current stress-strain curve and limit the maximum strain to 0.002, as illustrated in Fig.
<table>
<thead>
<tr>
<th>CODE</th>
<th>COMMENTARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>C2.</td>
<td></td>
</tr>
</tbody>
</table>

### Fig. C1 – Typical stress-strain relationship for concrete with different strengths (new figure)

![Stress-strain diagram](image1)

### Fig. C2 – Stress-strain curve for conventional concrete (IS 456 : 2000) and a proposed curve for high-strength concrete (new figure)

![Stress-strain diagram](image2)

### 5.3 –

Steel reinforcements 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.

### C5.3-

In case of discrepancy, requirements of 5.3.1, 5.3.2 and 5.3.3 shall overrule the requirements of IS 1786.
<table>
<thead>
<tr>
<th>CODE</th>
<th>COMMENTARY</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>5.3.1</strong> – Steel reinforcements 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.</td>
<td><strong>C5.3.1</strong> – The original (1993) version of the code required the use of Fe 415 steel grade (or less) for the seismic design applications. However, specifications for high strength deformed bars have evolved over time. Amendment 1 of IS 1786: 2008 standard (reaffirmed 2013) introduced steel grades 415S and 500S with ductility characteristics suitable for seismic design applications. These steel grades satisfy mechanical property requirements for steel as stipulated by this code (5.3.2 and 5.3.3).</td>
</tr>
<tr>
<td><strong>5.3.2</strong> – 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.</td>
<td><strong>C5.3.2</strong> – When the difference of actual yield strength and specified yield strength is very high, the shear or bond failure may precede the flexural hinge formation, and the capacity design concept may not work. Hence, a restriction is imposed on the maximum difference between the actual yield strength and the specified yield strength of steel to 20 percent – but it is proposed to set the maximum difference to 125 MPa in line with IS 1786.</td>
</tr>
<tr>
<td><strong>5.3.3</strong> – 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.</td>
<td><strong>C5.3.3</strong> – To develop inelastic rotation capacity, a structural member needs adequate length of yield region along axis of the member. The larger the ratio of ultimate to yield moment, the longer the yield region. Therefore, the code requires the ratio of actual ultimate tensile strength (UTS) to actual yield strength (YS) of at least 1.15. The minimum UTS/YS ratio of 1.15 will allow necessary strain hardening and energy dissipation for required ductility. It is required to have actual UTS/YS ratio of 1.25 or higher to ensure sufficient length of plastic hinges and their spread across the frame. The anticipated brittle shear failure mode is mitigated as actual YS is not allowed to exceed specified YS by 125 MPa in 5.3.2 (this is similar to ASTM A706 and ASTM A615 requirements). IS 1786 limits YS only for S grade steel but UTS/YS ratio is still 1.5.</td>
</tr>
<tr>
<td>PROPOSED MODIFICATIONS &amp; COMMENTARY IS 13920 : 2016</td>
<td></td>
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<tr>
<th>CODE</th>
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<tr>
<td><strong>5.4</strong> – In RC frame buildings, lintel beams <strong>shall</strong> preferably not be integrated into the columns to avoid short column effect. However, when lintel beams need to be integrated into the frame, they <strong>shall</strong> be included in the analytical numerical model for structural analysis. Similarly, plinth beams (where provided), and staircase beams and slabs framing into columns <strong>shall</strong> be included in the analytical numerical model for structural analysis.</td>
<td></td>
</tr>
</tbody>
</table>

**C5.4** – In industry practice it is common to provide lintels in the frames without modelling them in the analysis. This practice should be avoided as this may cause a captive column failure. When lintels are not integrated in the frame they should be well integrated within the walls as nonstructural components. |

| **5.5** – RC regular moment-resisting frame buildings **shall** have planar frames oriented along the two principal plan directions of buildings. Irregularities listed in IS 1893 (Part 1) **shall** 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 **shall** be followed. |
6 – Beams of Special Moment Resisting Frames

<table>
<thead>
<tr>
<th>CODE</th>
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<tbody>
<tr>
<td><strong>6.1 – General</strong></td>
<td><strong>C6.1</strong> – Beams are usually subjected to bending moments and shear forces, but internal axial forces may also be induced when beams act as chord members in diaphragms.</td>
</tr>
<tr>
<td><strong>6.1.1</strong> – Beams should preferably have a width-to-depth ratio of more than 0.3.</td>
<td><strong>C6.1.1</strong> – This clause restricts the applicability of ductility provisions to normally proportioned beams. Flexural members with very low width-to-depth ratio, such as deep beams, may develop high flexural resistance but are prone to shear failure under cyclic inelastic deformations. Also, it is difficult to confine concrete through stirrups in narrow beams.</td>
</tr>
<tr>
<td><strong>6.1.2</strong> – Beams shall not have width less than 200 mm.</td>
<td><strong>C6.1.2</strong> – Minimum beam width prescribed by the current clause (200 mm) is significantly less than the minimum column width of 300 mm (7.1.1). RC beams are usually constructed on top of 230 mm thick masonry walls, hence it is appropriate to set beam width equal to wall thickness.</td>
</tr>
<tr>
<td><strong>6.1.3</strong> – Beams shall 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.</td>
<td><strong>C6.1.3</strong> – When the ratio of total depth of a beam to its clear span is greater than 1/4, the beam may behave like a deep beam. The behaviour of deep beams under inelastic cyclic deformations is significantly different from that of relatively shallow ones and different design procedures are applicable. This is a rationale for setting maximum span-to-depth limit for beams in RC frames and is in line with ACI 318-14 Cl.18.6.2.1. Beams with a depth D less than or equal to 1/4th of clear span can be designed as coupling beams according to 10.5.2.</td>
</tr>
<tr>
<td>CODE</td>
<td>COMMENTARY</td>
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</tr>
<tr>
<td>6.1.4 –</td>
<td>It is recommended that the depth of a wide beam (band beam) shall not be less than 16 times the diameter of the largest column reinforcing bar.</td>
</tr>
</tbody>
</table>

Width of beam $b_w$ shall 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. 1BC shall be provided throughout the beam span, including within the beam column joint.

It is recommended that the depth of a wide beam (band beam) shall not be less than 16 times the diameter of the largest column reinforcing bar.
1A Plan View of a Beam Column Joint Showing Effective Breadth and Width of Wide Beam Joint

Transverse reinforcement through the column to confine beam longitudinal reinforcement passing outside the column core

PLAN

DIRECTION OF ANALYSIS

a - NOT GREATER THAN THE SMALLER OF c₂ AND 0.75c₁
1B Maximum Effective Width of Wide Beam and Required Transverse Reinforcement

1C Elevation of Wide Beam and Column showing Required Transverse Reinforcement (previously 1B)

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

| 6.2 – Longitudinal Reinforcement | C6.2 – Longitudinal Reinforcement |
### 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 \( \rho_{\text{min}} \) required on any face at any section is:

\[
\rho_{\text{min}} = 0.24 \frac{f_{ck}}{f_y}
\]

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

### C6.2.1 –

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

b) This clause is meant to ensure adequate reinforcement for resisting the tensile stresses after the cracking of concrete has taken place. Before the cracking the entire concrete section is engaged in resisting tensile stresses.

When the amount of tension steel is not adequate to resist the tensile stresses transferred by the concrete upon cracking, the beam will fail suddenly and in a brittle manner.

This provision governs for members with a large cross-section due to architectural requirements. A sudden failure can also be prevented by ensuring that the moment of resistance of the section is greater than its cracking moment. Note that cantilever T-beams with flange in tension will require significantly higher reinforcement ratio than specified in this clause to prevent brittle failure.

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


### 6.2.2 –

Maximum longitudinal steel ratio \( \rho_{\text{max}} \) provided on any face at any beam section is 0.025.

### C6.2.2 –

This provision is primarily intended to avoid congestion of reinforcement in the beam section, which may cause poor bond between the reinforcement and concrete. This clause prescribes a fairly generous upper bound value for steel ratio. In most situations, a lower than maximum amount of reinforcement should be used, because excessively high amount of longitudinal steel may lead to undesirable brittle compression failure.
6.2.3 –

The amount of longitudinal steel on bottom face of a beam framing into a column (at the face of the column) shall be at least half \( \frac{1}{2} \) the steel on its top face at the same section. At exterior joints, the anchorage length calculation shall be determined by considering the bottom steel to be as tension steel.

C6.2.3 –

This provision recognizes that substantial sagging moment may develop at beam ends during strong shaking which may not be reflected through analysis (Fig. C3). This is due to the nature of seismic excitation, which causes reversible seismic moments, and seismic forces may largely exceed design values during strong earthquake shaking.

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

Note that an additional objective of this provision is to ensure adequate compression reinforcement at the locations of potential yielding, since the compression reinforcement increases ductility. Fig. C4 shows an example for the application of this clause.

In some cases it may be appropriate to follow provisions of international codes (e.g. ACI 318-14) and compare the positive/negative design moment resistances in the beam - as opposed to the amount of longitudinal reinforcement at specific locations within the span. For example, positive design moment resistance at the bottom face of the beam framing into a column (at the face of the column) should be at least one-half the negative moment resistance on its top face at the same section. This is required for post-tensioned beams with a combination of prestressed and non-prestressed reinforcement having different yield strengths, and may be also appropriate for RC beams with longitudinal reinforcement having different yield strengths.
6.2.4 –

**Amount of Longitudinal Steel**: In beams, at any section on the top or bottom face, the amount of longitudinal steel shall be at least 1/4th of the longitudinal steel provided at the top face of the beam at the face of the column. When 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.

**C6.2.4** –

Sufficient reinforcement should be available at any section along the beam span to account for load reversal or unexpected distribution of bending moments. Hence, the code specifies that longitudinal steel is to be provided at both the top and bottom face of the member at any section along its length as a fraction of the corresponding maximum negative moment steel provided at the face of either joint. Fig. C4 is an example for the application of this clause. Refer to comment for clause 6.2.3.
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 (10$d_b$) minus the allowance for 90° bend. (see Fig. 2).

C6.2.5 –
During an earthquake, the zone of inelastic deformation that exists at the end of a beam, may extend for some distance into the column. This makes the bond between concrete and steel ineffective in the beam-column joint region, particularly in case of exterior joints. An experimental research study on this subject was performed by Murty, Rai, Bajpai, and Jain (2003).

The extension of top bars of beam into column below soffit of the beam causes construction problem, hence it is important to use adequate depth of the column members. In fact, the anchorage beyond the 90° bend is effective up to the length of 12 bar diameters, hence even the current provision of anchorage length up to beam soffit is not entirely satisfactory. The anchorage should be preferably limited to the length of 12 bar diameters only after the 90° bend.

In case, it is not possible to provide sufficiently deep columns, one of the following alternative anchorage solutions (in line with Eurocode 8, Part 1 Cl. 5.6.2.2.3) can be used:

a) The beam may be extended horizontally in the form of an exterior stub. (see Fig. C5A), or
b) Anchor plates welded to the end of the bars may be used (see Fig. C5B).

Fig. C5 – Alternative anchorage solutions of longitudinal beam bars at exterior beam-column joint (adapted from Eurocode 8, Part 1)
### 6.2.6 – Splicing of longitudinal bars

**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.

**C6.2.6.1 –**

Lap splices are not reliable under cyclic inelastic deformations, hence they should not be avoided in critical regions of beams. Closely spaced links within the spliced region help improve the performance of a lap splice after the concrete cover spalls off.

Part c) 3) was revised for improved clarity and is in line with ACI 318-14 Cl.18.6.3.3c). Plastic hinge region exists in structural members with flexure dominant seismic behaviour, such as RC beams, columns and structural walls. Special detailing is required in this region to ensure ductile response. Plastic hinge zone in RC structural walls is discussed in 10.1.11.

![FIG. 2 ANCHORAGE OF LONGITUDINAL BEAM BARS AT EXTERIOR BEAM-COLUMN JOINT](image-url)
### 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 per design is not permitted, in any seismic zone.

### C6.2.6.2 –

Application of mechanical couplers may be appropriate for beams with longitudinal bars of large diameters, since in those cases the application of lap splices may be challenging.

Some international codes (e.g. ACI 318-14) recommend staggering of splices when mechanical couplers are used. This is done for constructability purposes and for providing enough space around the splice for installation. Alternate coupling of bars shall not be mandatory, when the requirements of 6.2.6.2 (b) have been met.

### C6.2.6.3 –

Welding of reinforcement in columns can lead to local embrittlement of the steel and should not be permitted at the splice locations. 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. Welding may be permitted to facilitate fabrication or placement of column reinforcement at the

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

![Diagram of lap length and closed links at location of splicing of longitudinal bars in beam](image-url)
6.3 – Transverse Reinforcement

**C6.3**

Transverse reinforcement (links and cross-ties) in RC beams have three roles, namely (i) they carry resist the shear force and thereby prevent the occurrence of diagonal shear cracks, (ii) they provide confinement of the concrete section, and (iii) they prevent the buckling of the compression bars.

**6.3.1 –**

Only vertical links shall be used in beams (see Fig. 4A); inclined links shall not be used. And

a) 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).

b) 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.

**C6.3.1 –**

Vertical links should be bent into a 135° hook and extended sufficiently into the confined concrete beyond this hook to ensure that the link does not open out during strong earthquake shaking. It is proposed to provide hook extension length equal to 6 bar diameters (but not <65 mm). Larger extensions (8 diameters proposed in IS 13920 : 2016) may lead to considerable construction difficulties. Laboratory testing in the United States showed that 6 diameter extension may be adequate. As a result, ACI 318-02 standard had changed the requirement of 10 diameter extension to 6 diameter extension. ACI 318-19 standard also requires a 6 diameter hook extension for vertical links.

Cross-ties with a 90° hook are not as effective as either cross-ties with 135° hook or links in providing confinement. Construction problem arises in placing cross-ties with 135° hooks at both ends. Tests have shown that if cross-tie ends with 90° hooks are alternated, confinement will be sufficient. Therefore, it is allowed to use cross-ties with 90° hook at one end and 135° hook at other end.
6.3.2 –
The minimum diameter of a link shall be 8 mm.

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:
   \[
   V_{u,a} = V'_{u,a} - 1.4 \left( \frac{M_{u}^{As} + M_{u}^{Rh}}{L_{AB}} \right)
   \]
   ii) for sway to left:
   \[
   V_{u,b} = V'_{u,b} + 1.4 \left( \frac{M_{u}^{As} + M_{u}^{Rh}}{L_{AB}} \right)
   \]

C6.3.3 –
This clause ensures that a brittle shear failure does not precede the actual yielding of the beam in flexure. Clause 6.3.3(b) simplifies the process of calculating plastic moment capacity of a section by taking it to be 1.4 times the calculated moment capacity with usual partial safety factors. The 1.4 value is based on the consideration that plastic moment capacity of a section is usually calculated by assuming the stress in flexural reinforcement as 1.25 \( f_{y} \) (against 0.87 \( f_{y} \) in the moment capacity calculation).

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

When torsional effects are considered, the shear force capacities (maximum of 6.3.3a and b)
Proposed Modifications & Commentary IS 13920 : 2016

\[ V_{a,a} = V_{d+L}^{d+L} + 1.4 \left( \frac{M_{a}^{Ah} + M_{a}^{Rs}}{L_{AB}} \right) \]
\[ V_{a,b} = V_{d+L}^{d+L} - 1.4 \left( \frac{M_{a}^{Ah} + M_{a}^{Rs}}{L_{AB}} \right) \]

where \( M_{a}^{As} \), \( M_{a}^{Ah} \), \( M_{a}^{Rs} \) and \( M_{a}^{Rh} \) 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. \( V_{d+L}^{d+L} \) and \( V_{d+L}^{d+L} \) are the factored shears forces at ends A and B, respectively, due to vertical loads acting on the span. 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 \( V_{a,a} \) computed above. Similarly, the design shear force demand at end B shall be larger of the two values of \( V_{a,b} \) computed above.

The equations in this clause (part ii) were revised to show a correct sign in front of the multiplier 1.4 (in line with Fig. 5).
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.

Part c) is related to the shear resistance of concrete section. Note that the 2016 version of the code requires exclusion of concrete contribution in shear capacity calculation, which may be too conservative in some cases. It is proposed to neglect the concrete contribution to shear force capacity of RC beams which are subjected to higher seismic demands. When applicable, a reduced shear contribution of concrete shall be used for the design of transverse reinforcement at the beam ends (within the length of 2d from the column face). The proposed revision is in line with some international codes (e.g. ACI 318-14 Cl.18.6.5.2 and Eurocode 8, Part 1) which prescribe a partial reduction in the concrete shear contribution for ductile RC beams in some cases.

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) \( \frac{d}{4} \);

(b) \( \frac{d}{8} \times \) times the diameter of the smallest longitudinal bar; and

(c) \( 100 \) to \( 250 \) mm (see Fig. 6).

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

C6.3.4 –
Due to cyclic nature of seismic loads, shear force can change direction. The inclined links and bent up bars, effective in one direction for resisting shear force, will not be effective for opposite direction of shear force.

Part c) is related to the shear resistance of concrete section. Note that the 2016 version of the code requires exclusion of concrete contribution in shear capacity calculation, which may be too conservative in some cases. It is proposed to neglect the concrete contribution to shear force capacity of RC beams which are subjected to higher seismic demands. When applicable, a reduced shear contribution of concrete shall be used for the design of transverse reinforcement at the beam ends (within the length of 2d from the column face). The proposed revision is in line with some international codes (e.g. ACI 318-14 Cl.18.6.5.2 and Eurocode 8, Part 1) which prescribe a partial reduction in the concrete shear contribution for ductile RC beams in some cases.

C6.3.5 –
Closely spaced links at the ends of the beams which are parts of SMRF are required to achieve adequate confinement and energy dissipation capacity. However, it is proposed to increase the maximum spacing in parts b) and c) because excessively small spacing may be too restrictive in deep beams. Requirement b) is in line with IS 13920 : 1993. Requirement c) is in line with the ACI 318-19 provisions.
6.3.5.1 –
The first link shall be at a distance not exceeding 50 mm from the joint face.

6.3.5.2 –
Closely spaced links shall be provided over a length equal to 2d on either side of a section where flexural yielding may occur under earthquake effects. Over the remaining length of the beam, vertical links shall be provided at a spacing not exceeding d/2.

C6.3.5.2 –
The link spacing is specified as d/2 over the remaining length of the beam to prevent the occurrence of an unexpected shear failure in this region. IS 456 allows 3d/4 as against the requirement of d/2 in this clause. One should bear in mind that the provisions of IS 13920 are over and above those contained in IS 456.

6.3.5.3 –
Construction joint shall not be provided in the regions of beam having closely spaced transverse reinforcements.
7 – Columns and Inclined Members of Special Moment Resisting Frames

7.1 – Geometry

Requirements of this section shall apply to columns and inclined members 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. 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 7.10.

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

C7.1 –

When the factored axial load is low (less than 0.08 $f_{ck}$), the frame member will be considered as a beam and will be detailed as per clause 6. On the other hand, high axial compression level in columns may result in buckling of longitudinal reinforcement and a decrease of shear and axial capacity, hence the upper limit for axial stress of 0.40 $f_{ck}$ has been prescribed. Higher transverse reinforcement is required to hold the longitudinal bars in position and maintain desired shear and axial capacity.

International codes (e.g. ACI 318-14 and NZS 3101-2006) contain more stringent requirements for confinement reinforcement in columns with factored axial compressive stress of more than 0.25$f_{ck}$. Eurocode 8 prescribes a limit for the maximum normalised axial force to 0.65 for the medium ductility class and 0.55 for the high ductility class (note that these limits correspond to the cylinder compressive strength and that corresponding limits for cube compressive strength are 0.52 and 0.44).

Design of columns with L-, T-, X- and other irregular shapes should be performed with caution due to concerns related to torsional instability which is usually not addressed by commonly available commercial software. Chinese code JGJ 149-2017 (Technical specification for concrete structures with specially shaped columns) provides guidance regarding the analysis, design and detailing of columns with non-rectangular or non-circular shapes (including seismic design).

7.1.1 –

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

a) $20d_b$ for rectangular columns and $30d_b$ for circular columns, where $d_b$ is diameter of the largest diameter longitudinal reinforcing element bar (either single bar or individual bundled bar) in the beam passing through or anchoring into the column at the joint, and $f_{ck}$

C7.1.1 –

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

Hence, many seismic codes recommend that the dimension of a column should not be less than 20 times the diameter of largest bar in the beam running parallel to the column dimension. That is, if beam uses a 20 mm diameter bars, minimum
b) 300 mm (see Fig. 7).

dimension for column in the direction parallel with the beam should be 400 mm (see Fig. 7).

It is recommended to set the minimum diameter of 30d₀ for circular columns based on the equivalent square dimension.

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**FIG. 7 MINIMUM SIZE OF RC COLUMNS 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 10.1.3.

This clause is provided to ensure better confinement of concrete core in RC columns. The studies have shown that the confinement is better in relatively square columns than in columns with large width-to-depth ratio. However, the same clause suggests that columns with cross-sectional aspect ratio of less than 0.4 should be designed as special RC structural walls. Seismic design and detailing requirements for RC structural walls are different than RC columns and a rational distinction between these structural elements is provided in the proposed clause 10.1.3.

### 7.2 – Relative Strengths of Beams and Columns at a Joint

The provisions of 7.2 are not applicable to flat slab structures.

The intent of this clause is to reduce chances of yielding in columns that are considered as part of the lateral force-resisting system. It is intended to...
RC frames:

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 forces, 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-resisting system).

make the building fail in weak beam-strong
column mechanism, according to which beams
yield before columns. If columns are not stronger
than beams framing into a joint, there is a
likelihood of flexural yielding at both ends of all
columns in a given storey, resulting in undesirable strong beam-weak column failure
mechanism (storey mechanism) that could lead to
collapse. Therefore, a column should be stronger
than the beams meeting at a joint.

The current clause sets beam moment multiplier
of 1.4 in the summation. This value leads to high
column moments of resistance and high amount
of longitudinal reinforcement. The basis for the
1.4 value is same as for Clause 6.3.3. It is
proposed to keep the value of 1.2 which is in line
with ACI 318-14 Cl. 18.7.3.2. It is believed that
the value of 1.2 is appropriate given that a
multiplier higher than 1.0 has been first
introduced in IS 13920 : 2016. However, a 1.4
multiplier (or higher) may be recommended for
designs with higher seismic safety objectives.
Research studies have shown that the multiplier
of 1.5 or higher may be required to prevent a
formation of a soft storey mechanism (Kappos,
1997).

Current clause also stipulates that factored axial
forces arising in all load combinations should be
considered for the column moment of resistance.
It is proposed that only the factored axial load
resulting in the minimum flexural strength should
be considered for the column moment of
resistance calculation in this clause.

It is important that the check is satisfied for beam
bending moments acting in both principal
directions in the vertical plane of the frame
considered. The effect of biaxial moments should
be evaluated in the design; this can be
accomplished by considering the maximum
expected bending moments in the direction under
consideration and the corresponding bending
moment in perpendicular direction obtained from
the analysis. When the effect of biaxial moments
is disregarded in the design it is recommended to
reduce the column capacity by 30% (based on
Eurocode 8).

In T-beam construction, where slab is subjected
to tension under moments at the face of the joint,
it is recommended to consider contribution of slab
reinforcement to negative moment capacity of
beam within an effective slab width defined in IS
456 : 2000. The slab reinforcement shall be
developed at the critical section for flexure.
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.

It is proposed to waive this check for RC frames in single-storey buildings, and upper portions of multi-storey buildings (as specified in the code). This is in line with international codes and practices and is expected to facilitate more economical design solutions.

7.3 – Longitudinal Reinforcement

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 – **Splicing of Longitudinal Bars**

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 should be provided only in the 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

C7.3.2.1–

Seismic bending moments in columns are largest just above and below the floor level, and it is expected that the spalling of concrete may take place (Fig. C7), hence longitudinal reinforcement must not be spliced at those locations. Lap splices are permitted only around the column midheight where bending moments are the smallest. The current parts c 1) and 2) of the clause are proposed to be removed since part c) stipulates that the splices shall be provided within the column midheight. In part c) it is proposed to permit lap splices at the column base, but an increased lap splice length should be used in such cases.

The restriction on percentage of lapping bars at one location means that only half the bars can be spliced at one storey and the other half at the next storey. However, when this is not possible due to
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 $L_d$ 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.

construction challenges, it is proposed to allow all bars to be lapped at the same location but with increased lap length of 1.3$L_d$, where $L_d$ is the development length in tension as per IS 456 : 2000.

<table>
<thead>
<tr>
<th>7.3.2.2 – Mechanical Couplers</th>
</tr>
</thead>
<tbody>
<tr>
<td>Only mechanical couplers conforming to IS 16172 shall be used. Further,</td>
</tr>
<tr>
<td>(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.</td>
</tr>
<tr>
<td>(b) the spacing between adjacent longitudinal bars shall take into account the outer size of the coupler</td>
</tr>
</tbody>
</table>

Fig. C7 – Region for lap splices

C7.3.2.2-
Alternate coupling of bars shall not be mandatory, when requirements of 7.3.2.2(b) have been met.
ACI 318-14 commentary states that staggering is encouraged and may be necessary i) for constructability, ii) for providing enough space around the splice for installation, and/or iii) for meeting the clear spacing requirements.
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.

C7.3.2.3-
It is proposed to avoid welded splices in columns of Special moment resisting frames. 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.

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, 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, it shall have at least the minimum longitudinal and transverse reinforcement given in IS 456.

C7.3.3 –
Even when column extensions are considered as non-structural, they contribute to its stiffness. If the extensions are not properly tied with the column core, a severe shaking may cause spalling of this portion leading to a sudden change in the stiffness of the column. Therefore, the code requires that such extensions be detailed at least as per IS 456 requirements for columns.
<table>
<thead>
<tr>
<th>7.4 - Transverse Reinforcement</th>
<th>C7.4 –</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse reinforcement has the following three purposes: (a) provides shear resistance to the member, (b) confines the concrete core and thereby increases the ultimate strain of concrete which improves ductility, and (c) provides lateral resistance against buckling to the compression reinforcement.</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>7.4.1 -</th>
<th>C7.4.1 –</th>
</tr>
</thead>
</table>
| 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. 10A). |
| See commentary of clause 6.3.1. |
When rectangular links are used,

a) the minimum diameter permitted of transverse reinforcement bars is shall be 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;

C7.4.2 –
Part d) of IS 456 allows the link spacing to be equal to the least lateral dimension of the column while this clause restricts it to half the least lateral dimension. Closer spacing of links is desirable to ensure better seismic performance.
b) the maximum spacing of parallel legs longitudinal bars supported by corners of a rectangular link and/or cross-tie 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 \((L_c'\text{ or } B_c')\) exceeds 300 mm (see Fig. 10B); the cross-tie shall be placed perpendicular to the 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
\[
V_u = 1.4 \left[ \frac{M_u^{Ah} + M_u^{Bs}}{h_{st}} \right]
\]

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

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).

\( M_u^{Ah} \), \( M_u^{Bs} \) and \( M_u^{Bh} \) 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).

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

in line with international codes (e.g. ACI 318-14 Cl.18.7.6.1.1).

Note that the shear capacity of a column just above the foundation (where there are no framing beams) should be based on moment capacity (flexural overstrength) of column.
Proposed Modifications & Commentary IS 13920 : 2016

(A)

SWAY TO RIGHT

1.4M_u^A

1.4M_u^B

V_u

SWAY TO LEFT

1.4M_u^A

1.4M_u^B

V_u

h_{pl}
7.5.1 –

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

C7.5.1-

Contribution of concrete in the calculation of design shear force capacity of an RC column may not be considered when both (i) & (ii) occur:

(i) The earthquake-induced shear force calculated in accordance with 7.5 represents at least one-half of the maximum required shear strength from the analysis, and

(ii) The factored axial compressive stress, including earthquake effects, is less than 0.04$f_{ck}$.

Contribution of concrete in resisting shear should be considered outside the confined zone.

For rectangular or square columns, longitudinal reinforcement within 1/3rd of overall member depth from extreme tension face of section should be considered as tension reinforcement for calculation of concrete shear capacity as per IS 456: 2000.

For circular columns, 1/3rd of total longitudinal reinforcement should be considered as tension reinforcement for the calculation of concrete shear capacity as per IS 456: 2000. Equivalent section width should be taken equal to the column diameter, and the effective depth should be 0.8 times the column diameter.

IS 456: 2000 specifies a factor ($\delta$) in clause 40.2.2 to enhance concrete shear capacity in column to account for axial compression, but no such factor is provided to account for axial tension.
7.6 – Special Confining Reinforcement

The requirements of this section shall be applicable to 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.

C7.6–

Columns may be subjected to large inelastic deformations and experience flexural yielding during strong ground shaking. Hence, special confining reinforcement is provided to ensure adequate ductility and provide restraint against buckling to the compression reinforcement in columns. Clause 7.6 should apply to columns only, because provisions related to confining reinforcement for beams are included in Clause 6.3.5. This is in line with international codes (e.g. ACI 318-14).

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 special confining reinforcement shall (see Fig. 12).

a) be provided over a length \( l_0 \) from the face of the joint, towards mid-span, of beams and mid-heights of columns, on either side of the joint where \( l_0 \) shall not be less than the greatest of:

1) larger cross-sectional lateral dimension of the column member at the joint face or at the section where yielding is likely to occur,
2) 1/6 of clear span of the member column, or
3) 450 mm.

b) have a spacing not more than

1) \( \frac{1}{4} \) 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:

\[ A_{sh} \geq \frac{1}{0.05} \left( \frac{A_g}{A_k} - 1 \right) \]

where \( A_g \) is the gross area of the column and \( A_k \) is the net area of the column.

\[ h = \frac{A_{sh}}{A_{sh}} \times h \]

Term \( h \) in the \( A_{sh} \) equation for rectangular links has been clarified and Figure 10 has been revised to address the modification.

It should be noted that some international codes (e.g. ACI 318-14) contain more stringent
\[ A_{sh} = \text{Maximum of} \left[ \begin{array}{l}
0.09s_v D_k \frac{f_{ck}}{f_y} \left( A_k - 1 \right) \\
0.024s_v D_k \frac{f_{ck}}{f_y}
\end{array} \right] \]

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

\[ = \frac{\pi}{4} D_k^2 \]

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

\[ A_{sh} = \text{Maximum of} \left[ \begin{array}{l}
0.18s_v h \frac{f_{ck}}{f_y} \left( A_k - 1 \right) \\
0.05s_v h \frac{f_{ck}}{f_y}
\end{array} \right] \]

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. 10BC), 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. 10BC). In such cases, \( A_k \) should be reduced to the area of the new confined concrete core.

provisions for columns with higher axial stresses. For example, there is a more stringent requirement regarding the confinement reinforcement for columns with factored axial compressive stress higher than 0.25\( f_{ck} \) (corresponding to cylinder strength 0.3\( f_{ck} \)). High strength reinforcement (up to Fe700) can be used for calculation of confinement reinforcement as per clause 7.6.1. However, yield strength should be restricted to 415 MPa for the calculation of transverse reinforcement as required by IS 456 : 2000 for meeting the shear capacity requirements as per 7.5.
shall be measured as overall core area, regardless of link arrangement. Hooks of cross-ties shall engage peripheral longitudinal bars.

7.6.2 –
When a column terminates into a footing or mat, special confining reinforcement shall extend at least 300 mm into the footing or mat (see Fig. 13).

C7.6.2 –
During severe shaking, a plastic hinge may form at the bottom of a column that terminates into a footing or mat. Hence, special confining reinforcement of the column must be extended to at least 300 mm into the foundation.
<table>
<thead>
<tr>
<th>Section</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.6.3</td>
<td>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 $2l_0$ (instead of height $l_0$) 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.</td>
</tr>
<tr>
<td>C7.6.3</td>
<td>The point of contra-flexure is usually in the middle half of the column, except for columns in the top and bottom storeys of a multi-storey frame. When the point of contra-flexure is not within the middle half of the column, the zone of inelastic deformation may extend beyond the region that is provided with closely spaced link reinforcement. This clause requires the provision of special confining reinforcement over the critical portion of the column in such situations.</td>
</tr>
<tr>
<td>7.6.4</td>
<td>Special confining reinforcement shall be provided over the full height of a column which has significant variation in stiffness along its height. This variation in stiffness may result due to abrupt changes in cross-section, size, or unintended restraint to the column provided by stair-slab, mezzanine floor, plinth or lintel beams framing into the columns, RC wall or masonry wall adjoining column and extending only for partial column height.</td>
</tr>
<tr>
<td>C7.6.4</td>
<td>Column stiffness is inversely proportional to the cube of column height. Hence, columns with significantly less height than other columns in the same storey have much higher lateral stiffness, and consequently attract much greater seismic shear force. There is a possibility of a brittle shear failure occurring in the unsupported zones of such short columns. This has been observed in several earthquakes in the past. A mezzanine floor or a loft also results in the stiffening of some of the columns while leaving other columns of the same storey unbraced over their full height. Another example is of semi-basements where ventilators are provided between the soffit of beams and the top of the wall; here, the outer columns become the “short-columns” as compared to the interior columns. Hence, special confining reinforcement shall be provided over the full height in such columns to ensure adequate confinement and shear strength.</td>
</tr>
</tbody>
</table>

**FIG. 13 – PROVISION OF SPECIAL CONFINING REINFORCEMENT IN FOOTING**

(transverse reinforcement spacing was revised)
7.6.5 –
Columns supporting reactions from discontinued stiff members, such as walls, shall be provided with special confining reinforcement over their full height (see Fig. 14). This reinforcement shall also be placed above the discontinuity for at least the development length of the largest longitudinal bar in the column. Where the column is supported on a wall, this reinforcement shall be provided over the full height of the column; it shall also be provided below the discontinuity for the same development length.

C7.6.5 –
Observations in past earthquakes indicate very poor performance of buildings where a wall in the upper storeys terminates on columns in the lower storeys. Hence, special confining reinforcement must be provided over full height in such columns. This provision is related to both columns and structural walls. It is proposed to keep this clause unchanged but revise 10.1.10.
98 – Beam-Column Joints of Special Moment Resisting Frames

98.1 – Design of Beam-Column Joints for Distortional Shear

C8.1 –
Main considerations related to seismic design of beam-column joints are:

a) Serviceability: cracking due to diagonal compression and joint shear should be prevented;

b) Strength: joints should be stronger than the adjacent beams and columns;

c) Ductility of RC frames under seismic conditions is possible only when joints are sufficiently strong;

d) Anchorage: proper anchorage should be provided for the longitudinal bars of the beams;

e) Ease of construction: joints should not be congested with reinforcement.

Transverse reinforcement in beam-column joints is often not provided due to construction challenges. Similarly, in traditional construction practice bottom beam bars are often not continuous through the joints. These practices are not acceptable for SMRF construction.

98.1.1 – Shear Strength of Concrete in a Joint

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

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

(Aej was removed from the above equations)

Shear strength of joint is equal to the product of $\tau_{jc}$ and $A_{eq}$ where

$A_{eq}$ 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:

C8.1.1 – Shear Strength

The concept and specified values of nominal shear strength are in line with ACI 318-14 provisions. The nominal shear strength value takes into account the shear carried by the concrete as well as the joint (shear) reinforcement. For the sake of information, refer to a design procedure prescribed by Eurocode 8 (EN 1998-1:2005) contained in Clause 5.5.3.3, which distinguishes failure mechanisms for the joints and provides provisions for the corresponding confinement reinforcement.

The current revision contains minor clarifications and a new Fig. 15, which illustrates an isometric view of the joint to explain the joint dimensions while considering the direction of shear forces. Joint width equations were updated to comply with ACI 318-14 Cl. 18.8.4.3.
\[ b_j = \min[b_b, b_c + 0.5 h_c] \text{ if } b_c < b_b \]
\[ b_j = b_c \text{ if } b_c < b_b \]

and

\[ b_j = \min[b_b + 2x, b_c + h_c] \text{ if } b_c \geq b_b \]

where

- \( b_b \) = width of beam
- \( b_c \) = 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:

\[ w_j = h_c \text{ but not less than half the depth of beam framing into joint.} \]

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 \( 3/4 \)th 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.

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

\[
\tau_{j\xi} = \frac{V_{j\xi}}{b_jw_j} \quad \text{for shaking along plan direction } \xi \text{ of earthquake shaking, and}
\]

\[
\tau_{j\eta} = \frac{V_{j\eta}}{b_jw_j} \quad \text{for shaking along plan direction } \eta \text{ of earthquake shaking.}
\]

It shall be ensured that the joint shear capacity of joint concrete estimated using 9.8.1.1 exceeds both \( \tau_{j\xi} \) and \( \tau_{j\eta} \).

b) Design shear force demands \( V_{j\xi} \) and \( V_{j\eta} \) acting horizontally on the joint in principal plan directions \( \xi \) and \( \eta \) 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).

### C8.1.2 –

Seismic shear force in the joint can be calculated as shown in Fig. C8 for rectangular beam section. Refer to ACI 352-2002 for more information, including the seismic shear force calculation for a flanged beam section.

---

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

where,

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

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

**Fig. C8** – Evaluation of horizontal joint shear for rectangular beam section (Note: $T =$ tension force; $C =$ compression force; $V =$ shear force; subscript $b$ for beam; subscript $c$ for column; and subscript $s$ for slab - adapted from ACI 352-1989).

### 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.

It is proposed to remove this clause – it is a repetition of Clause 7.1.1.

### 98.2 - Transverse Joint Reinforcement

#### 98.2.1 - *Confining Reinforcement in Joints*

a) When all four vertical faces of the joint have one or more 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

C8.2.1 –

A joint can be confined by the beams/slabs framing into the joint, longitudinal bars (from beams and columns, passing though the joint), and transverse reinforcement. Transverse reinforcement can be reduced if structural members frame into all four sides of the joints.
<table>
<thead>
<tr>
<th>Proposed Modifications &amp; Commentary IS 13920 : 2016</th>
</tr>
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<tbody>
<tr>
<td>beam framing into it; and</td>
</tr>
<tr>
<td>b) Spacing of these transverse links shall not exceed 150 mm.</td>
</tr>
<tr>
<td>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,</td>
</tr>
<tr>
<td>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</td>
</tr>
<tr>
<td>2) spacing of these transverse links shall not exceed 150 mm.</td>
</tr>
</tbody>
</table>

| 98.2.2 – |
| In the exterior and corner joints, all 135° hooks of cross-ties should be **provided** along the outer face of columns. |

| C8.2.2 – |
| 135° hook in a cross-tie is more effective than a 90° hook to confine core concrete. As the interior face of the exterior beam-column joint is confined by beams it is preferable to place the cross-ties such that all the 90° hooks are on the inner side and 135° hooks at the exterior side of the joint. |

| 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. |

|  |
C9 – Intermediate Moment Resisting Frames (IMRF)

Implementation of ductile detailing provisions requires substantially higher effort in design, construction and quality control. IRMFs have somewhat lower ductility requirements and are assigned a lower R value (corresponding to higher seismic design force) than Special moment resisting frames (SMRFs).

C9.1.1 –

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

9 – Intermediate Moment Resisting Frames

9.1– Beams

9.1.1–

The shear force to be resisted by the vertical links 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 \( M_u^{As} \) and \( M_u^{Ah} \) 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

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.
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.

**C9.1.2**
Positive steel requirement at joint face of IMRFs is the same as for SMRFs (see 6.2.3).

### 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.

**C9.1.3**
This requirement for IMRFs is the same as for SMRFs (see 6.2.4).

### 9.1.4

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

- (a) \( \frac{d}{4} \),
- (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 shall have vertical links at a spacing not exceeding \( \frac{d}{2} \).

**C9.1.4**
Requirement of spacing of links is same as that for SMRF (see 6.3.5) to ensure adequate confinement of the concrete core.

### 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:

**C9.2.1**
This is similar to 7.5, except that the multiplier 1.4 in the equation is absent to reduce conservatism in column shear design.
2) For sway to left:

\[
V_u = \left(\frac{M_u^{As} + M_u^{Bl}}{h_u}\right) \geq \left(\frac{M_{u1C}^{+} + M_{u2C}^{-}}{h_u}\right)
\]

where \(M_u^{As}\) and \(M_u^{Bl}\) 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). \(M_{u1C}^{+}\) 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 – General Requirements

#### 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-resistance RC building.

#### C10.1.1 –

Structural wall is a single wall or an assembly of interconnected walls considered to be a part of the lateral force resisting system of a building. Structural walls support i) vertical loads, ii) moments about axis perpendicular to the plane of the wall, and iii) shear forces parallel to the plane of the wall. Structural walls need to be continuous from the foundation to the roof level. Their behaviour under seismic loading will depend on the overall height-to-length \((h_w/L_w)\) ratio. Walls in medium- to high-rise buildings usually have \(h_w/L_w \geq 2.0\) and their seismic behaviour is governed by bending. Special structural walls are expected to perform in a ductile manner and dissipate energy during an earthquake; this can be achieved by careful design and detailing. Distributed vertical reinforcement contributes to flexural resistance due to the combined axial load and bending. Boundary elements at the wall ends are provided to enhance flexural resistance when a wall is subjected to high flexural compression stresses. These boundary elements need to have sufficient length over which confinement is provided. Plastic hinge region of the wall, usually located at its base, is exposed to significant seismic deformations and requires special detailing and confinement of reinforcement.

Horizontal reinforcement contributes to the wall’s shear resistance - similar to stirrups in RC beams. A ductile structural wall needs to have high shear resistance to prevent the occurrence of shear failure.

Structural walls are stiff structural elements, hence lateral displacements in these walls are less than in similar RC frame structures.

#### 10.1.2 –

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

a) \(\text{larger of } 150 \text{ mm and } h_w/20\) for solid structural walls (without significant

#### C10.1.2 –

The minimum thickness requirement for special RC structural walls depends on the slenderness at the storey level (expressed as a fraction of the clear storey height \(h_o\)). Experimental studies
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 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.

C10.1.3 –

Wall length/thickness ratio of 4.0 is excessively low, hence designers tend to treat columns with elongated sections as special structural walls. It is recognized that some designs may require walls with length/thickness ratios less than 6.0. It is recommended to treat these walls as RC frame members. Some international codes have special provisions for these walls (e.g. ACI 318-14 Cl.18.10.8.1).

10.1.4 –

Special structural shear walls shall be classified as squat, intermediate or slender (also known as S-slim) 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 (or slender) walls: h_w/L_w > 2.

C10.1.4 –

Most international codes do not make a distinction between squat and intermediate walls (e.g. ACI 318-14, CSA A23.3-14), however Eurocode 8 and New Zealand code NZS 3101:2006 make a distinction between flexural (or slender) walls and squat walls. For design purposes it can be assumed that structural walls with h_w/L_w ≤ 2 behave like squat walls, while the walls with higher h_w/L_w ratio are called “flexural” since they typically demonstrate a flexure-dominant behaviour. Note that the term “slender” was replaced by “flexural”, which is more common in technical literature (alternatively the term “tall walls” could be used).

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

C10.1.5 –

The effective width of a flanged wall section is shown in Fig. C9. This flange width criterion is similar to flange width criteria for T beam.
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).

C10.1.6 –
Vertical distributed reinforcement in the wall contributes to its flexural resistance while horizontal reinforcement contributes to its shear resistance.

Table 1
Minimum Reinforcement in RC Shear Walls
(Clause 10.1.6)

<table>
<thead>
<tr>
<th>Sl. NO.</th>
<th>Type of wall</th>
<th>Reinforcement Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Squat walls</td>
<td>$(\rho_h)_\text{min} = 0.0025$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$(\rho_v)_\text{min} = 0.0025 + 0.5 \left(1 - \frac{h_w}{L_w}\right)(\rho_h - 0.0025)$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$(\rho_v,\text{net}) = (\rho_v,\text{web}) + \frac{t_w}{L_w} \left[0.02 - 2.5(\rho_v,\text{web})\right]$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$(\rho_v)<em>{\text{provided}} &lt; (\rho_h)</em>{\text{provided}}$</td>
</tr>
</tbody>
</table>
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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 \( \frac{f_d}{25} \) MPa,

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.

C10.1.7 –
It is expected that two curtains of reinforcement will be needed for most wall designs. The use of two curtains of reinforcement will reduce fragmentation and premature deterioration of the concrete under inelastic cyclic deformations.

It is important to consider \( h_w/L_w \) ratio as a criterion for providing two curtains of reinforcement in the wall (ACI 318-14 Cl. 18.10.2.2). It is difficult to provide two curtains of reinforcement within thinner walls so 200 mm minimum thickness has been prescribed.

The last sentence has been removed since it is related to the detailing of reinforcement which is addressed in clause 10.8.1.

10.1.8 –
The largest diameter of longitudinal steel bars used in any part of a wall shall not exceed \( \frac{1}{10} \)th 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.

C10.1.8 –
The intention of this clause is to prevent the use of very large diameter bars in thin wall sections. It is also not appropriate to use excessively small bar sizes for wall reinforcement. There is a high chance of eccentric bar placement when relatively small bar sizes are used (this particularly applies to vertical bars); eccentrically placed vertical bars may cause additional bending moments which are not accounted for by the design.

10.1.9 –
The maximum spacing of vertical or

C10.1.9 –
Part a) of this clause ensures that a sufficient
horizontal distributed wall reinforcement shall not exceed the smaller of,
a) 1/5th horizontal length \(L_w\) of wall, and
b) 3 times thickness \(t_w\) of web of wall; and
bc) 450 mm.

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

| 10.1.10 – | Special structural shear walls 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 \(L_w\) is the overall length of the longest shear wall or coupled wall in the direction under consideration, and \(h_w\) is the overall wall height. \(M_u\) and \(V_u\) are design moment and shear for a load combination including seismic load having largest \(M_u/4V_u\) ratio. |
| 10.1.11 – Plastic hinge region\(^3\) | It is proposed to revise the length of plastic hinge region within which stringent detailing of reinforcement in special structural walls is required. Currently clause 10.8.2 contains provision related to plastic hinge length for structural walls with \(h_w/L_w > 2\). There is no specific mention of the term “plastic hinge region”, but it is referred to as “region where flexural yielding may take place”. The proposed provision for the plastic hinge region length contained in this clause is based on the recent research done in North America and has been based on Canadian code CSA A23.3-14 and the US code ACI 318-14. A discussion on different proposals on plastic hinge length is provided in commentary to CSA A23.3-14 (Cl.21.5.2.1). |

\(^3\) This is a new clause.

number of bars in short walls. Original part b) was removed as redundant since the minimum wall thickness is set to 150 mm, and instead the spacing limit is set to 450 mm, which is in line with the ACI 318-14 Cl. 18.10.2.1 (18 inch max spacing). Note that IS 456 Cl.32.5 also sets maximum spacing limit of 450 mm for horizontal and vertical wall reinforcement for non-seismic design applications.

It is recommended to provide more closely spaced reinforcement within the plastic hinge region. CSA A23.3-14 Cl.21.5.5.2 specifies the maximum spacing of 300 mm for horizontal reinforcement within plastic hinge region. Note that some international codes (e.g. Eurocode 8) prescribe 250 mm maximum spacing for both horizontal and vertical distributed reinforcement.
10.2 – Design for Shear Force

10.2.1 –
Nominal shear stress demand $\tau_v$ in a wall shall be estimated as:

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

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.

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

a) $\alpha = 1.0$ when $h_w / L_w < 2.0$,

b) $\alpha = 1.4$ when $h_w / L_w \geq 2.0$ but $h_w \leq 60$ m, and

c) $\alpha = 2.1$ when $h_w > 60$ m.

C10.2.1 –
Shear strength provisions for structural walls are very similar to those for beams. The vertical reinforcement that is provided in the wall shall be considered for calculation of the design shear stress as per Table 19 of IS 456: 2000. The increase in shear strength due to axial compression may also be considered as per clause 40.2.2 of IS 456: 2000. However, for this, only 80% of the factored axial compressive force should be considered as effective. This is to consider possible effect of vertical acceleration.

Effective depth $d_w$ is taken as 0.8 times the actual wall length $L_w$.

In the current clause the factored shear force $V_u$ assumes the value determined from seismic analysis. Since the design objective for flexural special structural walls is to ensure ductile flexural behaviour, it is important to prevent a brittle shear failure. When a structural wall has excessive flexural resistance due to higher material strengths than assumed in the design, a brittle shear failure might take place unless the wall is designed for a higher than required shear strength; this is known as flexural overstrength. The factor $\alpha$ takes into account flexural overstrength and the effect of higher vibration modes in buildings taller than 60 m. Similar provisions related to the design shear force for special structural walls have been in place in international codes for several years (e.g. ACI 318-14 and CSA A23.3-14).

In buildings with underground parking levels, structural walls extend below the ground floor level down to the foundations and are often connected by multiple floor diaphragms to other walls, e.g. perimeter foundation walls. This results in an indeterminate system for resisting an overturning moment below the plastic hinge region. As a result, large reverse shear forces may develop, and these forces may exceed the design shear force at the base of the ground floor level. In those cases the factored shear force and corresponding factored bending moment shall be determined from an analysis that considers appropriate effective stiffnesses of all members at the underground level.

This phenomenon has been addressed by the
10.2.2 – Design shear strength of concrete

Design shear strength of concrete $\tau_c$ 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 $\tau_c$ calculation, unless depth of neutral axis is calculated by strain compatibility.

C10.2.2 –

Code suggests 0.8 $L_w$ as effective wall depth, hence approximately a 0.4 $L_w$ long portion of the wall can be taken as tension zone, and reinforcement within that zone may be considered to calculate $\tau_c$.

When a wall is in compression, then concrete shear capacity can be enhanced by factor ($\delta$) as per IS 456:2000 clause 40.2.2.

It is recommended to apply a reduction factor for concrete shear capacity when the wall is in tension. Refer to IS 16700:2017 for guidance. ACI 318-14 and NZS 3101:2006 also contain provisions regarding the reduced concrete shear capacity based on the level of axial tension.

10.2.3 – Design of horizontal shear reinforcement

When nominal shear stress demand $\tau_s$ 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 $\tau_{c,min}$ design distributed horizontal shear reinforcement shall be provided of area $A_h$ given by shall be determined as follows:

$$A_h = \frac{V_u}{0.87 f_y \left( \frac{d}{s_y} \right) 0.87 f_y \left( \frac{d}{s_y} \right)}$$

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 $\tau_c$ of concrete, horizontal shear reinforcement shall be the minimum area of horizontal steel reinforcement shall be provided as.
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 Limit States 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.

10.3.3 –
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.

C10.3.1 –
The equations in Annex A are derived assuming a rectangular wall section of length $l_w$ and thickness $t_w$ that is subjected to combined uniaxial bending and axial compression. The vertical reinforcement is represented by an equivalent steel plate along the length of the section. The stress-strain curve assumed for concrete is as per IS 456 : 2000 whereas that for steel is assumed to be bi-linear. Two equations are given for calculating the flexural strength of the section. Their use depends on whether the section fails in flexural tension or in flexural compression. Complete derivation of these equations is available in Medhekar and Jain (1993).

The properties of the wall cross-section that affect the bending resistance of the wall, including concrete geometry, concrete strength and the reinforcing steel, should be maintained over the plastic hinge region. The design moment of resistance of the wall will reduce over the height of the plastic hinge due to the reduction in axial compression force from gravity loads, which is not a property of the cross-section.

The factored bending moment in the wall should be assumed to vary linearly from the design moment of resistance at the top of the plastic hinge region to zero at a point at the top of the wall.

C10.3.2 –
This provision is not required, provided that other requirements of this section have been followed.
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

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, \( \theta_{ic} \), is greater than its inelastic rotational demand, \( \theta_{id} \), that is,

\[ \theta_{ic} > \theta_{id} \]

The inelastic rotational demand at the base of a wall, \( \theta_{id} \), shall be taken as

\[ \theta_{id} = \frac{(\Delta_u R - \Delta_u \gamma_w)}{(h_w - L_w/2)} \geq 0.003 \]

where

\[ \Delta_u = \text{elastic lateral displacement due to factored seismic force}, \]
\[ \Delta_u \times R = \text{design displacement}, \]
\[ \gamma_w = 1.4 \text{ flexural overstrength factor}, \]
\[ \Delta_u \gamma_w = \text{elastic portion of the lateral displacement}, \]
\[ L_w = \text{the length of the longest wall (within the hinge region) in the direction considered}. \]

The inelastic rotational capacity of a wall, \( \theta_{ic} \), shall be taken as

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

where \( \varepsilon_{cu} = 0.0035 \text{ maximum concrete compression strain} \)

C10.3.3 – Ductility check

The purpose of this clause is to ensure that special structural walls have sufficient displacement

---

4 This is a new clause
capacity, which depends on the compression strain capacity of concrete and the tension strain capacity of reinforcement. This provision has been included (implicitly or explicitly) in several international codes. Canadian code CSA A23.3-14 (Clause 21.5.7) and the explanatory notes contain a detailed discussion related to the ductility check (see also Adebar, Mutrie and DeVall, 2005).

Special RC structural walls must have adequate ductility to tolerate yielding of vertical reinforcement due to combined effect of axial load and bending at any point over the wall height. For that reason, the depth of compression zone should be limited, hence the upper limit was set to \( \frac{x}{L_w} \leq 0.4 \). Smaller values of \( \frac{x}{L_w} \) ratio are associated with larger inelastic deformations and can be as low as 0.15 or less for ductile walls.

Ductility check is intended to evaluate whether a wall has adequate displacement capacity (in line with the design R value). The distance from the extreme compression fibre to the neutral axis, \( x \), shall be determined by plane sections analysis for the factored axial load acting on the wall and a bending moment causing the maximum compression strain at the extreme compression fibre (0.0035), according to 10.3.1.

Inelastic rotational demand at the base of a wall, \( \theta_{id} \), is obtained as a rotation of the plastic hinge at the base of the wall due to inelastic displacement at the top of the wall. Inelastic displacement is determined by deducting elastic displacement \( \Delta u \times \gamma_w \) from the total design displacement \( \Delta u \times R \). The latter displacement is obtained from the elastic analysis by multiplying elastic displacement by the “R” value. The height over which the rotation takes place is the distance from the centroid of plastic hinge region to the top of the wall (see Fig. C10a).

Inelastic rotational capacity of the wall section, \( \theta_{ic} \), is a characteristic of the section with given geometric properties, materials, and reinforcement. It can be calculated as a difference between the ultimate curvature and yield curvature multiplied by the plastic hinge height \( L_w/2 \) (note that this value is used only for ductility check calculation purposes), as follows

\[
\theta_{ic} = (\varphi_u - \varphi_y) \left( \frac{L_w}{2} \right)
\]

where

\[
\varphi_u = \frac{\varepsilon_{cu}}{0.002}
\]

\[
\varphi_y = \frac{x}{L_w/2}
\]

depth (see Fig. C10b). Note that the yield curvature \( \varphi_y \) assumes equal strains (0.002) in concrete in steel, hence
10.4 – Boundary Elements

Boundary elements are portions along the wall edges that are strengthened by

C10.4.1 –

Boundary elements at the wall ends are subjected to high flexural compression stresses and need to

5 This clause was significantly revised and expanded
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 fibre 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 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 0.2$L_w$ (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 $h_{gt}/10$ but not less than the wall thickness ($t_w$).

ensure adequate confinement of vertical reinforcement within highly stressed wall’s compression zones. Longitudinal reinforcement in boundary elements must be confined by transverse links, similar to RC columns in SMRFs. Dimensional limits for boundary elements are presented in Figure C11.

As an alternative to the value prescribed by 10.4.1a), the length of a boundary element ($b_w$) can be determined from a rational procedure provided in international codes (e.g. ACI 318-14 Cl.18.10.6.2). The length of a boundary element can be determined as the greater of

a) $x/2$, where $x$ is neutral axis depth as determined from 10.3.1 for the minimum factored axial force and the corresponding moment of resistance, and

b) $x-0.1L_w$. 
### 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 a combined effect of factored gravity axial force and 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.

**10.4.2.12 –**

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 a boundary element 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 6 percent. It is recommended to limit the reinforcement ratio to not 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.

**C10.4.2.1 –**

A boundary element should be effective in resisting the effect of gravity axial load and earthquake-induced bending moments along with the distributed vertical reinforcement in the wall web. The original (1993) version of IS 13920 contained a procedure for determining capacity of a boundary element by treating it as a short column subjected to uniaxial tension/compression. Most design codes and technical literature treat boundary elements as integral part of a structural wall. The moment of resistance can be estimated for the wall section, which includes the effect distributed web reinforcement and additional longitudinal reinforcement in boundary elements. It is proposed to follow a similar approach in IS 13920. As a result, equations in Annex A (which were originally developed for walls with uniformly distributed vertical reinforcement) have been expanded to enable design of walls with boundary elements (see Section A-2).

**C10.4.2.2 –**

Moderate axial compression results in higher moment capacity of the wall. Hence, beneficial effect of axial compression due to gravity loading should not be fully relied upon in design due to the possible reduction in its magnitude by vertical acceleration.

**C10.4.3 –**

There is a need to set the maximum limit for the amount of reinforcement in the boundary elements in order to avoid the congestion. This is in line with the international code provisions (e.g. CSA A23.3-14 Cl.21.5.4.3).
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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:

$$A_{sh} = 0.05 \frac{s_{v} f_{y}}{f_{ck}}$$

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) $1/2 \times 1/3$ 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 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.

C10.4.4–

The provisions for confining reinforcement in boundary elements are similar to those pertaining to special confining reinforcement in RC columns of special moment resisting frames. This is in line with the international codes, e.g. ACI 318-14 Cl.18.10.6.4. Detailing of confinement reinforcement is provided similar to RC columns (Clause 7.4).

Reinforcement detailing in boundary elements is summarized in Figure C12. It is recommended to extend the confining reinforcement to the bottom of foundation.

Fig. C12 – Reinforcement detailing for boundary elements in special structural walls (new drawing)

10.4.5 –

Boundary elements need not be provided, if the entire wall section is provided with special confining reinforcement, as per 7.6.
10.5 – Coupled Beams
Shear Structural Walls

10.5.1 –

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

C10.5 – Coupled Structural Walls

C10.5.1 –

Coupled structural walls subjected to lateral loading develop complex internal forces in the coupling beams and the connected walls. When the walls deflect under lateral loads the coupling beams bend in double curvature. The bending moments at the beam ends induce shear forces in the beams, which in turn induce bending moments and axial forces (tension/compression) in the walls (Fig. C13). When the coupling beams are relatively rigid, the coupling beams and the connected walls act as a framed system (model R). Alternatively, when the coupling beams are flexible the walls act as independent cantilevers; this can be considered as a partially coupled wall (model F). These two models are illustrated in Fig. C14.

The total overturning moment at the base of the wall (\(M_1+M_2+T\times\alpha\)) comprises the bending moments in each wall (\(M_1\) and \(M_2\)) and the force couple consisting of axial forces “T” at lever arm “\(\alpha\)”. The resulting moment (product of “T” and “\(\alpha\)” is zero in case of Model F (flexible beams) and reaches maximum when the coupling beams are infinitely rigid. The coupling beams are effective in reducing the magnitudes of the moments in the two walls. Because of relatively large lever arm \(\alpha\), a relatively small axial stress could induce a large moment of resistance for a coupled wall, which is considered as an advantage of this structural system.

An appropriate numerical model (F or R) can be identified by calculating the ratio of the bending moment resisted by the axial forces (\(T\times\alpha\)) and the total overturning moment (\(M_1+M_2+T\times\alpha\)). When the ratio \(\frac{T\alpha}{M_1+M_2+T\times\alpha}\) is greater than 2/3 the wall can be idealized as Model R, otherwise Model F can be used.

Fig. C13– A coupled structural wall under lateral loading: internal forces (Stafford Smith and Coul, 1991) (new drawing).
A coupled structural wall shall be designed for the effects of combined shear force, axial load, and bending moments in the connected walls. In case of Model R, an equivalent frame model can be used to analyse the coupled wall and obtain realistic internal forces in the walls and coupling beams (Fig. C15). The wall is represented by an equivalent wide column aligned along the centroidal axis. The “plane sections” assumption may be incorporated through stiff arms located at the connecting beam levels, which span between the wide column centerline and the external fibers. In model F the connecting flexible beams may be represented as line elements with the corresponding flexural, axial, and shear stiffness properties (Stafford Smith and Coul, 1991). Coupling beams need to be designed for the bending moments and shear forces obtained from the analysis.

---

**Fig. C14 – Coupled structural walls: a) rigid coupling beams (model R) and b) flexible coupling beams (model F). (Source: CSA A23.3-04 Explanatory notes) (new drawing)**

<table>
<thead>
<tr>
<th>a) Model R</th>
<th>b) Model F</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1.png" alt="Model R Diagram" /></td>
<td><img src="image2.png" alt="Model F Diagram" /></td>
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<thead>
<tr>
<th>LW</th>
<th>LC</th>
<th>M1</th>
<th>M2</th>
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<tr>
<td><img src="image3.png" alt="Diagram" /></td>
<td><img src="image4.png" alt="Diagram" /></td>
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</table>
Axial forces in the walls adjacent to the coupling beams are determined considering the effect of gravity loading and shear forces at the coupling beam ends (see Fig. C16). The coupling beams will cause either tension or compression in the adjacent walls. The following load combinations may be used to determine axial forces in the walls:

a) Coupling beam end shears cause tension in the wall: $1.2\sum V_i - 0.8P_{DL}$

b) Coupling beam end shears cause compression in the wall: $1.2\sum V_i + 1.2P_{DL} + 1.2P_{IL}$
The above discussion is related to the elastic analysis of coupled structural walls. Capacity Design approach can be applied to determine the ultimate capacity of a coupled wall by considering different failure mechanisms, e.g. plastic hinging at the beam ends and/or the base of the wall. This approach has been explained through examples by SEAOC (1999).

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 $\tau_{ve}$ in coupling beam which exceeds the

C10.5.2.1 –

The coupling beams connecting structural walls may be subjected to high shear stresses induced by earthquake effects. It is desirable that these beams act as fuses and dissipate earthquake energy. In many cases, due to geometry constraints coupling beams are deep relative to their clear span. Behaviour of deep coupling beams may be controlled by shear, and these beams may be susceptible to strength and stiffness deterioration under earthquake loading. Most international codes prescribe use of diagonal reinforcement in deep coupling beams subjected to high shear stresses. Several
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 \( \tau_{ve} \) 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:

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 \( \alpha \) is the angle made by the diagonal reinforcement and 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 confinement. The confinement in a coupling beam shall be achieved by one of the following alternative arrangements shown in Fig. 16:

i) 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 used to promote the efficiency of the diagonal reinforcement:

\[ 2M_u = V_u L_s \]

Experimental studies on coupled structural walls showed that diagonal reinforcement provides adequate resistance in deep coupling beams (Barney et al. 1980). The experiments also showed that diagonal reinforcement is effective only if the bars are placed with a large inclination, hence diagonal reinforcement is restricted to coupling beams with \( L_s/D \) ratio of less than 4.0.

Diagonal bars should be placed approximately symmetrically in the beam cross-section, in two or more layers. The diagonally placed bars are intended to provide the entire shear and corresponding moment resistance of the beam. Note that longitudinal bars are also going to contribute to the moment resistance and should be considered in the design.

The design of a diagonally reinforced coupling beam is based on the assumption that the shear force resolves itself into diagonal compression and tension forces (Fig. C17).
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.

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

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

Two alternative confinement arrangements are proposed (10.5.2.1). The arrangement i) is currently included in 10.5.2 (with some modifications), while the arrangement ii) is new—and it has been adopted from ACI 318-14 Cl.18.10.7. The latter arrangement is expected to be easier to implement in practice than arrangement i).

![Diagram of coupling beams with diagonal reinforcement]

i) Confinement of individual diagonals

$A_{td} =$ TOTAL AREA OF REINFORCEMENT IN EACH GROUP OF DIAGONAL BARS

- LINE OF SYMMETRY
- WALL BOUNDARY REINFORCEMENT

![Diagram of coupling beams with diagonal reinforcement (new drawing)]

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 $1.25 \times L_d$ (see Fig. 16).

C10.5.3 –
The current provision is excessively stringent and difficult to implement in practice. International codes require smaller anchorage lengths (e.g. CSA A23.3-14 Cl.21.5.8.2.5) or do not specify anchorage length for diagonal reinforcement – implicitly recommending standard development length for bars in tension (ACI 318-19 Cl.18.10.7.4).

10.6 – Openings in Walls

10.6.1 –
Shear strength of a wall with openings should be checked at critical horizontal planes passing through openings.

C10.6.1 –
An opening in a structural wall causes high shear stresses in the region of the wall adjacent to it. Hence, it is necessary to check such regions for adequacy of horizontal shear reinforcement in order to prevent a diagonal tension failure due to shear.

10.6.2 –
Additional steel reinforcement shall be provided along all four edges of openings in walls. Further,
   a) the area of these 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:

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

where $\tau_v$ 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 $\mu$ is the coefficient of friction.

The coefficient of friction, $\mu$, 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.

C10.7 – Sliding friction resistance check at construction joints

This provision is based on the shear-friction concept which is applicable along the interfaces, such as construction joints, where shear stress is resisted by longitudinal reinforcement. In the context of special structural walls it is particularly important to check sliding shear resistance at the wall-to-foundation interface.

The design shear force at the joint must be less than the shear force that can safely be transferred across the joint, $V_j$. This is calculated by applying the shear friction concept and is given by

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

It is proposed to ignore the contribution of axial load ($P_u = 0$), which is conservative and in line with the ACI 318-14 provisions.

The shear strength at the construction joint is

$$\tau_v = \frac{V_j}{A_g}$$

10.8 – Development, Splice and Anchorage Requirement

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

C10.8 – Development, Splice and Anchorage Requirement

C10.8.1 –

Anchorage of horizontal reinforcement is currently addressed in 10.1.7, but it is considered more appropriate to discuss it in clause 10.8.1. The proposed provision is illustrated in Fig. C18, and is in line with anchorage requirements of international codes (CSA A23.3-14 Cl.21.5.5.2 and ACI 318-14 Cl.18.10.6.4h).

![Fig. C18 – Anchorage of horizontal reinforcement (source: CSA A23.3-14 Cl.21.5.5.2) (new drawing)](new drawing)
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<th>Proposed Modifications &amp; Commentary IS 13920 : 2016</th>
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**10.8.2 – Splicing of vertical reinforcement**

In slender walls \( H/L > 2 \), 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) \( L_d \) above the base of the wall; and
b) \( 1/6 \)th of the wall height;

but not larger than \( 2L_d \).

a) Within the plastic hinge region of slender flexural walls \( H/L > 2 \), 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 \( L_d \).  

**C10.8.2 –**

The current clause (part a) suggests that splicing of vertical reinforcement should be avoided within a plastic hinge region, however that is rarely possible. The proposed provision allows for splicing of 50% reinforcement within the plastic hinge region, which is in line with international codes (e.g. CSA A23.3-14 Cl.21.5.6.5), however it is also permitted to splice 100% vertical reinforcement under certain conditions.

It is proposed to increase lap splice length within plastic hinge region to 125% of the bar development length in tension. Current Clause 10.8.3.1 requires lap splice length equal to 100% of the development length at all locations. An increased lap splice length is in line with the requirements of international codes (e.g. CSA A23.3-14 Cl.21.5.4.1).

The current provision also defines plastic hinge region for detailing purposes. It is proposed to include a separate clause where the length of plastic hinge region is defined (Clause 10.1.11).

Part b) is related to splicing of vertical reinforcement outside the plastic hinge length, where the requirements are less stringent and splice length is less than within plastic hinge zone.

**10.8.3 – Splices-and cross-ties for vertical reinforcement**

**C10.8.3. –**

Current provision addresses both lap splices and closed links over the spliced length. It is proposed to separate clauses related to lap splices and cross-ties in vertical reinforcement within 10.8.2.

<table>
<thead>
<tr>
<th>10.8.3.1 – Lap-splices</th>
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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...
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<tr>
<th><strong>Proposed Modifications &amp; Commentary IS 13920 : 2016</strong></th>
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<tr>
<td><strong>10.8.3.2 – Mechanical couplers (conforming to IS 16172)</strong> 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.</td>
</tr>
<tr>
<td><strong>10.8.3.3 – Welded splices</strong></td>
</tr>
<tr>
<td>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.</td>
</tr>
<tr>
<td><strong>10.8.4 –</strong> 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.</td>
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**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.
10.9 Additional requirements for squat structural walls\textsuperscript{6}

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

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, $\rho_h$, shall be determined as follows:

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

Where shear stress $\tau_v$ 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:

C10.9 –

Squat structural walls may not be able to develop a ductile flexural failure mechanism like flexural structural walls, hence additional design rules are required for these walls. Comprehensive provisions for design of squat structural walls have been provided in some international codes, e.g. Canadian code CSA A23.3-14 Cl.21.5.10.

C10.9.1 –

Distributed reinforcement in squat walls has a significant role in controlling the width of diagonal cracks, hence two curtains of reinforcement are needed to control diagonal cracking.

C10.9.3 –

When $h_w/L_w < 1.0$ the shear force is resisted by diagonal compression stresses that are relatively uniform across the base of the wall. The required vertical reinforcement determined in 10.9.3 is needed to balance the vertical component of compression.

\textsuperscript{6}This is a new clause
\[ \rho_v = \rho_h - \frac{P_u}{0.87 f_y A_g} \]

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

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 $\tau_v$ 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.

Shear design requirements are more relaxed than for the special structural walls (Clause 10.2).

C11 – Intermediate Structural Walls (ISW)
Implementation of ductile detailing provisions requires substantially higher effort in design, construction and quality control. Intermediate Structural Walls (ISW) have somewhat lower ductility requirements and are assigned a lower $R$ value (corresponding to higher seismic design force) than special structural walls.

C11.1 –
Requirements regarding the minimum distributed reinforcement are the same as for special structural walls.

A boundary element is not required, but there is a need to provide confined concentrated reinforcement at the wall ends.

C11.2 –
Shear design requirements are more relaxed than for the special structural walls (Clause 10.2).

---

7 This is a new clause
### 11.3 – Design for axial load and bending

Design moment of resistance $M_u$ 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.

### C11.3 –

Design requirements for axial load and bending are more relaxed than for the special structural walls (Clause 10.3).
1112 –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).

C12.1 –

One of the most common causes of building collapses during earthquakes is failure of one or more components which are not considered to be part of the lateral force resisting system, such as gravity load-resisting frames. These frames are particularly common in buildings with RC structural walls, where structural walls constitute the main lateral force resisting system (see Fig. C19).

It is very important to consider structural members (frames, walls) that are not a part of the lateral force resisting system in a numerical model of the structure which is used for seismic analysis. This can be accomplished by providing horizontal axially rigid links with hinges (moment releases) at the ends to connect these members with the lateral force resisting system. Refer to Stafford Smith and Coull (1991) for more information on numerical modelling of buildings using rigid links.

The members that are not a part of the lateral force resisting system need not be designed for increased forces, but the design must ensure that these members either remain elastic or yield in bending. These members should be detailed to ensure adequate shear and flexural resistances, as well as ductility.

GRAVITY LOAD RESISTING FRAME

RC SHEAR WALL

Fig. C19 – Plan of a building with structural walls and gravity-load resisting frame (new drawing)

Seismically-induced bending moments in the columns and beams of gravity-load resisting frames can be obtained by multiplying the applied seismic forces for the entire structure obtained from IS 1893 (Part 1) by ‘R/I’ value taken for the seismic force resisting system. This effectively means that the structure is subjected to “elastic” seismic forces corresponding to R=1 and I=1.

An increase in the seismic forces results in a multifold increase of the design lateral displacements (‘R/I’ times), as stated in Clause 12. The resulting internal forces in the beams and columns of the gravity-load resisting frame obtained in this manner shall be used to check
seismic safety of a gravity-load resisting frame. According to 7.11.2 of IS 1893 : 2016 (Part 1), it is required to consider displacement under unfactored loads while evaluating deformation compatibility of non-seismic members.

### 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.

### 142.31.4 – 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:

---

**C12.3 –**

Detailing requirements for the columns that are part of the lateral force resisting system assume that the members may undergo deformations that exceed the yield limit of the member without a significant loss of strength. On the other hand, the columns that are not part of the lateral force resisting system are not required to meet stringent detailing requirements; however, these columns should be able to sustain the gravity loads even when subjected to earthquake-induced lateral displacements.

Factored bending moments in columns of gravity-load resisting frames due to applied forces per 12.1 may be very high due to uncertainty in displacement demands. The actual resistance of the column may be larger than estimated according to the code provisions, but the displacement demands may also be larger due to the variability of earthquake ground motions. This variability must be taken into account when designing potentially brittle members that may cause structural collapse.

The proposed limits on the calculated induced bending moments are based on CSA A23.3-14 Cl.21.11.3.3.3, which has drawn information from experimental studies, as well as guidelines and codes for seismic evaluation of existing buildings.
<table>
<thead>
<tr>
<th>Induced bending moment depending on the level of factored axial compressive stress due to factored load combinations including seismic loads</th>
<th>Column detailing requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \leq 0.16f_{ck} )</td>
<td>( \geq 0.32f_{ck} )</td>
</tr>
<tr>
<td>( &lt;1.5M_u )</td>
<td>( &lt;1.0M_u )</td>
</tr>
<tr>
<td>( \geq 1.5M_u ) but ( &lt;3.0M_u )</td>
<td>( \geq 1.0M_u ) but ( &lt;2.0M_u )</td>
</tr>
<tr>
<td>( \geq 3.0M_u ) but ( &lt;5.0M_u )</td>
<td>( \geq 2.0M_u ) but ( &lt;3.0M_u )</td>
</tr>
</tbody>
</table>

**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).

112.1.2 –
Gravity columns with factored gravity axial stress exceeding 0.4\( f_{ck} \) shall satisfy 11.1.1 and shall have transverse reinforcement at least one half of special confining reinforcement required by 7.6.

112.42 – **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.

This clause should be removed, because its content is covered by 12.3.

C12.4 –
See commentary for 12.3.
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:

<table>
<thead>
<tr>
<th>Induced bending moment</th>
<th>Beam detailing requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>$&lt; 1.0M_u$</td>
<td>IS 456 beam detailing requirements</td>
</tr>
<tr>
<td>$\geq 1.0M_u$ but $&lt; 3.0M_u$</td>
<td>Beams of intermediate moment resisting frames (IMRFs) (9.1.4)</td>
</tr>
<tr>
<td>$\geq 3.0M_u$ but $&lt; 5.0M_u$</td>
<td>Beams of special moment resisting frames (SMRFs) (6.3.1, 6.3.2 and 6.3.5)</td>
</tr>
</tbody>
</table>

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).

112.2.1 – Mechanical and welded splices shall satisfy 7.3.2.2 and 7.3.2.3. This clause should be removed.

112.2.2 – Gravity columns shall satisfy 7.4 and 7.6. This clause should be removed, since its content is covered by 12.3.

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:

<table>
<thead>
<tr>
<th>Induced bending moment depending on the level of factored axial compressive stress due to factored load combinations including seismic loads</th>
<th>Wall detailing requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤0.16f\text{ck}</td>
<td>≥0.32f\text{ck}</td>
</tr>
<tr>
<td>&lt;0.7M\text{u}</td>
<td>&lt;0.5M\text{u}</td>
</tr>
<tr>
<td>≥0.7M\text{u} but &lt;1.0M\text{u}</td>
<td>≥0.5M\text{u} but &lt;0.7M\text{u}</td>
</tr>
<tr>
<td>≥1.0M\text{u} but &lt;1.2M\text{u}</td>
<td>≥0.70M\text{u} but &lt;0.80M\text{u}</td>
</tr>
</tbody>
</table>

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 \(0.2[0.035 - 0.05(r_{sv}/k_{sv})]\).

C12.6 –
The seismic design requirements for walls that are part of the gravity-load resisting system depend on the inelastic flexural deformation demand. When seismic demands on the walls 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 Clause 12.1 exceeds the moment of resistance M\text{u}. The proposed provision is in line with CSA A23.3-14 Clause 21.11.3.3.

C12.7-
As drift limitation (0.1%) for flat slabs/plates is removed in IS 1893 (Part 1) : 2016, it is necessary to check slab-column connections for drift-induced punching shear. The proposed provision is in line with ACI 318-19 Cl. 18.14.5.
<table>
<thead>
<tr>
<th><strong>Proposed Modifications &amp; Commentary IS 13920 : 2016</strong></th>
</tr>
</thead>
</table>

| Where, $\tau_{uv}$ is factored shear stress at critical section $d/2$ from face of support, for seismic load combination only. |
| $k_1 \tau_s$ is concrete shear capacity as per clause 31.6.3 of IS 456 : 2000, and |
| $d$ = effective depth of slab/drop panel. |

*Note: The table contains technical information related to structural engineering, specifically concerning shear stress and concrete capacity in the context of seismic load combinations.*
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 \(\alpha\) times unbalanced moment, where \(\alpha\) 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.

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\(^8\) This is a new clause
<table>
<thead>
<tr>
<th>Section</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>13.3</strong>-</td>
<td>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.</td>
</tr>
<tr>
<td><strong>13.4</strong>-</td>
<td>At least one-fourth of the top reinforcement at the support in the column strip shall be continuous throughout the span.</td>
</tr>
<tr>
<td><strong>13.5</strong>-</td>
<td>Continuous bottom reinforcement in the column strip shall be at least one-third of the top reinforcement at the support in the column strip.</td>
</tr>
<tr>
<td><strong>13.6</strong>-</td>
<td>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. <strong>C13.6</strong>- Experience has shown that flat slabs/plates may be prone to collapse when reinforcement is not placed continuously through the slab-column joint, as illustrated in Fig. C20. For that reason it is critical to ensure continuity of bottom reinforcement in flat plates/slabs.</td>
</tr>
<tr>
<td><strong>13.7</strong>-</td>
<td>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.</td>
</tr>
<tr>
<td><strong>13.8</strong>-</td>
<td>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...</td>
</tr>
</tbody>
</table>

*Fig. C20 – Shear failure of a flat plate showing the role of continuous reinforcement (Source: Brzev and Pao, 2016) (new drawing)*
accordance with clause 31.6.3 of IS 456 : 2000. This requirement need not be satisfied if the slab satisfies 12.7.
### 14 – Foundations

Foundations of RC frames and structural walls designed in accordance with IS 13920 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 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.

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**C14. – Foundations**

Foundations are critical elements of the lateral load path in a building structure. The foundations need to be designed to remain elastic during earthquake shaking. Also, foundation movements need to be considered when estimating lateral displacements in RC structural walls and frames. CSA A23.3-14 Cl.21.10 and explanatory notes provide a detailed coverage on the foundation design for earthquake ground shaking. ACI 318-14 Cl.18.13 also contains provisions related to foundation design for structures subjected to earthquake ground shaking.

**C14.1 –**

It is of critical importance to ensure that the foundations have sufficient capacity to remain elastic during an earthquake. This can be achieved by providing a margin of safety for the moment of resistance for the foundation design.

Note that IS 16700 : 2017 requires to check overturning and sliding considering overstrength factor of 2.5 for seismic loads, however it does not require foundations to be designed considering the overstrength factor.

ACI 318-19 commentary (clause 18.13.1) mentions that "requirement of design and detailing for foundations are of minimum level. However, because repairs to foundations can be extremely difficult and expensive, it may be desirable that the elements of the foundation remain essentially elastic during strong ground motions. Methods to achieve this goal include designing the foundation to include an overstrength factor or an increased seismic demand level when compared to the superstructure, or comparing strengths to demands predicted by nonlinear response history analyses with appropriate consideration of uncertainty in demands.

**C14.2 –**

Foundations supporting RC structural wall and frame structures are expected to move under earthquake loading. The resulting lateral displacements due to rotation at the foundation level may cause a significant increase of overall lateral displacements in the building. For

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\(^9\) This is a new clause
example, foundation-induced displacements in structural wall structures are due to the foundation rotations at the base, but they will cause lateral displacements at the top due to rigid body wall movements. A detailed guidance regarding the estimation of foundation movements is provided in CSA A23.3-14 Cl. 21.10.3.3.

The foundation movements can be calculated using a static analysis. The footing rotation is calculated as the difference in vertical displacements at the ends of the footing, divided by the overall footing length. The analysis shall account for the assumed bearing stress distribution in the soil to resist the applied loads and the corresponding soil stiffness. The rotation shall be considered by increasing interstorey displacements from a fixed-base model at every floor level by an interstorey drift equal to the footing rotation (in radians).
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:

$$M_u = \varphi \left[ \left( \frac{x_u}{L_w} \right) \left( 0.416 \frac{x_u}{L_w} \right) \left( 0.168 + \frac{\beta^2}{3} \right) \right]$$

where

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

$$\lambda = \frac{0.0035}{L_u \left( 0.0035 + \left( \frac{0.002 + 0.87 f_y / E_s}{0.87 / E_s} \right) \right)}$$

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

$\varphi = \frac{0.87 f_y \rho}{f_{th}}$

$\lambda = \frac{P_u}{f_{th} t_u L_w}$

$\rho = \text{vertical reinforcement ratio} = A_{sv} / (t_u L_w)$

$A_{sv} = \text{area of uniformly distributed vertical reinforcement,}$

$E_s = \text{elastic modulus of steel, and}$

$P_u = \text{factored compressive axial force on wall}$

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

(b) For $x_u/L_w < x_u/L_u < 1.0$

The procedure for $M_u$ calculation for rectangular walls with distributed reinforcement was developed by Medhekar and Jain (1993). Note that expressions for $X_u/L_u$ and $\beta$ have been revised. Addition of 0.002 in steel strain leads to $\beta$ greater than unity indicating that even extreme most steel bar is not yielding. This was added in IS 13920:2016. It is proposed to be restored to the IS 13920:1993 formulation which is correct.

A-2

The same approach as presented in A-1 has been expanded to design structural walls with boundary elements which contain distributed vertical reinforcement and concentrated longitudinal reinforcement in boundary elements. Internal forces considered in the design are presented in Fig. CA1.
\[
\frac{M_u}{f_u t_u L_w} = a_1 \left( \frac{x_u}{L_w} \right) + a_2 \left( \frac{x_u}{L_w} \right)^2 - a_3 \frac{\lambda}{2}
\]

where

\[
a_1 = \left[ 0.36 + \varphi \left( 1 - \frac{B}{2} - \frac{1}{2\beta} \right) \right]
\]

\[
a_2 = \left[ 0.15 + \frac{\varphi}{2} \left( 1 - \beta + \frac{\beta^2}{3} - \frac{1}{3\beta} \right) \right] ; \text{and}
\]

\[
a_3 = \frac{\varphi}{6\beta} \left( \frac{1}{x_u/L_w} - 3 \right)
\]

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

\[
a_1 \left( \frac{x_u}{L_w} \right) + a_2 \left( \frac{x_u}{L_w} \right)^2 - a_3 - \alpha = 0
\]

where

\[
\alpha = \left( \frac{\varphi}{\beta} - \lambda \right) ; \text{and}
\]

\[
a_3 = \frac{\varphi}{2\beta}
\]

A-2* (Clause 10.4.2)

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

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

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

where

\[
\varphi = \left( \frac{0.87t_u \rho_u}{f_u} \right)
\]

\[
\lambda = \left( \frac{P_u}{f_u t_u L_w} \right)
\]

---

9 This is a new clause.
\[ \beta = \frac{0.87 f_y}{0.0035 E_s} \]

Note that reinforcement ratio for distributed vertical reinforcement \( A_{svw} \) within the wall web is

\[ \rho_w = A_{svw} / \left[ t_w (L_w - 2\alpha) \right] \]

Internal forces in steel and concrete are as follows

\[ T_1 = C_2 = 0.435 f_y \cdot t_w \cdot \rho_s \cdot \beta \cdot x_u \]

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

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

\[ C_c = 0.36 f_y \cdot t_w \cdot x_u \]

\[ T_3 = C_3 = 0.87 f_y \cdot A_t \] (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:

\[ M_u = C_c \cdot d_{c3} + C_3 \cdot d_{c2} + C_1 \cdot d_{c1} + C_2 \cdot d_{c1} - T_1 \cdot d_{t1} - T_2 \cdot d_{t2} - P_u \cdot d_{pu} \]

where

\[ d_{c3} = L_w - 0.416x_u \]

\[ d_{c1} = L_w - 0.5 \left[ x_u (1 - \beta) + a \right] \]

\[ d_{c2} = L_w - x_u (1 - 2\beta / 3) \]

\[ d_{c3} = L_w - a \]

\[ d_{t1} = L_w - x_u (1 + 2\beta / 3) \]

\[ d_{t2} = 0.5 \left[ L_w + a - x_u (1 + \beta) \right] \]

\[ d_{pu} = 0.5 L_w \]

Note that steel resultant forces in boundary elements \( (T_3 \text{ and } C_3) \) form a couple moment (at a lever arm \( d_{c3} = L_w - a \)), hence only one of these forces \( (C_3) \) appears in the equation for moment of resistance \( M_u \).
Fig. CA1 – Rectangular wall section with boundary elements (new drawing based on Medhekar and Jain, 1993)
COMMENTARY REFERENCES

1. ACI 318-14 and ACI 318-19, Building Code Requirements for Structural Concrete and Commentary, issued by the American Concrete Institute.

2. ACI 352, Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures, American Concrete Institute, 1989 and 2002.


9. CSA A23.3-14, Design of Concrete Structures and Explanatory Notes, issued by the Canadian Standards Association, Mississauga, ON, Canada, 2014.


22. NZS 3101.1: 2006, *Concrete Structures Standard*, including Amendments 1, 2, and 3, issued by Standards Council, New Zealand.


