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BUREAU OF INDIAN STANDARDS

MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG, NEW DELHI 110002
Extn 011-23608301, 011-23608588; Website: www.bis.gov.in

व्यापक परिचालन मसौदा

हमारा संदर्भ : सीईडी 39/टी-10 **26 अप्रैल 2023**

तकनीकी समिति : भूकंप इंजीनियरिंग अनुभागीय समिति, सीईडी 39

प्राप्तकर्ता:

1. **सिविल इंजीनियरिंग विभाग परिषद, सीईडीसी के सभी सदस्य**
2. **भूकंप इंजीनियरिंग विषय समिति, सीईडी 39 के सभी सदस्य**
3. **सीईडी 39 की उपसमितियों और अन्य कार्यदल के सभी सदस्य**
4. **रुचि रखने वाले अन्य निकाय।**

महोदय/महोदया,

निम्नलिखित मसौदा संलग्न है:

प्रलेख संख्या	**शीर्षक**
CED 39(22345)WC	**संरचनाओं के भूकम्परोधी डिज़ाइन के लिए मानदंड भाग 2 इमारतें** [आई एस 1893 (भाग 1) का सातवाँ पुनरीक्षण] **(आई सी एस संख्या: 91.120.25)**

कृपया इस मसौदे का अवलोकन करें और अपनी सम्मतियाँ यह बताते हुए भेजे कि यह मसौदा प्रकाशित हो तो इन पर अमल करने में आपको व्यवसाय अथवा कारोबार में क्या कठिनाइयां आ सकती हैं।

सम्मतियाँ भेजने की अंतिम तिथि: 25 जून 2023

सम्मति यदि कोई हो तो कृपया अधोहस्ताक्षरी को ई-मेल द्वारा **ced39@bis.gov.in** पर या उपरलिखित पते पर, संलग्न फोर्मेट में भेजें। टिप्पणियां बीआईएस ई-गवर्नेंस पोर्टल, www.manakonline.in के माध्यम से ऑनलाइन भी भेजी जा सकती हैं।

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प्रमुख (सिविल इंजीनियरिंग विभाग)

संलग्न: उपरिलिखित

भारतीय मानक ब्यूरो
BUREAU OF INDIAN STANDARDS

MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG, NEW DELHI 110002
Phone: + 91 11 23230131, 23233375, 23239402 Extn 8301, 23608588; Website: www.bis.gov.in

DRAFT IN WIDE CIRCULATION

DOCUMENT DESPATCH ADVICE

Reference	Date
CED 39/T-10	26 April 2023

TECHNICAL COMMITTEE:
EARTHQUAKE ENGINEERING SECTIONAL COMMITTEE, CED 39

ADDRESSED TO:

1. All Members of Civil Engineering Division Council, CEDC
2. All Members of Earthquake Engineering Sectional Committee, CED 39
3. All Members of CED 39's Subcommittees and Working Groups
4. All others interested

Dear Sir/Madam,

Please find enclosed the following draft:

Doc. No.	Title
CED 39(22345)WC	**Draft Indian Standard Criteria for earthquake resistant design of structures Part 2 Buildings** [Seventh revision of IS 1893 (Part 1)] **(ICS No. 91.120.25)**

Kindly examine the attached draft and forward your views stating any difficulties which you are likely to experience in your business or profession, if this is finally adopted as the National Standard.

Last Date for comments: 25 June 2023

Comments if any, may please be made in the enclosed format and emailed at ced39@bis.gov.in or sent at the above address. Additionally, comments may be sent online through the BIS e-governance portal, www.manakonline.in .

In case no comments are received or comments received are of editorial nature, you will kindly permit us to presume your approval for the above document as finalized. However, in case comments, technical in nature are received, then it may be finalized either in consultation with the Chairman, Sectional Committee or referred to the Sectional Committee for further necessary action if so desired by the Chairman, Sectional Committee.

The document is also hosted on BIS website **www.bis.gov.in**.

Thanking you,

Yours faithfully,
Sd/-
(ArunKumar S)
Scientist 'E'/Director &
Head (Civil Engg. Department)

Encl: As above

FORMAT FOR SENDING COMMENTS ON BIS DOCUMENTS

(Please use A4 size sheet of paper only and type within fields indicated. Comments on each clause/subclause/table/fig, etc be started on a fresh box. Information in column 5 should include reasons for the comments, and those in column 4 should include suggestions for modified wording of the clauses when the existing text is found not acceptable. Adherence to this format facilitates Secretariat's work) {Please e-mail your comments to ced39@bis.gov.in }

DOC. NO: CED 39(22345)WC
TITLE: CRITERIA FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES PART 2 BUILDINGS *[Seventh revision of IS 1893 (Part 1)]* (ICS 91.120.25)

LAST DATE OF COMMENTS*: *25 JUNE 2023

NAME OF THE COMMENTATOR/ORGANIZATION: ....

Sl. No. (1)	Clause/ Subclause/ Para No. (2)	Comments/Suggestions (3)	Modified Wordings of the Clause (4)	Reasons/ Justifications for the Proposed Change (5)

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Draft Indian Standard

CRITERIA FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

PART 2 BUILDINGS

[*Seventh Revision of IS 1893 (Part 1)*]

ICS No. 91.120.25

Earthquake Engineering Sectional Committee, CED 39	**Last Date for Comments: 25 June 2023**

FOREWORD

(*Formal Clause will be added later*)

About 59 percent of India's land area (with over 78 percent of its population living in it) is prone to moderate to strong earthquake shaking intensities (*see* Annex E of IS 1893 (Part 1) for 1964 MSK Intensity Scale for earthquake ground shaking). Hence, earthquake resistant design and construction is essential.

The standard IS 1893: 1962 'Recommendations for earthquake resistant design of structures' was first published in 1962, and revised in 1966, 1970, 1975 and 1984. In the fifth revision, IS 1893 was decided to be published in parts. Part 1 “General provisions and buildings” was then published in the year 2002 and revised again in 2016. The significant changes in the 2016 version were: definition of design spectra for natural period up to 6 s; use of same design spectra for all buildings (irrespective of material of construction); intermediate importance category of buildings; flat slab buildings; clarity on dealing the irregularity in structural systems; inclusion of effect of masonry infill walls in analysis and design; introduction of method for arriving at the natural period of buildings with basements, step back buildings & buildings on hill slopes; and simplified method for liquefaction potential analysis. The other parts of IS 1893 which have been published are:

Part 2: 2014 Liquid retaining tanks
Part 3: 2014 Bridges and retaining walls
Part 4: 2015 Industrial structures and stack-like structures
Part 6: 2022 Base isolated buildings

Now, in 2023, the Committee decided to present the provisions for different types of structures in separate parts, to keep abreast with rapid developments and extensive research carried out in earthquake-resistant design of various structures. Thus, IS 1893 is split into 11 Parts, namely:

Part 1: General provisions
Part 2: Buildings
Part 3: Liquid retaining tanks
Part 4: Bridges and retaining walls
Part 5: Industrial structures
Part 6: Base isolated buildings
Part 7: Pipelines
Part 8: Dams and embankments *(to be formulated)*
Part 9: Coastal structures *(to be formulated)*
Part 10: Steel towers *(to be formulated)*
Part 11: Tunnels *(to be formulated)*

This standard contains provisions specific to earthquake-resistant design of buildings. Unless stated otherwise, the provisions in this part of IS 1893 shall be read necessarily in conjunction with the general provisions as laid down in IS1893 (Part 1). Also, the provisions of buildings in IS1893 (Part 2) are presented in 7 sections, namely:

Section 1: Additional Criteria for All Buildings
Section 2: Additional Criteria for *Masonry* Buildings
Section 3: Additional Criteria for *Concrete* Buildings
Section 4: Additional Criteria for *Steel* Buildings
Section 5: Additional Criteria for *Adobe* Buildings
Section 6: Additional Criteria for *Steel-Concrete Composite* Buildings
Section 7: Additional Criteria for *Timber* Buildings

However, in this standard (seventh revision), the following major changes have been included:

Section 1: Additional Criteria for All Buildings

a) Response reduction factors have been modified and renamed as Elastic force reduction factors;
b) Importance factors have been adjusted in view of the revision in the hazard levels;
c) Provisions on torsion have been improved;
d) Provisions on buildings have been harmonized from relevant provisions given in IS 4326, IS 13826, IS 13827 and IS 15988; and
e) Additional provisions have been added for earthquake resistant design of critical and lifeline buildings.

Section 2: Additional Criteria for Masonry Buildings

a) Provisions on buildings have been harmonized from IS 4326 and IS 15988; and
b) The admissibility of different structural systems in the earthquake zones is clarified.

Section 3: Additional Criteria for Concrete Buildings

a) The admissibility of different structural systems in the earthquake zones is clarified; and
b) The structural plan density of the structural walls is varied with the earthquake zone and category of the building.

Section 4: Additional Criteria for Steel Buildings

a) New provisions have been prepared for steel buildings; and
b) The structural plan density of the structural walls is varied with the earthquake zone and category of the building.

Thus, a parallel series of standards on design & detailing of structures will be formulated under IS 13920 (Parts 1 to 11) and another series of standards on safety assessment and retrofitting of structures under IS 13935 (Parts 1 to 11).

In the formulation of this standard, effort has been made to coordinate with standards and practices prevailing in different countries in addition to relating it to the practices in the field in this country. Assistance has particularly been derived from the following publications:

a) IBC 2021, International Building Code, International Code Council, USA, 2021;
b) NEHRP 2020, NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Volume I: Part 1 Provisions, Part 2 Commentary, Report No.FEMA P2082-1, Federal Emergency Management Agency, Washington, DC, USA, September 2020;
c) ASCE/SEI 7-22, Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, USA, 2022;
d) NZS 1170.5: 2016, Structural Design Actions, Part 5: Earthquake Actions – New Zealand, Standards New Zealand, Wellington, New Zealand, 2016;
e) JICA, Specifications for Highway Bridges, Part IV, Japan International Cooperation Agency, 1994

The compostion of the Committee including the Drafting Group responsible for the formulation of this standard will be included later (after finalization) in Annex B.

This standard contributes to the following Sustainable Development Goals: Goal 9 'Industry, Innovation and Infrastructure' towards building resilient infrastructure, promote inclusive and sustainable industrialization and foster innovation; and *Goal 11* 'Sustainable cities and communities' towards making cities and human settlements inclusive, safe, resilient and sustainable.

For the purpose of deciding whether a particular requirement of this standard is complied with, the final value observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS 2: 2022 'Rules for rounding off numerical values (*revised*)'. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

Draft Indian Standard

CRITERIA FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

PART 2 BUILDINGS

[*Seventh Revision of IS 1893 (Part 1)*]

SECTION 1

ADDITIONAL CRITERIA FOR ALL BUILDINGS

1 SCOPE

The provisions of this section shall be applicable to all buildings covered in Section 2 to 7.

1.1 Masonry Buildings

1.1.1 Section 2 of this standard deals with the selection of materials, special features of design and construction for earthquake resistant masonry buildings, including masonry construction using rectangular masonry units, timber construction and buildings with prefabricated flooring/roofing elements.

1.1.2 Guidelines for earthquake resistant buildings constructed using masonry of low strength and earthen buildings are covered in IS 13826 and IS 13827.

1.2 Concrete Buildings

Section 3 of this standard deals with the selection of materials, special aspects related to design and construction for earthquake resistant concrete buildings.

1.3 Steel Buildings

Section 4 of this standard deals with the selection of materials, special aspects related to design and construction for earthquake resistant steel buildings.

2 REFERENCES

The standards listed in Annex A 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 edition of the standards indicated in Annex A.

3 TERMINOLOGY

The definitions given below shall apply for the purpose of earthquake resistant design of buildings, as enumerated in IS 1893 (Part 1).

3.1 All Buildings

For the purpose of buildings referred to in this standard, the following definitions shall apply.

3.1.1 *Base* — The level at which inertia forces generated in the building are considered to be transferred to the ground through the foundation. For buildings with basements, it is considered at the bottommost basement level. For buildings resting on:

a) pile foundations, it is considered to be at the top of pile cap;
b) raft foundations, it is considered to be at the top of raft; and
c) footings, it is considered to be at the top of the footing.

For buildings with combined types of foundation, the base is considered as the bottom-most level of the bases of the constituent individual foundations as per definitions above.

3.1.2 *Base Dimension (d)*— The dimension (in metre) of the base of the building along a direction of shaking.

3.1.3 *Beam* **—** Member (generally horizontal) resisting loads through flexural and shearing actions.

3.1.4 *Building* **—** An enclosed structure for whatsoever purpose and of whatsoever materials constructed with structural and non-structural elements and whose intended use maybe for human habitation or not.

3.1.5 *Centre of Mass (CM)* — The point in the floor of a building through which the resultant of the inertia force of the floor is considered to act during earthquake shaking. Unless otherwise stated, the inertia force considered is that associated with the horizontal shaking of the building.

3.1.6 *Centre of Resistance (CR)*

3.1.6.1 *For Single Storey Buildings* — The point on the roof of a building through which when the resultant internal resistance acts, the building undergoes:

a) pure translation in the horizontal direction; and
b) no twist about vertical axis passing through the *CR*.

3.1.6.2 *For Multi-Storey Buildings* — The set of points on the horizontal floors of a multi-storey building through which, when the resultant incremental internal resistances across those floors act, all floors of the building undergo:

a) pure translation in the horizontal direction; and
b) no twist about vertical axis passing through the *CR*.

3.1.7 *Column* — A member (generally vertical) resisting loads through axial, flexural and shearing actions.

3.1.8 *Concrete Grades* — The 28-day compressive strength (in MPa) of concrete cubes of 150 mm size as per IS 456.

3.1.9 *Core Structural Walls, Perimeter Columns, Outriggers and Belt Truss System* — A structural system comprising of a core structural walls and perimeter columns, resisting the vertical and lateral loads, with

a) the core structural walls connected to select perimeter column element(s) (often termed outrigged columns) by deep beam elements, known as outriggers, at discrete locations along the height of the building; and

b) the outrigged columns connected by deep beam elements (often known as belt truss), typically at the same level as the outrigger elements.

A structure with this structural system has enhanced lateral stiffness, wherein core structural walls and perimeter columns are mobilized to act with each other through the outriggers, and the perimeter columns themselves through the belt truss. The global lateral stiffness is sensitive to: flexural stiffness of the core element, the flexural stiffness of the outrigger element(s), the axial stiffness of the outrigged column(s), and the flexural stiffness of the outrigger elements connecting the core structural walls to the perimeter columns.

3.1.10 *Design Earthquake Base Shear* (V_B) — The horizontal lateral force in the considered direction of earthquake shaking that the structure shall be designed for.

3.1.11 *Diaphragm* — A horizontal or nearly horizontal structural system (for example, reinforced concrete floors and horizontal bracing systems), which transmits lateral forces to vertical elements connected to it.

3.1.12 *Eccentricity*

3.1.12.1 *Design eccentricity* (e_{di}) — The value of eccentricity to be used for floor *i* in calculations of design torsion effects.

3.1.12.2 *Static eccentricity* (e_{si}) — It is the distance between centre of mass *CM* and centre of rigidity *CR* of floor *i*.

3.1.13 *Earthquake Zone* — The Earthquake Zones II to VI as classified in **6.2.1** of IS 1893 (Part 1).

3.1.14 *Floating Column* — A vertical element (column) which at its lower level (termination level) rests on a beam.

3.1.15 *Gravity Column* — Column, which is not a part of the designated lateral load resisting system, and are designed to resist: (a) gravity loads, and (b) force effects due to displacement compatibility induced during earthquakes, through axial, flexural and shearing actions.

3.1.16 *Height of Floor* (h_i) — The difference in vertical elevations (in m) of the base of the building and top of floor *i* of the building.

3.1.17 *Height of Building* (h) — The height of building (in m) from its base to top of roof level,

a) excluding the height of basement storeys, if basement walls are connected with the ground floor slab or basement walls are fitted between the building columns, but
b) including the height of basement storeys, if basement walls are not connected with the ground floor slab and basement walls are not fitted between the building columns.

In step-back buildings, it shall be taken as the average of heights of all steps from the base, weighted with their corresponding floor areas. In buildings founded on hill slopes, it shall be taken as the height of the roof from the top of the highest footing level or pile cap level.

3.1.18 *Horizontal Bracing System* — A horizontal truss system that serves the same function as a diaphragm.

3.1.19 *Joints* — Portions of columns that are common to beams/braces and columns, which frame into columns.

3.1.20 *Lateral Force Resisting System* — The arrangement of structural members, which consists of all structural members that resist lateral load actions imposed or applied on the structure, especially the inertia forces induced in the structure during earthquake shaking. It is part of the overall structural system of the structure.

3.1.21 *Moment-Resisting Frame* (*MRF*) — An assembly of interconnected beams and columns (without structural walls and inclined members as braces, which function as a complete self-contained unit with or without the aid of horizontal diaphragms of floor bracing systems), which resist induced and externally applied forces primarily by axial and flexural actions.

3.1.21.1 *Ordinary moment-resisting frame* (*OMRF*) — A MRF designed and detailed as per IS 456 or IS 800, but not meeting special detailing requirements for ductile behaviour as per IS 13920 or IS 800, respectively.

3.1.21.2 *Special moment-resisting frame* (*SMRF*) — A *MRF* whose members are specially designed and detailed to provide ductile behaviour, and meet basic design requirements given in IS 456 or IS 800, and special detailing requirements given in IS 13920 or IS 800, respectively.

3.1.22 *Moment-Resisting Frame with Structural Walls* — A MRF with a structural wall in one or more of its bays.

3.1.23 *Structural Wall* — A planar member (generally vertical), generally to be provided along the full height of the building, which resists load actions through axial, flexural and shearing actions in its own plane.

3.1.24 *Number of Storeys, n* — The number of levels of a building above the base at which mass is present in substantive amounts. This,

a) excludes the basement storeys, where basement walls are connected with the ground floor deck or fitted between the building columns; and
b) includes the basement storeys, when they are not so connected.

3.1.25 *Principal Plan Axes* — Two mutually perpendicular horizontal directions in plan of a building along which the geometry of the building is oriented.

3.1.26 *P-Δ Effect* — The secondary effect on shear forces and bending moments of lateral force resisting elements generated under the action of the vertical loads, interacting with the lateral displacement of a building resulting from earthquake effects.

3.1.27 *RC Structural Wall* — A wall designed to resist lateral forces acting in its own plane.

3.1.27.1 *Ordinary RC structural wall* — Reinforced concrete (RC) structural wall designed and detailed as per IS 456, but not meeting special detailing requirements for ductile behaviour as per IS 13920.

3.1.27.2 *Special RC structural wall* — RC structural wall designed and detailed as per IS 13920, and meeting special detailing requirements for ductile behaviour as per IS 13920.

3.1.28 *Storey* — The space between two adjacent floors.

3.1.28.1 *Soft storey* — It is one in which the lateral stiffness is less than that in the storey above. The storey lateral stiffness is the total stiffness of all earthquake force resisting elements resisting the lateral earthquake shaking effects in the considered direction.

3.1.28.2 *Weak storey* — It is one in which the storey lateral strength [cumulative design shear strength of all structural members other than that of unreinforced masonry (URM) infills] is less than that in the storey above. The storey lateral strength is the total strength of all earthquake force resisting elements sharing lateral storey shear in the considered direction.

3.1.29 *Storey Drift* — The relative displacement between the floors above and/or below the storey under consideration.

3.1.30 *Storey Lateral Shear Strength* (S_i) — The total lateral strength of all lateral force resisting elements in the storey considered in a principal plan direction of the building.

3.1.31 *Storey Lateral Translational Stiffness* (K_i) — The total lateral translational stiffness of all lateral force resisting elements in the storey considered in a principal plan direction of the building.

3.1.32 *Storey Shear* (V_i) — The sum of design lateral forces at all levels above the storey *i* under consideration.

3.1.33 *Structural Wall Plan Density* (ρ_{sw}) — The ratio of the cross-sectional area at the plinth level of RC structural walls resisting the lateral load, and the plinth of the building, expressed as a percentage.

3.1.34 *Structural Wall* — A vertical plate-like member of a structure designed to resist forces in the own plane by axial, flexural and shearing actions.

3.2 Masonry Buildings

For the purpose of masonry buildings referred to in this standard, the following additional definitions shall apply.

3.2.1 *Band* — A reinforced concrete or reinforced brick runner provided in the walls to tie them together and to impart horizontal bending strength in them.

3.2.2 *Box System* — A bearing wall structure without a space frame, the horizontal forces being resisted by the walls that act as shear walls.

3.2.3 *Separation Section* — A gap of specified width between adjacent buildings or parts of the same building either left uncovered or covered suitably to permit movement in order to avoid pounding due to earthquake.

3.2.3.1 *Crumple section* — The separation gap filled with appropriate material that crumples or factures in the event of an earthquake.

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

3.3 Concrete Buildings

For the purpose of concrete buildings referred to in this standard, the following additional definitions shall apply:

3.3.1 *Beam-Column Joint* — The portion of a RC MRF at the junction of beams and columns. It consists of the reinforcement from the beams and columns, with or without stirrups around the longitudinal steel bars of the column or of the beam.

3.3.2 *Infill* — The masonry material placed between the beams and columns after the construction of the MRF.

3.4 Steel Buildings

For the purpose of steel buildings referred to in this standard, the following additional definitions shall apply:

3.4.1 *Brace* — A member (generally inclined) resisting loads through axial actions.

3.4.2 *Connection* — The assembly of fasteners, including plates, bolts and welds, that are used to join two members.

3.4.3 *Concentrically Braced Frame* (*CBF*) — A lateral load resisting system composed of interconnected beams and columns with inclined members as braces, which function as a complete self-contained unit with or without the aid of horizontal diaphragms of floor bracing systems, in which the system resists gravity and lateral force effects primarily by axial actions.

3.4.3.1 *Special concentrically braced frame* (*SCBF*) — A specially designed and detailed to provide ductile behaviour as per the requirements specified in this standard.

3.4.4 *Eccentrically Braced Frame* (*EBF*) — A lateral load resisting system composed of interconnected beams and columns with inclined members as braces that has at least one end connected to a beam through a link with a defined eccentricity from another beam-to-brace connection, which functions as a complete self-contained unit with or without the aid of horizontal diaphragms of floor bracing systems, in which the system resist gravity and lateral force effects primarily by axial action in the braces and shearing and flexural actions in the links. It is specially designed and detailed to provide ductile behaviour as per the requirements specified in this standard.

3.4.5 *Joint Panel Zone* — The part of the column that is adjoining the beam. It consists of the column web, column flanges, and continuity plates in line with the flanges of beams that connect to the column.

3.4.6 *Link* — The segment of a beam that is located between the ends of the connections of two diagonal braces in EBFs. The length of the link is defined as the clear distance between the ends of two diagonal braces.

4 SYMBOLS

The symbols and notations given below apply to the provisions of this standard:

$A_{\mathrm{H}}(T)$	Elastic maximum horizontal PSA depending on the natural period T of the building of horizontal translational mode, corresponding to the return period T_{RP} and the category of building specified in Table 1 of IS 1893 (Part 1) for strength design or serviceability check
$A_{\mathrm{HD}}(T)$	Design horizontal acceleration coefficient along any horizontal direction of a building of a given structural system in a given earthquake zone and resting over a site of given category for strength design of structure (by Limit State Method), which is a function of the natural period T of the structure
$A_{\mathrm{HD,k}}$	Design horizontal PSA using natural period of oscillation T_{k} of mode k obtained from free vibration analysis
$A_{\mathrm{HS}}(T)$	Horizontal acceleration coefficient for serviceability check along any principal direction of a building of a given structural system in a given

	earthquake zone and resting over a site of given category
$A_V(T)$	Elastic maximum vertical PSA depending on the natural period T of the building of horizontal translational mode, corresponding to the return period T_{RP} and the category of building specified in Table 1 of IS 1893 (Part 1) for strength design or serviceability check
$A_{VD}(T)$	Design vertical acceleration coefficient along vertical direction of a building of a given structural system in a given earthquake zone and resting over a site of given category for strength design of structure (by Limit State Method), which is a function of the natural period T of the structure
$A_{VS}(T)$	Vertical acceleration coefficient for serviceability check along any principal direction of a building of a given structural system in a given earthquake zone and resting over a site of given category
$A_{a,gross}$	Gross axial area of the cross-section of a member
$A_{a,e}$	Effective axial area of the cross-section of a member
$A_{s,e}$	Effective shear area of the cross-section of a member
A_o	Area of openings in the slab
A_s	Full area of the floor slab
$A_{s,gross}$	Gross shear area of the cross-section of a member
A_w	Total effective area (m^2) of walls in the first storey of the building
A_{wi}	Effective cross-sectional area (in m^2) of wall *i* in first storey of building
B	Outer dimension of the building along the direction perpendicular to the considered direction of shaking
b_i	Floor plan dimension of floor *i*, perpendicular to the considered direction of earthquake force
d	Base dimension (in m) of the building at the plinth level along the considered direction of earthquake shaking
E_c	Modulus of elasticity of concrete used in the moment resisting frame
E_b	Modulus of elasticity (in MPa) of clay brick masonry
e_{di}	Design eccentricity to be used at floor *i*
e_K	Stiffness eccentricity of the building with respect to *CR*
e_{si}	Static eccentricity at floor *i* defined as the distance between *CM* and *CR*
F_i	Design lateral force at level of floor *i*
F_{roof}	Design lateral force at roof level
f_b	Compressive strength (in MPa) of brick
f_m	Compressive strength of masonry prism (in MPa)
f_{mo}	Compressive strength (in MPa) of mortar
g	Acceleration due to gravity
h	Height (in m) of the building, excluding the basement storeys, where basement storey, walls are connected with the ground floor deck or fitted between the building columns, but including the basement storeys, when they are not so connected, or clear height of URM infill wall between the top beam and bottom floor slab
h_i	Height of floor *i* measured from base
I	*Importance Factor* that reflects the relative importance within each category of the structures
I_e	Effective second moment of area of the cross-section of a member

I_c	Second moment of area of the column adjoining the infill
I_{gross}	Gross second moment of area of the cross-section of a member
K_i	Stiffness of storey *i* of the building
L	Horizontal dimension of the lateral force resisting system at the base of the building, or overall plan dimension of the building at the base of the building
L_o	Offset length of the structural wall in plan
L_w	Length of the structural wall in plan
L_{wi}	Length (in m) of structural wall *i* in first storey in the considered direction of lateral forces
L_X	Overall dimension in plan of the wing of the building oriented along X-direction
L_Y	Overall dimension in plan of the wing of the building oriented along Y-direction
L_{pX}	Projected dimension in plan of the wing of the building oriented along X-direction beyond the core
L_{pY}	Projected dimension in plan of the wing of the building oriented along Y-direction beyond the core
l	Clear length of the URM infill wall between the vertical RC elements (columns, walls or a combination thereof) between which it spans
M_k	Modal mass of mode k of oscillation of the building
N	Number of storeys in building, that is, levels at which masses are located
N_w	Number of walls in the considered direction of earthquake shaking
P_k	Mode participation factor of mode *k* of oscillation of the building
Q_i	Design Storey Lateral Force at floor *i*
Q_{ik}	Design Storey Lateral Force at floor *i* in mode *k* of oscillation of the building
R	Elastic force reduction factor *R* for buildings with different structural systems used in the estimation of the design earthquake lateral force of the structure
R_X	Elastic force reduction factor of building for the lateral load resisting structural system employed along X-direction
R_{XA}	Elastic force reduction factor of building A for the lateral load resisting structural system employed along X-direction
R_{XB}	Elastic force reduction factor of building B for the lateral load resisting structural system employed along X-direction
R_Y	Elastic force reduction factor of building for the lateral load resisting structural system employed along Y-direction
R_{YA}	Elastic force reduction factor of building A for the lateral load resisting structural system employed along Y-direction
R_{YB}	Elastic force reduction factor of building B for the lateral load resisting structural system employed along Y-direction
R_1	Elastic force reduction factor of building 1 or unit 1 of the same building
R_2	Elastic force reduction factor of building 2 or unit 2 of the same building
r	Translational radius of gyration of the mass of the building at the floor level
$r_{k\theta}$	Torsional radius of gyration of the mass of the building at the floor level
S_i	Lateral strength of storey *i* of the building

T_a	Approximate fundamental translational natural period of oscillation (in s) estimated by empirical expressions
T_X	Fundamental translational modes of oscillation of the building along plan direction X
T_Y	Fundamental translational modes of oscillation of the building along plan direction Y
T_θ	Fundamental torsional modes of oscillation of the building
t	Thickness of infill masonry wall in building
$V_{BD,H}$	Design horizontal base shear force along any principal horizontal direction of a building for *strength design* of members (by Limit State Method), or for serviceability check of members (by Working Stress Method)
$V_{BD,V}$	Design vertical base shear force along vertical direction of a building for strength design of members (by Limit State Method), or for serviceability check of members (by Working Stress Method)
$V_{BD,H,min}$	Minimum design horizontal earthquake base shear force below which $V_{BD,H}$ cannot be taken
V_{BX}	Design base shear force for earthquake shaking considered along X-direction in plan
V_{BY}	Design base shear force for earthquake shaking considered along Y-direction in plan
$\bar{V}_{BX}$	Design base shear force for earthquake shaking considered along X-direction in plan, which is estimated using the empirical expression given in IS 1893 (Part 2) for approximate fundamental natural period of buildings
$\bar{V}_{BY}$	Design base shear force for earthquake shaking considered along Y-direction in plan, which is estimated using the empirical expression given in IS 1893 (Part 2) for approximate fundamental natural period of buildings
V_i	Design lateral storey shear force in storey *i*
V_{ik}	Design peak shear force acting in storey *i* in mode k of oscillation of the building
W	Seismic weight of the structure
W_i	Seismic weight of floor *i* of the building
w_{ds}	Width of equivalent diagonal strut generated in URM infill walls without any opening
Z	Earthquake zone factor, which reflects the mean horizontal peak ground acceleration (PGA) corresponding to a return period T_{RP} (years) and the earthquake zone in which the structure lies
Δ_{ave}	Average displacement between the ends of the diaphragm
Δ_{max}	Largest horizontal displacement of the floor slab in plan of a building in the direction of the lateral force at one end of the floor slab
Δ_{middle}	Lateral displacement at the centre of the diaphragm (measured from the undeformed position of the slab)
Δ_{min}	Smallest horizontal displacement of the floor slab in plan of a building in the same direction of the lateral force at the other end of the same floor slab than that where the displacement is the largest
Δ_1	Storey lateral displacement of building 1 or unit 1 of the same building
Δ_2	Storey lateral displacement of building 2 or unit 2 of the same building
$\Delta_{X,min}$	Minimum deformation capacity to be provided between the ends of a

	Displacement-sensitive AEUs for earthquake shaking considered along X-direction
Δ_{X1}	Design displacements along X direction of the structure at height h_1 at level 1 from its base
Δ_{X2}	Design displacements along X direction of the structure at height h_2 at level 2 from its base
Δ_{XA1}	Design displacements along X direction of structure A at height h_1 at level 1 from its base
Δ_{XA2}	Design displacements along X direction of structure A at height h_2 at level 2 from its base
Δ_{XB1}	Design displacements along X direction of structure B at height h_1 at level 1 from its base
Δ_{XB2}	Design displacements along X direction of structure B at height h_2 at level 2 from its base
$\Delta_{Y,min}$	Minimum deformation capacity to be provided between the ends of a Displacement-sensitive AEUs for earthquake shaking considered along Y-direction
Δ_{Y1}	Design displacements along Y direction of the structure at height h_1 at level 1 from its base
Δ_{Y2}	Design displacements along Y direction of the structure at height h_2 at level 2 from its base
Δ_{YA1}	Design displacements along Y direction of structure A at height h_1 at level 1 from its base
Δ_{YA2}	Design displacements along Y direction of structure A at height h_2 at level 2 from its base
Δ_{YB1}	Design displacements along Y direction of structure B at height h_1 at level 1 from its base
Δ_{YB2}	Design displacements along Y direction of structure B at height h_2 at level 2 from its base
Δ_{Z1}	Design displacements along Z direction of the structure at height h_1 at level 1 from its base
Δ_{Z2}	Design displacements along Z direction of the structure at height h_2 at level 2 from its base
Δ_{ZA1}	Design displacements along Z direction of structure A at height h_1 at level 1 from its base
Δ_{ZA2}	Design displacements along Z direction of structure A at height h_2 at level 2 from its base
Δ_{ZB1}	Design displacements along Z direction of structure B at height h_1 at level 1 from its base
Δ_{ZB2}	Design displacements along Z direction of structure B at height h_2 at level 2 from its base
δ	Drift ratio specified for buildings
φ_{ik}	Mode shape coefficient at floor *i* in mode k of oscillation of the building
θ	Angle of the diagonal strut with the horizontal
τ	Ratio of Natural Periods and of Fundamental Torsional and Fundamental Translational Modes of oscillation of the building
ξ	Damping ratio
ψ	Torsional flexibility factor of a building

When other symbols are used, they are explained at the appropriate place. Unless otherwise specified, all dimensions are in millimetres (*mm*), force in Newton (*N*), stresses in Mega Pascal (*MPa*) and time in seconds (*s*).

5 GENERAL PROVISIONS

The four main desirable attributes of an earthquake resistant building are:

a) Robust structural configuration,
b) At least a minimum initial elastic lateral stiffness,
c) At least a minimum overall lateral strength, and
d) Adequate ductility.

5.1 Irregularities

Buildings with simple regular geometry and uniformly distributed mass and stiffness in plan and in elevation, suffer much less damage, than buildings with irregular configurations. All efforts shall be made to eliminate irregularities by modifying architectural planning and structural configurations.

The irregularities considered in this standard are presented in Table 1.

A building shall be considered to be irregular for the purposes of this standard, if any of the conditions in **5.1.1** to **5.1.5** are applicable. Also, **5.1.1** to **5.1.5** specify requirements to address the irregularities.

Table 1 Irregularities in Buildings
(*Clause* 5.1)

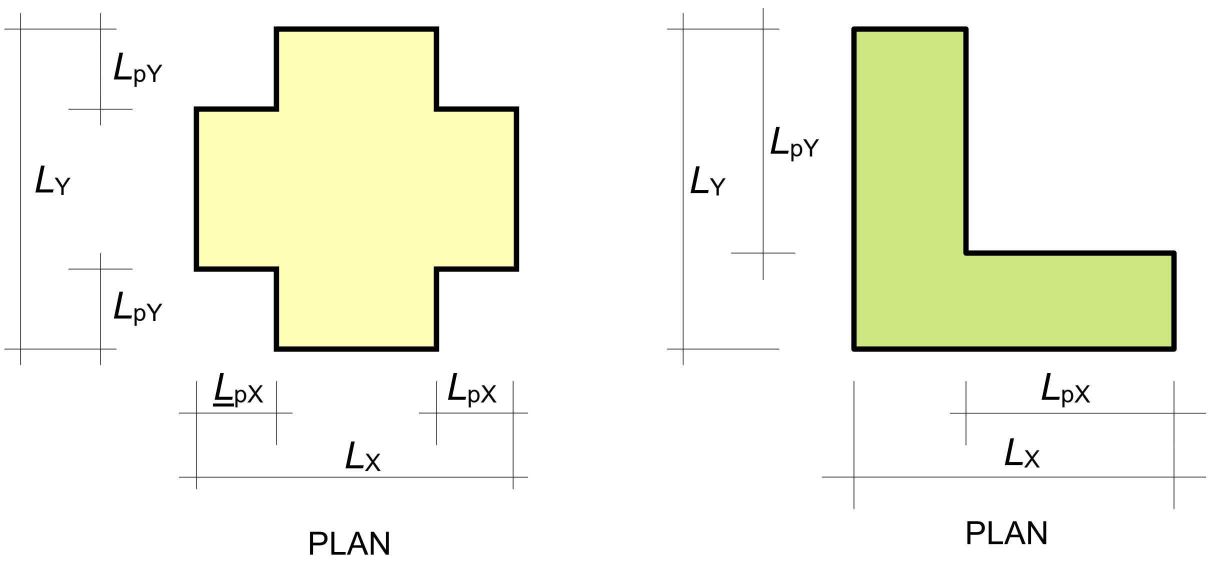
<table>
<tr><th>Sl No.</th><th>Irregularity Type</th><th>In Plan</th><th>In Elevation</th></tr>
<tr><td>(1)</td><td>(2)</td><td>(3)</td><td>(4)</td></tr>
<tr><td>i)</td><td>Geometry</td><td>Re-entrant corners</td><td>Vertical geometric irregularity</td></tr>
<tr><td>ii)</td><td>Mass</td><td>Horizontal mass irregularity</td><td>Vertical mass irregularity</td></tr>
<tr><td rowspan="3">iii)</td><td rowspan="3">Stiffness</td><td>Non-parallel lateral force resisting system</td><td>Soft storey</td></tr>
<tr><td>Floor slabs with excessive cut-outs or openings</td><td>In-plane discontinuity in vertical elements resisting lateral force</td></tr>
<tr><td>Out-of-Plane Offsets in Vertical Elements Design to Resist Lateral Forces</td><td>Floating Columns</td></tr>
<tr><td>iv)</td><td>Strength</td><td>-</td><td>Weak storey</td></tr>
<tr><td rowspan="5">v)</td><td rowspan="5">Behaviour</td><td colspan="2">Torsional flexibility</td></tr>
<tr><td colspan="2">Torsional irregularity</td></tr>
<tr><td colspan="2">Flexible floor diaphragm</td></tr>
<tr><td colspan="2">Closely spaced modes</td></tr>
<tr><td colspan="2">Irregular modes of oscillation in two principal plan directions</td></tr>
</table>

5.1.1 *Geometry*

5.1.1.1 *Irregularities in plan* — Re-entrant corners

A building is said to have a re-entrant corner in any plan direction, when its structural configuration in plan has a projection of size greater than 15 percent of its overall plan dimension in that direction (Fig. 1).

In a building with re-entrant corners, three-dimensional dynamic analysis method with flexible floor diaphragm shall be adopted to capture the concentration of forces generated in the re-entrant corners especially *in the floor diaphragm and special elements adjoining the re-entrant corner. This is in addition to the case of rigid diaphragm analysis, if applicable, and the worst effect considered.*



$L_{pX}/L_X > 0.15$
$L_{pY}/L_Y > 0.15$

FIG. 1 RE-ENTRANT CORNERS IN BUILDINGS

5.1.1.2 *Irregularities in elevation* — Vertical Geometric Irregularity

A building is said to have vertical geometric irregularity, when the horizontal projected length L_p of the irregular part of the lateral force resisting system is more than the fraction of the overall horizontal dimension L of the lateral force resisting system at the base of the building, as shown in Fig. 2 depending on the overall structural configuration of the building.

In a building with vertical geometric irregularity and located in Earthquake Zones III, IV, V and VI, the earthquake *effects shall be estimated by Dynamic Analysis Method (as per* **5.3.1***).*

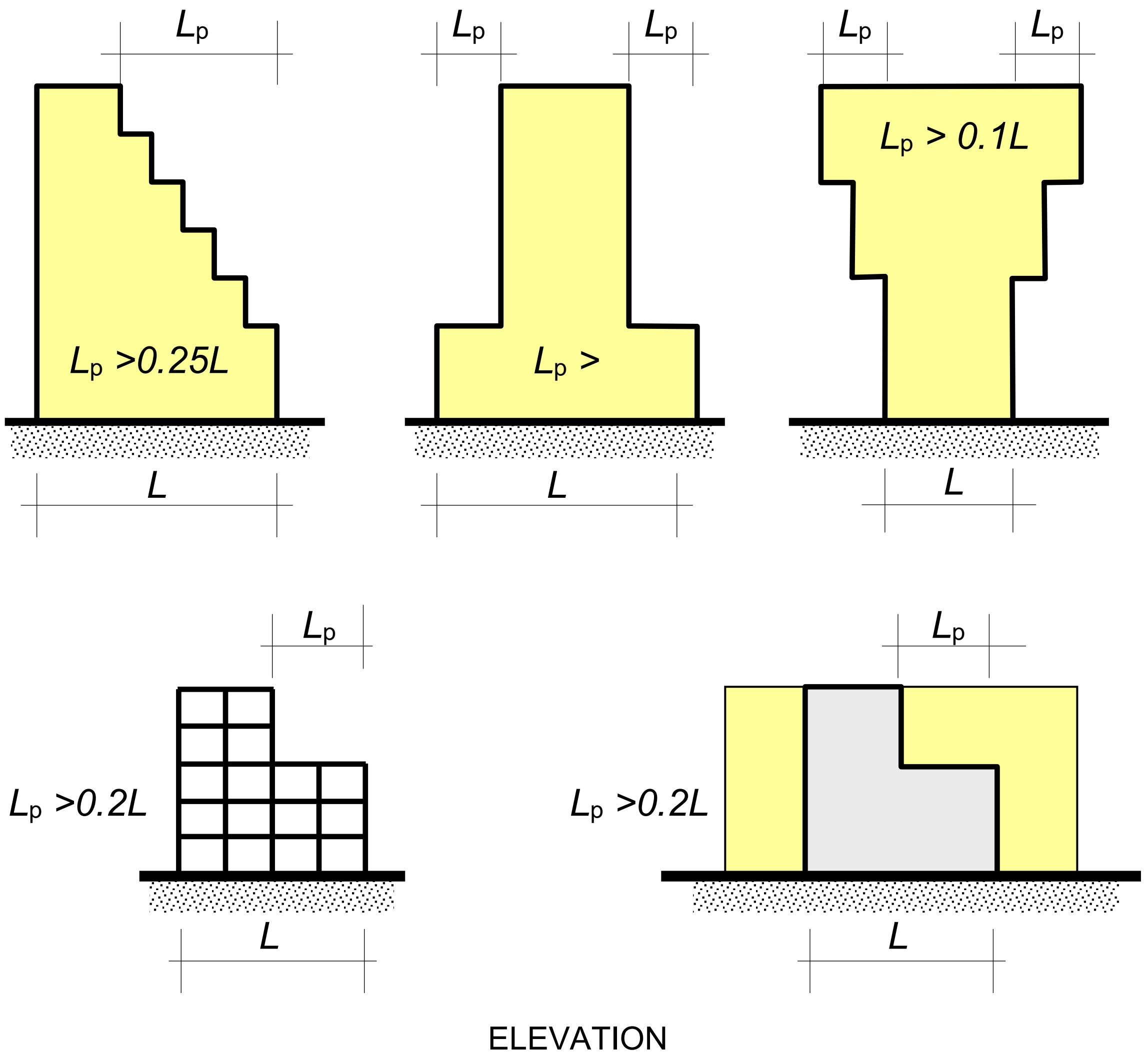


FIG. 2 VERTICAL GEOMETRIC IRREGULARITIES IN BUILDINGS

5.1.2 *Mass*

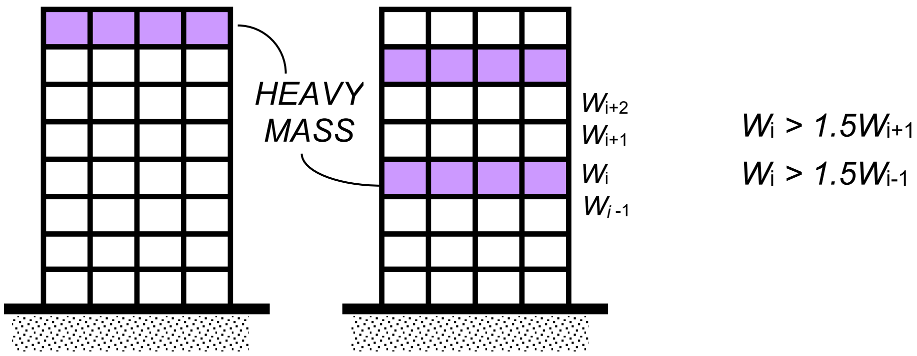
5.1.2.1 *Irregularities in plan*

The mass irregularity in plan is addressed through the design eccentricity at each floor in **5.2.5**, which accounts for both mass and stiffness irregularity in plan at each floor level.

5.1.2.2 *Irregularities in elevation* — Mass irregularity

A building is said to have mass irregularity, when the seismic weight *(as per* **5.2.2.2***)* of any floor is more than 150 percent of that of the floors below (Fig. 3).

In a building with mass irregularity and located in Earthquake Zones III, IV, V and VI, the earthquake effects shall be estimated by dynamic analysis (as per **5.3.1***).*



ELEVATION

FIG. 3 MASS IRREGULARITIES IN BUILDINGS

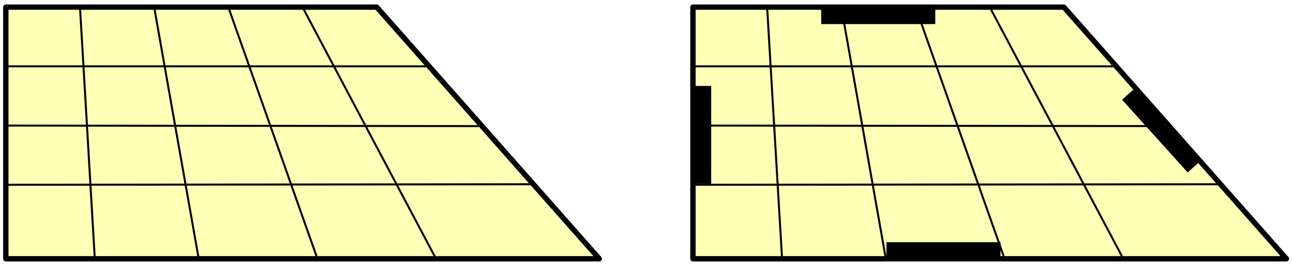
5.1.3 *Stiffness*

5.1.3.1 *Irregularities in plan*

a) **Non-Parallel Lateral Load Resisting System**

A building is said to have non-parallel lateral load resisting system when the vertical structural elements resisting the lateral loads are not oriented along the two principal orthogonal axes in plan (Fig. 4). Buildings undergo complex earthquake behaviour and hence damage, when they do not have lateral load resisting systems oriented along two plan directions that are orthogonal to each other.

In a building with non-parallel lateral load resisting system, the building shall be analyzed for load combinations *mentioned in* ***7.5*** *of IS 1893 Part 1.*



PLAN PLAN

4A MOMENT RESISTING FRAME BUILDING

4B MOMENT RESISTING FRAME BUILDING WITH STRUCTURAL WALLS

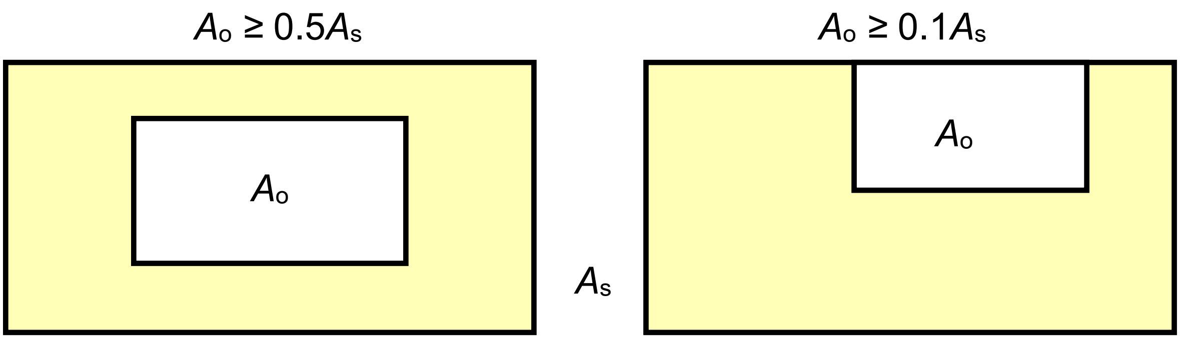
FIG. 4 NON-PARALLEL LATERAL LOAD RESISTING SYSTEM IN BUILDINGS

b) Floor Slabs with Excessive Cut-Outs or Openings

A building is said to have discontinuity in their in-plane stiffness, when floor slabs have cut-outs or openings of area A_o , 50 percent or more of the full area A_s of the floor slab (Fig. 5). Openings in slabs can result in flexible diaphragm behaviour. Lateral shear force is not shared by the vertical members in proportion to their lateral translational stiffness, especially when the opening is close to the edge of the slab.

In a building with floor slabs having excessive *cut-outs or openings, if the area of the geometric cut-out is:*

1) Less than or equal to 50 percent, the floor slab shall be modelled with appropriate structural elements in structural analysis of the building, which reflect its in-plane stiffness (partial flexibility) depending on the location of and size of openings; and
2) More than 50 percent, the floor slab shall be taken as flexible in-plane in structural analysis of the building.



PLAN PLAN

5A OPENING LOCATED ANYWHERE IN THE SLAB

5B OPENING LOCATED ALONG AN EDGE OF THE

FIG. 5 FLOOR SLABS IN BUILDINGS HAVING EXCESSIVE CUT-OUT AND OPENINGS

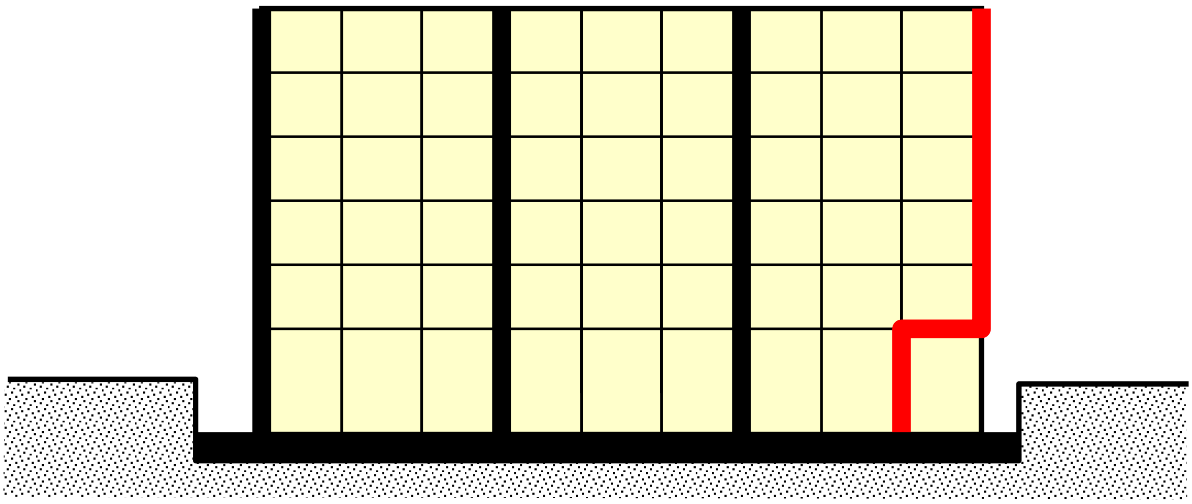
c) Out-of-Plane Offsets in Vertical Elements Design to Resist Lateral Loads

A building is said to have out-of-plane offset in vertical elements, when structural walls or frames are moved out of plane in any storey along the height of the building (Fig. 6). Out-of-plane offsets in vertical elements resisting lateral loads cause discontinuities and detours in the load path, which is known to be detrimental to the earthquake safety of the building.

In a building with *out-of-plane offsets in vertical elements design to resist lateral forces,*

1) *Specialist literature shall be referred for design of such a building, if the building is located in Earthquake Zone II; and*
2) *Both of the following two conditions shall be satisfied, if the building is located in Earthquake Zones III, IV, V and VI:*

   i) *The forces and moments due to earthquake effects in the elements connecting the two vertical* elements *with out of plane offset elements, the vertical element supporting the offset, and connections shall be designed for earthquake effects enhanced by a factor of at least 2.5; and*
   ii) *Lateral drift shall be less than* 0.2 *percent in the storey having the offset and also in the storeys below.*



ELEVATION

FIG. 6 OUT-OF-PLANE OFFSETS IN VERTICAL ELEMENTS IN BUILDINGS

5.1.3.2 *Irregularities in elevation*

a) Soft Storey

A building is said to have a soft storey, when lateral stiffness of a storey is less than that of the storey above (Fig. 7). This provision is not applicable to such storeys, which have:

1) lower height, where services and utilities are placed, or
2) outrigger frame members placed in them.

In a building with stiffness irregularity, the following shall be complied with:

i) Dynamic analysis shall be employed to capture the actual distribution of lateral stiffness along the height of the building; and

ii) The inter-storey drift shall be limited to 0.2 percent in that storey and all storeys below, if any, with stiffness irregularity.

iii) Also, in buildings with URM infills, when contribution of masonry infills exceeds 20 percent in the structural plan density (SPD) of the building, the effect of URM infills shall be considered by explicitly modeling the same in structural analysis. In buildings with stiffness irregularity arising out of URM Infills, provisions of ***5.3.2.3*** *shall apply.*

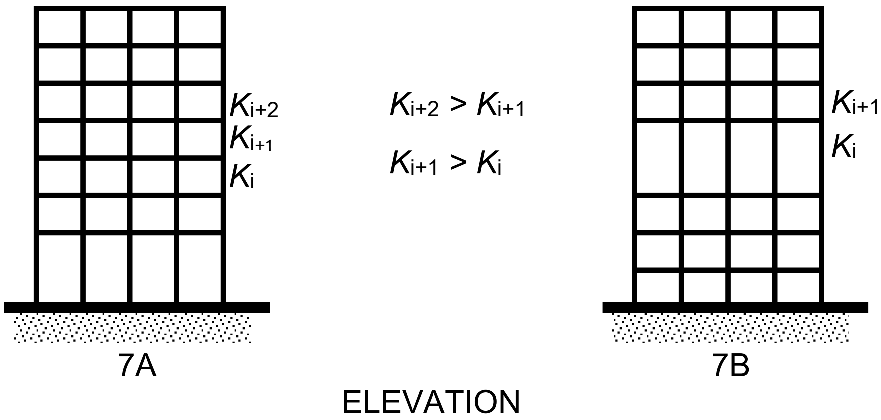


FIG. 7 SOFT STOREY IN BUILDINGS

b) In-Plane Discontinuity in Vertical Elements Resisting Lateral Force

A building is said to have an in-plane discontinuity in vertical elements resisting lateral force, when in-plane offset of the lateral force resisting elements is greater than 20 percent of the plan length of those elements (Fig. 8).

In a building with in-plane discontinuity and located in Earthquake Zone II, the lateral drift of the building under the design lateral force shall be limited to 0.2 percent of the building height. Buildings with in-plane discontinuity shall not be permitted in Earthquake Zones III, IV, V and VI.

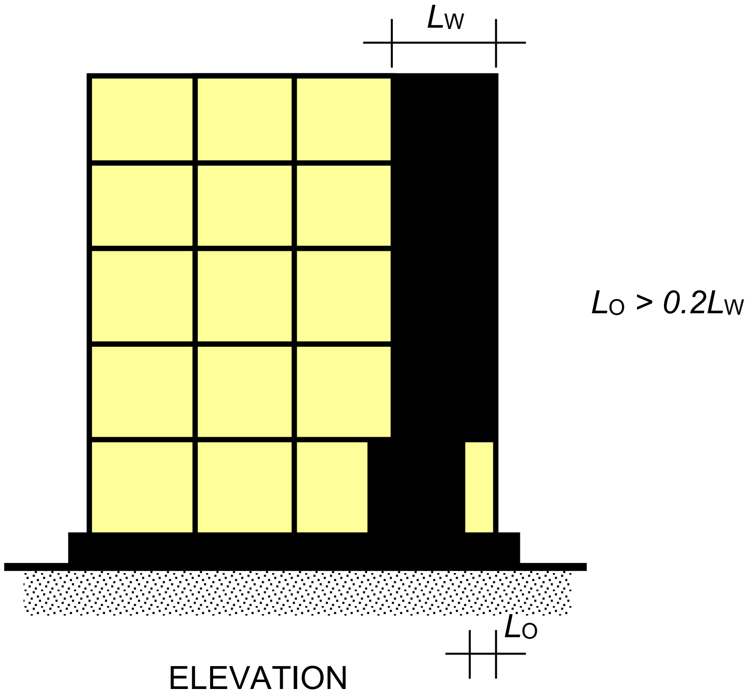


FIG. 8 IN-PLANE DISCONTINUITY IN VERTICAL ELEMENTS RESISTING LATERAL FORCE IN BUILDINGS

c) Floating Columns

A building is said to have a floating column, when any column terminates on a beam at an elevation above the base level of the building (Fig. 7B). Such columns are likely to cause concentrated damage in the structure, and are undesirable.

A building with floating columns shall not be permitted, if the floating columns are part of or supporting the primary lateral load resisting *system.*

5.1.4 *Strength*

5.1.4.1 *Irregularities in plan*

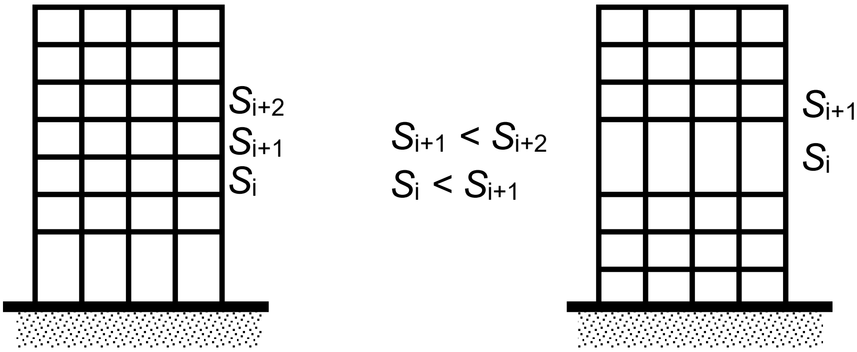
Currently, the standard does not have a provision.

5.1.4.2 Irregularities in Elevation

a) Weak Storey

A building is said to have weak storey, when the lateral strength of a storey is less than that of the storey above.

*Buildings with strength irregularity shall not be permitted. In case, the weak storey is because of URM infills, provisions of **5.3.2.3** shall be followed.*



8A 8B

ELEVATION

FIG. 8 WEAK STOREY IN BUILDINGS

5.1.5 *Behaviour*

5.1.5.1 *Closely spaced modes*

A building is said to have closely spaced modes, when fundamental lateral translational natural periods of the building in the two principal plan directions are within 10 percent of the larger of the two natural periods.

Buildings with closely spaced modes shall not be permitted. The structural elements of the members shall be re-*proportioned to separate the two modes.*

5.1.5.2 *Torsional Flexibility*

A building is said to be flexible torsionally, when the natural period corresponding to the fundamental torsional mode of oscillation is more than those of the first two translational modes of oscillation along each principal plan direction.

Buildings with torsional flexibility shall not be permitted. The structural elements of the members shall *be re-proportioned to make a lateral translational mode the fundamental torsional mode separate the two modes.*

5.1.5.3 *Irregularities in plan*

a) Torsional Irregularity

A building is said to be torsionally irregular, when:

1) The largest horizontal displacement $\Delta_{\max}$ of the floor slab in plan of a building in the direction of the lateral force at one end of the floor slab is more than 1.5 times $\Delta_{\min}$, the smallest horizontal displacement of the floor slab in plan of a building in the same direction of the lateral force at the other end of the same floor slab (Fig. 9); and

2) The building is torsionally flexible as per **5.1.5.2**.

Usually, a well-proportioned building does not twist about its vertical axis, when,

1) The stiffness distribution of the vertical elements resisting lateral loads is balanced in plan according to the distribution of mass in plan (at each storey level); and

2) The floor slabs are stiff in their own plane (this happens when its plan aspect ratio is less than 3).

In a building with torsional irregularity, when Δ_{max} is:

a) In the range of $1.2\Delta_{\text{ave}}$ to $1.4\Delta_{\text{ave}}$ [where $\Delta_{\text{ave}} = (\Delta_{\text{max}} + \Delta_{\text{min}}/2)$], then:

1) the building configuration shall be revised to ensure that the natural period of the fundamental torsional mode of oscillation shall be smaller than those of the first two translational modes along each of the principal plan directions, and

2) three dimensional dynamic analysis method shall be adopted; and

b) More than $1.4\Delta_{\text{ave}}$, the structural configuration of the building shall be revised.

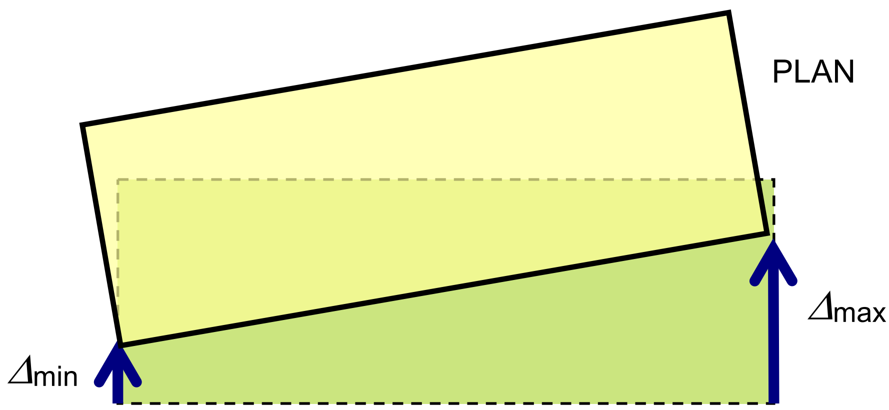
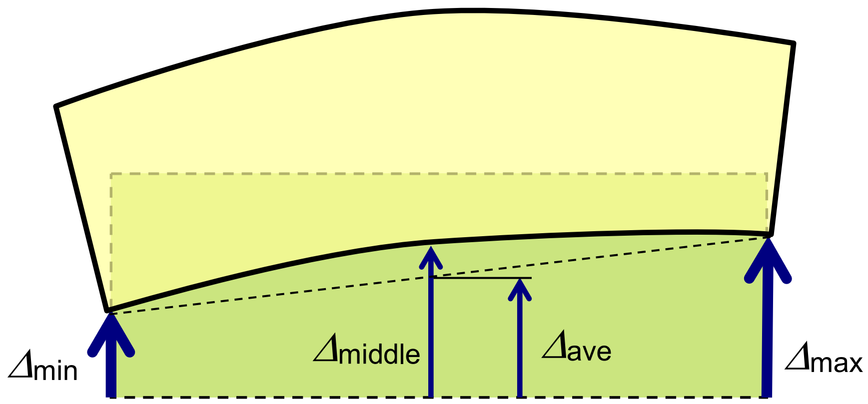


FIG. 9 TORSIONAL IRREGULARITY IN BUILDINGS

b) Flexible Floor Diaphragm

A building is said to have a flexible floor diaphragm, if its floor diaphragm deforms such that the lateral displacement Δ_{middle} at the centre of the diaphragm (measured from the undeformed position of the slab) is more than 1.2 times the average displacement Δ_{ave} between the ends of the diaphragm (Fig. 10).

In a building with flexible floor diaphragm, the earthquake effects shall be estimated using three-dimensional models of the buildings considering flexibility of the floor diaphragm.



$$\Delta_{\text{middle}} > 1.2\Delta_{\text{ave}}$$
where $\Delta_{\text{ave}} = (\Delta_{\text{max}} + \Delta_{\text{min}}/2)$

PLAN

FIG. 10 FLEXIBLE FLOOR DIAPHRAGMS IN BUILDINGS

c) Flexible Floor Diaphragm

In buildings whose floor diaphragms cannot provide rigid horizontal diaphragm action in their own plane, design storey shear shall be distributed to the various vertical elements of lateral force resisting system considering the in-plane flexibility of the diaphragms. Usually, reinforced concrete monolithic slab-beam floors or those consisting of prefabricated or precast elements with reasonable reinforced screed concrete (at least a minimum of 50 mm on floors and of 75 mm on roof, with at least a minimum reinforcement of 6 mm bars spaced at 150 mm centers) as topping, and of plan aspect ratio less than 3, can be considered to be providing rigid diaphragm action.

5.1.5.4 *Vertical irregularities*

a) Irregular Modes of Oscillation in Two Principal Plan Directions

A building is said to have lateral storey irregularity in a principal plan direction, if:

1) The first three lateral translational modes contribute less than 65 percent mass participation factor in each principal plan direction, and
2) The building has closely spaced modes as per **5.1.5.1.**

In a building with lateral irregularity in any of its two principal plan directions, and which is located in:

1) Earthquake Zones II and III, it shall be ensured that the first three lateral translational modes together contribute at least 65 percent mass participation factor in each principal plan direction.

2) Earthquake Zones IV, V and VI, it shall be ensured that:

i) the first three lateral translational modes together contribute at least 65 percent mass participation factor in each principal plan direction, and
ii) the fundamental lateral translational natural periods of the building in the two principal plan directions are away from each other by at least 10 percent of the larger value.

COMMENTARY

C-5.1.5.4 *Vertical irregularities*

a) Irregular Modes of Oscillation in Two Principal Plan Directions

Stiffness of beams, columns, braces and structural walls determine the lateral stiffness of a building in each principal plan direction.

5.2 Design Requirements

The design requirements related to stiffness, strength and deformability specified hereunder shall be complied with.

5.2.1 *Stiffness requirements*

The deformation of buildings under the design load combinations specified in **7.5.1** and **7.5.2** of IS 1893 (Part 1) corresponding to the Limit State of Strength and the Limit State of Serviceability, respectively, shall be obtained from structural analysis using a structural model based on section properties given in Table 1.

Table 1 Effective Second Moment of Area of Cross-Sections of Members, to be used in Structural Analysis of Buildings
(*Clause* 5.2.1)

Sl No.	***Member***	**Limit State of Strength**			**Limit State of Serviceability**		
		Normalized Effective Axial Area $A_{a,e}/A_{a,gross}$	*Normalized Effective Shear Area* $A_{s,e}/A_{s,gross}$	*Normalized Effective Second Moment of Area* I_e/I_{gross}	*Normalized Effective Axial Area* $A_{a,e}/A_{a,gross}$	*Normalized Effective Shear Area* $A_{s,e}/A_{s,gross}$	*Normalized Effective Second Moment of Area* I_e/I_{gross}
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i) Masonry Buildings							
a)	Slabs	1.0	1.0	0.35	1.0	1.0	0.50
b)	Beams	1.0	1.0	0.35	1.0	1.0	0.50
c)	Columns	1.0	1.0	0.70	1.0	1.0	0.90
d)	Walls	1.0	1.0	0.70	1.0	1.0	0.90
ii) Concrete Buildings							
a)	Slabs	1.0	1.0	0.35	1.0	1.0	0.35
b)	Beams	1.0	1.0	0.35	1.0	1.0	0.35

c)	Columns	1.0	1.0	0.70	1.0	1.0	0.70
d)	Walls	1.0	1.0	0.70	1.0	1.0	0.70
iii) Steel Buildings							
a)	Slabs	1.0	1.0	1.0	1.0	1.0	1.0
b)	Beams	1.0	1.0	1.0	1.0	1.0	1.0
c)	Columns	1.0	1.0	1.0	1.0	1.0	1.0
d)	Walls	1.0	1.0	1.0	1.0	1.0	1.0

5.2.1.1 *Lateral storey drift limitation*

The lateral storey drifts in buildings, under the load combinations corresponding to the Limit State of Serviceability specified in **7.5.2** and **7.5.3** of IS 1893 (Part 1), shall not exceed the values given in Table 2.

For estimating the above lateral storey drifts, the results of the Linear Structural Analysis, namely the Equivalent Static Method, or the Dynamic Method, mentioned in **7.6.2.1** or **7.6.2.2** of IS 1893 (Part 1), respectively, shall be used. In doing so, the displacement responses need not be amplified by the ratio mentioned in **7.6.2.3** of IS 1893 (Part 1).

Table 2 Limits on Lateral Storey Drifts in Buildings
(*Clause* 5.2.1.1)

Earthquake Zone	*Lateral Storey Drift* $\delta_{\text{storey}}/h_{\text{storey}}$
All Buildings	
II	0.0040
II	0.0040
IV	0.0030
V	0.0025
VI	0.0020

5.2.2 *Strength Requirements*

5.2.2.1 *Design horizontal earthquake base shear force*

The design horizontal earthquake base shear force is different for strength and serviceability designs.

a) Strength Design

The design horizontal acceleration coefficient $A_{\text{HD}}(T)$ along any principal direction of a building of a given structural system in a given earthquake zone and resting over a site of given category, and its design vertical acceleration coefficient $A_{\text{VD}}(T)$ for strength design (by Limit State Method) shall be estimated as:

$$A_{\text{HD}}(T) = \frac{A_{\text{H}}(T)}{R}, \text{ and } A_{\text{VD}}(T) = \frac{A_{\text{V}}(T)}{R},$$

where

$A_{\mathrm{H}}(T)$ = Elastic maximum horizontal PSA depending on the natural period *T* of the building of horizontal translational mode [as per **6.2.4.4** or **6.3.1** of IS 1893 (Part 1)], corresponding to the return period T_{RP} and the category of building specified in Table 1 of IS 1893 (Part 1) for strength design,

$A_{\mathrm{V}}(T)$ = Elastic maximum horizontal PSA depending on the natural period *T* of the building of horizontal translational mode [as per **6.2.4.4** or **6.3.1** of IS 1893 (Part 1)], corresponding to the return period T_{RP} and the category of building specified in Table 1 of IS 1893 (Part 1) for strength design,

R = Elastic force reduction factor *R* for buildings with different structural systems shall be taken as per Table 3. These values of *R* shall be used for design of buildings with said structural systems, and NOT for design of said structural systems built in isolation. Structural system referred to in Table 3 shall be as per Sections 2, 3 and 4 of this standard.

And, the design horizontal base shear force $V_{\mathrm{BD,H}}$ and design vertical base shear force $V_{\mathrm{BD,V}}$ along any principal direction of a building for strength design (by Limit State Method) shall be estimated as:

$V_{\mathrm{BD,H}} = A_{\mathrm{HD}}(T)\, W$, and

$V_{\mathrm{BD,V}} = A_{\mathrm{VD}}(T)\, W$,

where

$A_{\mathrm{HD}}(T)$ = Design Horizontal Acceleration Coefficient given above,
$A_{\mathrm{VD}}(T)$ = Design Vertical Acceleration Coefficient given above, and
W = Seismic Weight of the Structure as per **5.2.2.2**.

Table 3 Elastic Force Reduction Factor R for Buildings with different Structural Systems

[*Clause* 5.2.2.1(a)]

Sl No.	*Structural System*	*R*
1	**Buildings with moment frames**	
	a) Reinforced concrete buildings	
	Ordinary Moment Resisting Frames (OMRFs) (designed and detailed as per IS 456)	3.0
	Special Moment Resisting Frames (SMRFs) (designed and detailed as per IS 13920)	5.0
	b) Steel Buildings	
	Ordinary Moment Resisting Frames (OMRFs) (designed and detailed as per IS 800)	3.0
	Special Moment Resisting Frames (SMRFs) (designed and detailed as per IS 13920)	5.0
2	**Buildings with braced frames**	
	a) Steel buildings	
	Ordinary Concentrically Braced Frames (OCBFs) (designed and detailed as per IS 800)	3.0
	Special Concentrically Braced Frames (SCBFs) [designed and detailed as per Doc. No. CED 39(18640)WC]	4.5
	Eccentrically Braced Frames (EBFs) [designed and detailed as per Doc. No. CED 39(18640)WC]	5.5
3	**Buildings with structural walls**	
	a) Load-bearing masonry buildings	
	Unreinforced masonry (designed and detailed as per IS 1905) and provided *with* horizontal RC earthquake bands	1.5
	Unreinforced masonry (designed and detailed as per IS 1905) and provided *with* horizontal RC earthquake bands & vertical reinforcements	2.5
	Confined Masonry (designed and detailed as per IS 17848)	3.0
	Reinforced Masonry	3.0
	b) Concrete Buildings	
	Ordinary Structural Walls of RC (OSWs–RC) (designed and detailed as per IS 456)	3.0
	Special RC Structural Walls without boundary elements (SSWs–RC-NBE) (designed and detailed as per IS 13920)	4.5
	Special RC Structural Walls with boundary elements (SSWs–RC-BE) (designed and detailed as per IS 13920)	5.0
4	**Buildings with Moment Frames and Structural Walls**	
	OSWs–RC and OMRFs–RC (designed and detailed as per IS 456)	3.0
	SSWs–RC-NBE and SMRFs–RC (both designed and detailed as per IS 13920)	4.0
	SSWs–RC-BE and SMRFs–RC	5.0

	(both designed and detailed as per IS 13920)	
	Buildings with Dual Structural System	
	SSWs–RC-NBE and SMRFs–RC (both designed and detailed as per IS 13920)	5.5
	SSWs–RC-BE and SMRFs–RC (both designed and detailed as per IS 13920)	6.0
5	**RC Buildings with Flat Slab and Special RC Structural Walls**	
	RC building with features given below: a) SSWs–RC designed to resist 100 percent of design lateral force, b) Perimeter SMRFs–RC designed to resist independently 25 percent of design lateral force), and c) A system connecting SSWs–RC and perimeter SMRFs–RC (for example, outrigger belt truss system), in tall buildings	3.0

NOTES

In *RC Buildings with Flat Slab and Special RC Structural Walls*,

a) Punching shear failure shall be avoided under load combinations for strength design, and
b) Lateral drift at the roof under lateral force shall not exceed 0.1 percent.
Lateral drift shall be estimated:
1) using 3-dimensional building models, and considering total lateral displacement, including torsional effects, and
2) without necessarily scaling of displacement response quantities, as stated in **5.3.1.4.**

5.2.2 *Strength Requirements*

5.2.2.1 *Design horizontal earthquake base shear force*

a) Strength Design

Elastic Force Reduction Factor *R* reflects ductile inelastic actions in desirable and predetermined locations in buildings during strong earthquake ground shaking, and accounts for inherent system ductility, redundancy and overstrength available in buildings, if designed and detailed as per this standard and the associated Indian Standards.

b) Serviceability Check

The Horizontal Acceleration Coefficient $A_{\mathrm{HS}}(T)$ for Serviceability Check and Vertical Acceleration Coefficient $A_{\mathrm{VS}}(T)$ for Serviceability Check along any principal direction of a building of a given structural system in a given earthquake zone and resting over a site of given category, shall be estimated as:

$$A_{\mathrm{HS}}(T) = \frac{A_{\mathrm{H}}(T)}{R}, \text{ and } A_{\mathrm{VS}}(T) = \frac{A_{\mathrm{V}}(T)}{R},$$

where,

$A_{\mathrm{H}}(T)$ = Elastic maximum horizontal PSA depending on the natural period *T* of the building of horizontal translational mode [as per **6.2.4.4** or **6.3.1** of IS 1893 (Part 1)], corresponding to the return period T_{RP} and the category of building specified in Table 1 of IS 1893 (Part 1) for serviceability check,

$A_{\mathrm{V}}(T)$ = Elastic maximum vertical PSA depending on the natural period *T* of the building of horizontal translational mode [as per **6.2.4.4** or **6.3.1** of IS 1893 (Part 1)], corresponding to the return period T_{RP} and the category of building specified in Table 1 of IS 1893 (Part 1) for serviceability check, and

R = Elastic force reduction factor *R* for buildings with different structural systems shall be taken as per Table 3. These values of *R* shall be used for design of buildings with said structural systems, and NOT for design of said structural systems built in isolation. Structural system referred to in Table 3 shall be as per Sections 2, 3 and 4 of this standard.

And, the design horizontal base shear force $V_{\mathrm{BD,H}}$ and design vertical base shear force $V_{\mathrm{BD,V}}$ along any principal direction of a building for serviceability check shall be estimated as:

$$V_{\mathrm{BD,H}} = A_{\mathrm{HS}}(T)\,W,$$
$$V_{\mathrm{BD,V}} = A_{\mathrm{VS}}(T)\,W,$$

where

$A_{\mathrm{HS}}(T)$ = Horizontal Acceleration Coefficient given above,
$A_{\mathrm{VS}}(T)$ = Vertical Acceleration Coefficient given above, and
W = Seismic Weight of the Structure as per **5.2.2.2**.

5.2.2.2 *Seismic weight W*

a) **Seismic Weight of Floors**

Seismic weight of each floor shall be taken as its full dead load plus appropriate amount of design imposed load specified in IS 875 (Part 2), as specified in Table 4. This seismic weight shall be converted into seismic mass and used in the three-dimensional dynamic analysis of buildings also.

Table 4 Percentage of Imposed Load to be Considered in Estimation of Seismic Weight
[*Clause* 5.2.2.2(a)]

Sl No.	**Design Imposed Loads Distributed Uniformity on the Floor** (kN/m^2)	**Percentage of Imposed Load**
(1)	(2)	(3)
i)	Up to and including 3.0	25
ii)	Above 3.0	50

Further,

1) While computing the seismic weight of each floor, the weight of columns and walls in any storey shall be appropriately apportioned to the floors above and below the storey. Any other weight supported in between storeys

shall be distributed to floors above and below in inverse proportion to its distance from the floors.

2) In the calculation of design earthquake forces of buildings, imposed load need not be considered on roofs. But, weights of equipment and other permanently fixed facilities on roof shall be considered, but, the reductions of imposed loads mentioned in Table 4 shall not be applicable to that part of the load.
3) In the estimation of seismic weight, the reduction allowed in imposed load as per Table 4 alone shall be admissible, and not that allowed by IS 875 (Part 2).
4) Imposed Loads indicated in Table 4 for estimating seismic weight are applicable to normal conditions. When imposed loads during earthquakes are more accurately assessed, designers may alter or even replace the imposed loads given in Table 4 with actual assessed loads, such that the revised imposed loads are not less than those given in Table 4. In such cases, when imposed loads are assessed:
   i) Only that part of imposed load, which possesses mass, shall be considered; and
   ii) No impact factor shall be considered in the assessed imposed loads.
5) Snow loads and dust storm loads shall be considered appropriately. In regions of severe snow loads and sand storms exceeding intensity of 1.5 kN/m^2, 20 percent of uniform design snow load or sand load respectively, shall be included in the estimation of seismic weight; these values shall not be lesser than those given IS 875.
6) The seismic weight of partitions on floors shall be included in the estimation of seismic weight; it shall not be less than that:
   i) arising from a uniform loading on the floor of 0.5 kN/m^2, and
   ii) Specified for partitions in IS 875 (Part 2).

5.2.2.3 *Importance factor (I)*

In estimating design horizontal *and* vertical elastic PSA of buildings as per **6.2.4.4** or **6.3.1** of IS 1893 (Part 1), the importance factor *I* of buildings shall be taken as per Table 5.

Further,

a) Owners and design engineers of buildings or structures may choose values of Importance Factor *I* more than those specified in Table 5.
b) In Table 5, the occupancy size shall be for each of the structurally independent units of the building, when a building is composed of more than one structurally independent unit.
c) In buildings with mixed occupancies, wherein different *I* factors are applicable for the respective occupancies, larger of the applicable Importance Factors shall be used for estimating the design earthquake force of the building.

5.2.2.4 *Damping Ratio (ξ)*

The value of damping ratio ξ shall be taken as 5 percent of critical damping for the purposes of estimating design horizontal and vertical elastic PSA of buildings [as per

6.2.4.4 or **6.3.1** of IS 1893 (Part 1)], irrespective of:

a) The material of construction (namely steel, reinforced concrete, masonry, or a combination thereof of these three basic materials) of its lateral load resisting system, considering that buildings experience inelastic deformations under design level earthquake effects, resulting in much higher energy dissipation than that due to initial structural damping in buildings; and

b) The method of the structural analysis employed, namely Linear Equivalent Static Analysis Method (as per **5.3.1.1**) or Linear Dynamic Analysis Method (as per **5.3.1.3**).

Table 5 Importance Factor *I* for Buildings
(*Clause* 5.2.2.3)

Sl No.	**Category of Structures**	**Example Buildings**	**Importance Factor *I***
(1)	(2)	(3)	(4)
i)	*Normal Structures* (Design Return Period 475 Years) :: **Building Set 1**	Buildings that control and operate **Bridges Set 1**	1.0
		Buildings in **Industrial structures Set 1**	
		Buildings that control and operate **Liquid retaining structures Set 1**	
		Buildings that control and operate **Pipelines Set 1**	
		Buildings not listed below: a) Residential buildings with occupancy 100-200 persons b) Commercial buildings with occupancy 100-200 persons	1.15
ii)	*Important Structures* (Design Return Period 975 Years) :: **Building Set 2**	Residential buildings with occupancy more than 200 persons Commercial buildings with occupancy more than 200 persons Tall buildings Educational buildings Public buildings	1.0
		Buildings that control and operate **Bridges Set 2**	
		Buildings in **Industrial structures Set 2**	
		Buildings that control and operate **Liquid retaining structures Set 2**	
		Buildings that control and operate **Pipelines Set 2**	
iii)	*Critical and Lifeline Structures* (Design Return Period 2,475 Years) :: **Building Set 3**	Buildings of strategic importance Buildings and structures identified by Government of India and State Governments to be critical for governance or Disaster Management Hospital buildings Buildings that house the control or operations of utilities (for example, water, power, and sewage disposal), and lifeline (for example, communication and transportation facilities) structures Buildings related to space application	1.0
		Buildings that control and operate **Bridges Set 3**	
		Buildings in **Industrial structures Set 3**	
		Buildings that control and operate **Liquid retaining structures Set 3**	
		Buildings that control and operate **Pipelines Set 3**	
		Buildings that control and operate Dams other than the Specified Dams	

iv)	*Special Structures* (Design Return Period 4,995 Years) :: **Building Set 4**	Buildings identified by Government of India and State Governments to be of special importance	1.0
		Buildings used to in the Control or Operation of Dams classified as Specified Dams (as per *the Dam Safety Act*, 2021)	
		Buildings that control and operate **Bridges Set 4**	
		Buildings that control and operate Liquid Retaining Structures **Set 4**	
v)	*Nuclear Power Plant & Facility Structures* **(Design Return Period to be specified by the AERB)**	Buildings related to Nuclear Power Plants and Facilities	To be specified by the *Atomic Energy Regulatory Board (AERB), Government of India*

5.2.2.5 *Approximate fundamental translational natural period* T_a

The approximate fundamental translational natural period T_a of oscillation (in s), shall be estimated by the following expressions:

a) *Bare MRF buildings (without masonry infills):*

$$T_a = \begin{cases} 0.075h^{0.75} & \text{(for RC MRF building)} \\ 0.080h^{0.75} & \text{(for RC - Steel Composite MRF building)} \\ 0.085h^{0.75} & \text{(for Steel MRF building)} \end{cases},$$

where

h = Height (in m) of the building (Fig. 11); this excludes the basement storeys, where basement storey, walls are connected with the ground floor deck or fitted between the building columns, but includes the basement storeys, when they are not so connected.

b) *Buildings with RC Structural Walls (without or with masonry infills):*

$$T_a = \frac{0.075h^{0.75}}{\sqrt{A_w}} \leq \frac{0.09h}{\sqrt{d}},$$

where

A_w is total effective area (m^2) of walls in the first storey of the building given by:

$$A_w = \sum_{i=1}^{N_w}\left[A_{wi}\left\{0.2 + \left(\frac{L_{wi}}{h}\right)^2\right\}\right],$$

in which

h = Height (in m) of the building as defined in Fig. 11;
A_{wi} = Effective cross-sectional area (in m^2) of wall *i* in first storey of building;
L_{wi} = Length (in m) of structural wall *i* in first storey in the considered direction of lateral forces;
d = Base dimension (in m) of the building at the plinth level along the considered direction of earthquake shaking; and
N_w = Number of walls in the considered direction of earthquake shaking.

The value of L_{wi}/h to be used in this equation shall not exceed 0.9.

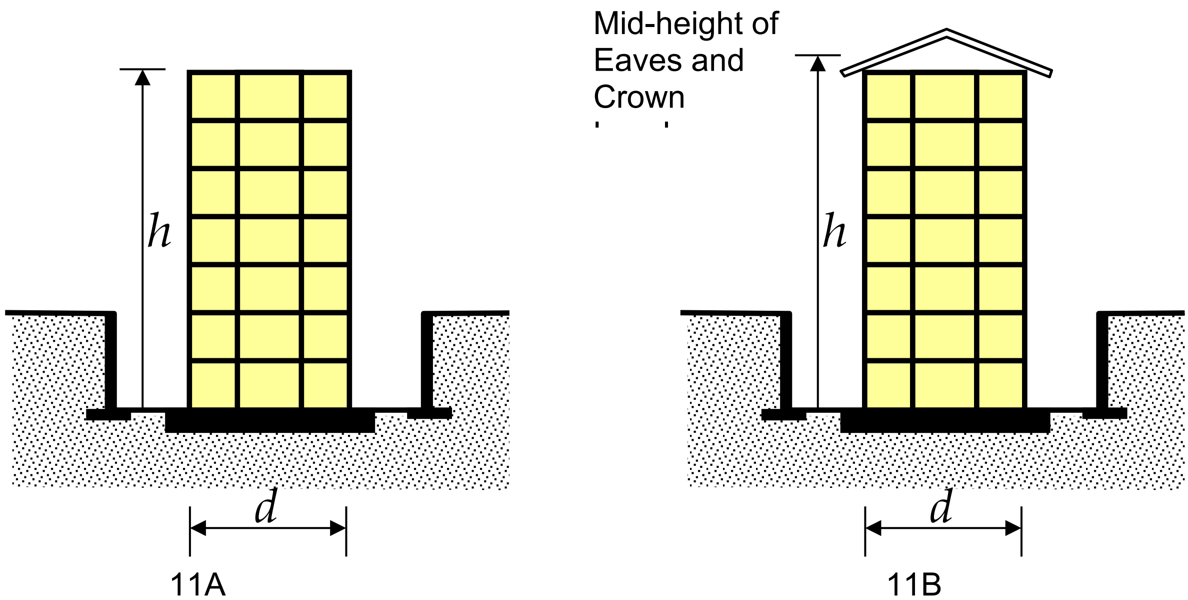
c) *All other Buildings:*

$$T_a = \frac{0.09h}{\sqrt{d}},$$

where

h = Height (in m) of building as defined in Fig. 11; and
d = Base dimension (in m) of the building at the plinth level along the considered direction of earthquake shaking; and

T_a obtained shall neither be taken to be more than that given in **5.2.2.5**(a) nor less than that given in **5.2.2.5**(c).



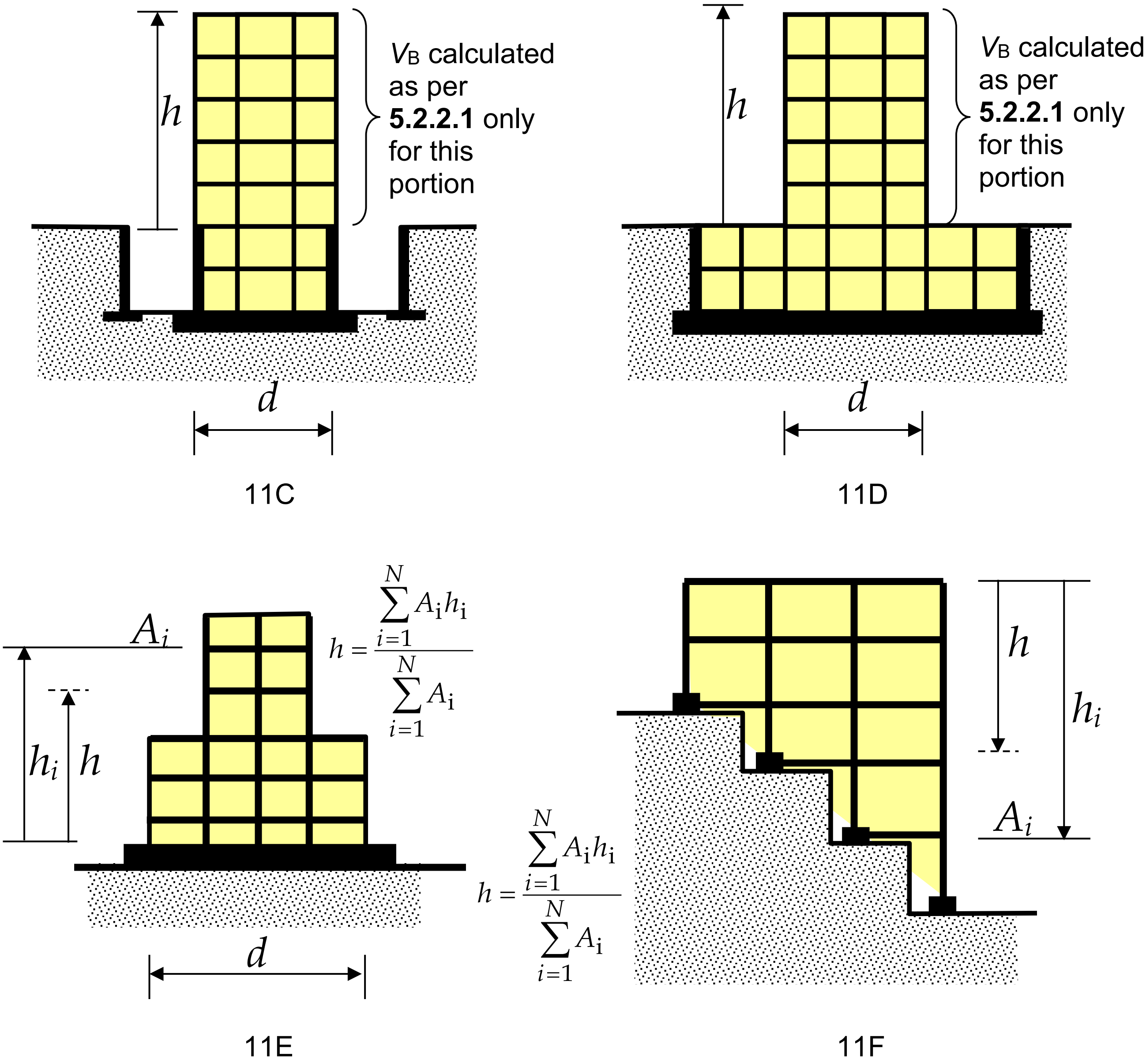


FIG. 11 HEIGHT h OF A BUILDING TO BE CONSIDERED IN THE ESTIMATION OF ITS T_A

5.2.3 *Minimum Design Horizontal Base Shear Force*

The design horizontal earthquake base shear force $V_{BD,H}$ obtained from **5.2.3.1**(a) for strength design shall not be taken less than $V_{BD,H,min}$ given by:

$$V_{BD,H,\min} = 0.625 \frac{ZI}{R} W ,$$

where

Z = Earthquake Zone Factor as per **6.2.2.3** of IS 1893 (Part 1);

I = Importance Factor as per **5.2.2.3** (*see* Table 5); and
R = Elastic force reduction factor as per Table 3.
W = Seismic weight as per **5.2.2.2**(a).

In no case, the value of $V_{\mathrm{BD,H,min}}$ shall be less than $0.015W$.

5.2.4 Distribution of Design Horizontal Base Shear Force

The design horizontal earthquake base shear force $V_{\mathrm{BD,H}}$ obtained from **5.2.2.1**(a) for strength design and from **5.2.2.1**(b) for serviceability check shall be distributed along the height of the building and in plan at each floor level as specified hereunder.

a) Distribution of Base Shear to Different Floor Levels

The design horizontal earthquake base shear force $V_{\mathrm{BD,H}}$ for strength design and for serviceability check shall be distributed along the height of the building as design storey lateral force Q_{i} at floor *i* as per:

$$Q_{\mathrm{i}} = \left(\frac{W_{\mathrm{i}} h_{\mathrm{i}}^2}{\sum_{j=1}^{N} W_{\mathrm{j}} h_{\mathrm{j}}^2}\right) V_{\mathrm{B}},$$

where

W_{i} = Seismic weight of floor *i*;
h_{i} = Height of floor *i* measured from base; and
N = Number of storeys in building, that is, levels at which masses are located.

b) Distribution of Design Storey Lateral Force Q_{i} at floor *i* to Different Lateral Force Resisting Elements at that Storey

The design storey shear force V_{i} in storey *i* for strength design and for serviceability check shall be taken as:

$$V_{\mathrm{i}} = \sum_{j=i}^{N} Q_{\mathrm{j}}.$$

In buildings whose floors are capable of providing rigid horizontal diaphragm action in their own plane, V_{i} can be distributed to the various vertical structural elements in proportion to their lateral stiffness. And, in buildings whose floors are NOT capable of providing rigid horizontal diaphragm action in their own plane, V_{i} shall be distributed to the various vertical structural elements in proportion to the masses from tributary floor areas.

5.2.5 *Torsion*

Provision shall be made in all buildings for increase in shear forces on the lateral force resisting elements resulting from twisting about the vertical axis of the building, arising due to eccentricity between the centre of mass (*CM*) and centre of resistance (*CR*) at the floor levels. The design forces calculated as in **5.2.4**(a) shall be applied at the displaced centre of mass so as to cause design eccentricity (as given by **5.2.5.1**) between the displaced *CM* and *CR*.

5.2.5.1 Design Eccentricity

While performing structural analysis by the Linear Equivalent Static Method of analysis or the Linear Response Spectrum Method of analysis for strength design and for serviceability check, the design eccentricity e_{di} to be used at floor *i* shall be taken as one of the following two values, whichever gives the more severe effect on lateral force resisting elements:

$e_{\mathrm{di,1}} = 1.8e_{\mathrm{si}} + 0.05b_{\mathrm{i}}$
$e_{\mathrm{di,2}} = -0.05b_{\mathrm{i}}$ for *Linear Equivalent Static Method,*

$e_{\mathrm{di,1}} = +0.05b_{\mathrm{i}}$
$e_{\mathrm{di,2}} = -0.05b_{\mathrm{i}}$ for *Linear Response Spectrum Method, and*

$e_{\mathrm{di,1}} = +0.05b_{\mathrm{i}}$
$e_{\mathrm{di,2}} = -0.05b_{\mathrm{i}}$ for *Linear Response History Method, and*

where

e_{si} = Static eccentricity at floor *i*
= Distance between centre of mass and centre of stiffness, and
b_{i} = Floor plan dimension of floor *i*, perpendicular to the direction of force.

COMMENTARY

C-5.2.5.1 The factor 1.8 represents dynamic amplification factor, and 0.05b_{i} represents the extent of accidental eccentricity.

5.2.5.2 *Torsional flexibility of buildings*

The Torsional Flexibility Factor ψ of a building shall be estimated as:

$$\psi = \left(\frac{e_K}{B}\right)\left(\frac{B}{r}\right)^2\left(\tau^2\right) = \left(\frac{e_K}{r_{K\theta}}\right)\left(\frac{B}{r_{K\theta}}\right),$$

where

B	=	Outer Dimension of the building along the direction to the considered direction of shaking,
e_K	=	Stiffness eccentricity of the building with respect to *CR*,
$\left(\frac{e_K}{B}\right)$	=	Normalized stiffness eccentricity of the building,
r	=	Translational radius of gyration of the mass of the building at the floor level,
$r_{K\theta}$	=	Torsional Radius of gyration of the mass of the building at

the floor level, and

τ = Ratio of Natural Periods of Fundamental Torsional and Fundamental Translational Modes of oscillation of the building, that is, T_θ / T_X or T_θ / T_Y for considered direction of shaking of X and Y, respectively.

If $\psi < 0.4$, the building is said to be torsionally stiff, else is termed as torsionally flexible.

a) Buildings with Rectangular Regular Plan Geometry

In torsionally stiff buildings, the analysis using the design eccentricity specified in **5.2.5.1** shall suffice, provided the values of τ and (e_K/B) are within the range specified in Table 6.

b) Buildings with Non-Rectangular Irregular Plan Geometry

In torsionally flexible buildings, the building shall be re-proportioned to ensure that $\psi < 0.4$.

Table 6 Design Limit for (e_K/B) and τ for Rectangular Regular Buildings
[*Clause* 5.2.5.2(a)]

<table>
<tr><th rowspan="2">Ratio τ of Natural Periods, T_θ/T_X or T_θ/T_Y</th><th colspan="5">Total Torsional Eccentricity (e_K/B)</th></tr>
<tr><th>e_K/B ≤ 0.05</th><th>0.05 < e_K/B ≤ 0.07</th><th>0.07 < e_K/B ≤ 0.10</th><th>0.10 < e_K/B ≤ 0.125</th><th>0.125 < e_K/B</th></tr>
<tr><td>τ ≤ 0.6</td><td colspan="4">Perform Torsional Analysis</td><td rowspan="4">Revise structural configuration to reduce e_K/B</td></tr>
<tr><td>0.6 < τ <0.7</td><td colspan="3">Perform Torsional Analysis</td><td rowspan="3">Revise structural configuration to ensure τ ≤ 0.6</td></tr>
<tr><td>0.7 < τ <0.8</td><td colspan="2">Perform Torsional Analysis</td><td rowspan="2">Revise structural configuration to ensure τ ≤ 0.7</td></tr>
<tr><td>0.8 < τ <0.9</td><td>Perform Torsional Analysis</td><td>Revise structural configuration to ensure τ ≤ 0.8</td></tr>
<tr><td>0.9 ≤ τ</td><td colspan="5">Not Permitted</td></tr>
</table>

COMMENTARY

C-5.2.5.2 *Torsional flexibility of buildings*

Buildings with poor structural configuration sustain large lateral-torsional response under earthquake shaking arising from torsional *eccentricity* e_K and torsional *flexibility.* The former, that is, distance between centers of mass (*CM*) and of resistance (*CR*), arises due to unsymmetric distribution in plan of mass, lateral stiffness and/or lateral strength of vertical structural elements at each storey; such buildings *twist* about a vertical axis during earthquake shaking – edges of floor slab translate laterally in plan by different amounts. And, the latter that is, the first mode is torsional mode with large fraction of total mass participating in it, arise due to floors *twisting* about a vertical axis without much translation in their *CMs*.

Simple idealised single-storey analytical models are used to identify the critical effects of torsion in buildings under earthquake shaking. Overall observations are validated through linear (ESA and LTHA) and nonlinear (NLPoA and NLTHA) analyses of multi-storey building. Earthquake design codes should limit normalized stiffness eccentricity e_K/B and torsional flexibility (with $\tau > 1$), and offer provisions to mitigate these negative effects of torsion. Together, they control the lateral displacements of structural elements on the edge of the building. Limits on critical parameters (e_K/B = *0.05 to 0.13* and τ = *0.60 to 0.90*, Table C1, for buildings having rectangular plan geometry) or geometric parameter *(*$\psi < 0.4$*,* Eq.(C1), for buildings with different plan geometry*)* are arrived at based on edge displacements using *elastic* torsional response. These limits help in ensuring that buildings have good earthquake-resistant structural configuration with almost regular geometric distribution of stiffness in plan, which avoids the problem of first mode of the building being the torsional mode (that is, $\tau > 1$). Lateral load analysis of buildings needs to be performed using *ESA or RSA* with the proposed DAF *(= 1.8 on flexible side and 1.0 on stiff side)*, or based on *LTHA* using $e_a/B = 0.05$*, and* then buildings designed and detailed. This would ensure buildings to perform well under strong earthquake shaking [Tamizharasi *et al*, 2021].

$$\psi = \left(\frac{e_K}{B}\right)\left(\frac{B}{r}\right)^2\left(\tau^2\right) = \left(\frac{e_K}{r_{K\theta}}\right)\left(\frac{B}{r_{K\theta}}\right) \qquad \text{(C1)}$$

r is the radius of gyration of mass about *CM;* $r_{K\theta}$, the radius of gyration of stiffness about *CM*; *B*, the building lateral dimension perpendicular to direction of earthquake shaking.

In the provision, Table C-1 has been simplified to have larger ranges of τ and (e_K/B).

Table C-1 Design Limit for e_K/B and *T* for Rectangular Regular Buildings

<table>
<tr><th rowspan="2">Natural Period Ratio τ</th><th colspan="7">Total Torsional Eccentricity (e_K/B)</th></tr>
<tr><th>≤ 0.05</th><th>0.05-0.07</th><th>0.07-0.08</th><th>0.08-0.10</th><th>0.10-0.11</th><th>0.11-0.13</th><th>≥ 0.13</th></tr>
<tr><td>≤0.60</td><td colspan="6">Torsional Analysis</td><td rowspan="4">Redesign to reduce e/B</td></tr>
<tr><td>0.60-0.65</td><td colspan="5">Torsional Analysis</td><td rowspan="3">Redesign to ensure τ ≤ 0.60</td></tr>
<tr><td>0.65-0.70</td><td colspan="4">Torsional Analysis</td><td rowspan="2">Redesign to ensure</td></tr>
<tr><td>0.70-0.75</td><td colspan="3">Torsional Analysis</td><td>Redesign to</td></tr>
</table>

0.75-0.80	Torsional Analysis		Redesign to ensure $\tau \leq 0.75$	ensure $\tau \leq 0.70$	$\tau \leq 0.65$		
0.80-0.90	Torsional Analysis	Redesign to ensure $\tau \leq 0.80$					
≥0.90	Not Permitted						

5.2.6 *Deformability Requirements*

Deformations in concrete buildings shall be obtained from structural analysis using a structural model based on section properties given in **5.3.2** and Table 1.

5.2.6.1 *Structure*

a) Separation between Adjoining Units

Two adjoining buildings or two adjoining units of the same building shall be separated (with a seismic joint between them) by a distance equal to:

1) $(\Delta_1 R_1 + \Delta_2 R_2)$, when the floor levels of the adjoining units of a building or buildings are at different levels, and
2) $(\Delta_1 R_1 + \Delta_2 R_2)/2$, when floor levels of the adjoining units of a building or buildings are at the same level,

where

Δ_1 = Storey lateral displacement of building 1 or unit 1 of the same building, estimated using the load combinations specified in **7.5** of IS 1893 (Part 1) for strength design;
Δ_2 = Storey lateral displacement of building 2 or unit 2 of the same building,
R_1 = Elastic Force Reduction Factor of building 1 or unit 1 of the same building, and
R_2 = Elastic Force Reduction Factor of building 2 or unit 2 of the same building.

Here, Δ_1 and Δ_2 are estimated using the load combinations specified in **7.5** of IS 1893 (Part 1) for strength design, and reduced cross-section properties specified in Table 1 for strength design.

COMMENTARY

C-5.2.6 *Deformability Requirements*

C-5.2.6.1 *Structure*

a) Separation between Adjacent Units

This provision is made to avoid pounding of the two adjoining buildings or of the two units of the same building, when they independently oscillate towards each other.

5.3 Structural Analysis

5.3.1 *Methods of Analysis*

In the strength design or serviceability check of new buildings, effects of design earthquake loads applied on them can be assessed in two ways, namely:

a) Linear Equivalent Static Method for analysis of regular structures with approximate natural period T_a less than 0.4 s, and
b) Linear Dynamic Analysis Method by:

    1) Response Spectrum Method, and
    2) Matrix Response History Method.

5.3.1.1 *Linear equivalent static analysis method*

The design base shear force V_{BD} shall be computed for the building as a whole. Then, this V_{BD} shall be distributed to the various floor levels at the corresponding centres of mass. And, finally, this design earthquake force at each floor level shall be distributed to individual lateral load resisting elements through structural analysis considering the floor diaphragm action.

This method shall be applicable for regular buildings with height less than 15 m in Earthquake Zone II.

5.3.1.2 *Simplified linear dynamic analysis method*

Regular buildings may be analyzed as a system of masses lumped at the floor levels with each mass having one degree of freedom, that of lateral displacement in the direction under consideration. In such a case, the following expressions can be used:

a) Modal Mass

Modal mass M_{k} of mode *k* shall be estimated as:

$$M_k = \frac{\left(\sum_{i=1}^{N}\left(W_i\phi_{ik}\right)\right)^2}{g\sum_{j=1}^{N}\left\{W_j\left(\phi_{jk}\right)^2\right\}},$$

where

W_i = Seismic weight of floor *i* of the building,

ϕ_{ik} = Mode shape coefficient at floor *i* in mode *k* ,
g = Acceleration due to gravity, and
N = Number of floors of the building.

b) Modal Participation Factor

Mode participation factor P_k of mode *k* shall be estimated as:

$$P_k = \frac{\sum_{i=1}^{N} W_i \phi_{ik}}{\sum_{j=1}^{N} \left\{ W_j \left(\phi_{jk} \right)^2 \right\}} .$$

c) Design Peak Lateral Force at each floor in each mode

Design peak lateral force Q_{ik} at floor *i* in mode *k* shall be estimated as:

$$Q_{ik} = A_{HD,k} \phi_{ik} P_k W_i ,$$

where

$A_{HD,k}$ = Design horizontal Elastic PSA as per **6.2.4.4** of IS 1893 (Part 1) using natural period of oscillation T_k of mode k obtained from free vibration analysis.

d) Design Peak Storey Shear Forces in Each Mode

Design peak shear force V_{ik} acting in storey i in mode k shall be estimated as:

$$V_{ik} = \sum_{j=i+1}^{N} Q_{jk} .$$

e) Design Peak Storey Shear Force Due to all Modes Considered

Design peak storey shear force V_{i} in storey *i* due to all modes considered shall be obtained by combining those due to each mode in accordance with **7.6.2.2**(a) of IS 1893 (Part 1).

f) Design Lateral Forces at each storey Due to all Modes Considered

Design lateral forces F_{roof} at roof level and F_{i} at level of floor *i* shall be obtained as:

$$F_{\mathrm{roof}} = V_{\mathrm{roof}} \text{, and}$$
$$F_{\mathrm{i}} = V_{\mathrm{i}} - V_{\mathrm{i+1}} .$$

5.3.1.3 *Linear dynamic analysis method*

a) Linear Response Spectrum Method

Refer **7.6.2.2**(a) of IS 1893 (Part 1).

b) Linear Response History Method

Refer **7.6.2.2**(b) of IS 1893 (Part 1).

5.3.1.4 *Protection of minimum base shear force from linear dynamic analysis methods*

The design base shears V_{BX} and V_{BY} estimated by Linear Dynamic Analysis Methods (for earthquake shaking along X- and Y-directions in plan, respectively) shall not be less than the corresponding design base shears $\overline{V}_{BX}$ and $\overline{V}_{BY}$ (for earthquake shaking along X- and y-directions in plan, respectively) estimated by Equivalent Linear Analysis Method (using approximate fundamental natural period T_a estimated as per **5.2.2.5**).

When V_{BX} is less than $\overline{V}_{BX}$ along X-direction in plan, then:

a) The stress resultants (axial force, shear force, bending moment and torsional moment) in all members, storey shear forces and base reactions, arising from the earthquake shaking along the X-direction in plan alone, shall be amplified by the ratio $\overline{V}_{\mathrm{BX}}/V_{\mathrm{BX}}$.
b) The deformations (for example, displacements, rotations and storey drifts) at all points in the structure, arising from the earthquake shaking along the X-direction in plan alone, need not be amplified by the said ratio.

Similarly, when V_{BY} is less than $\overline{V}_{BY}$ along Y-direction in plan, then:

a) The stress resultants (axial force, shear force, bending moment and torsional moment) in all members, storey shear forces and base reactions, arising from the earthquake shaking along the Y-direction in plan alone, shall be amplified by the ratio $\overline{V}_{\mathrm{BY}}/V_{\mathrm{BY}}$.
b) The deformations (for example, displacements, rotations and storey drifts) at all points in the structure, arising from the earthquake shaking along the Y-direction in plan alone, need not be amplified by the said ratio.

When V_{BX} is more than $\overline{V}_{BX}$ along X-direction in plan, then the said scaling need not be applied. Similarly, when V_{BY} is more than $\overline{V}_{BY}$ along Y-direction in plan, then the said scaling need not be applied.

5.3.2 *Modeling*

5.3.2.1 *Section properties*

For structural analysis, the second moment of are of cross-sections shall be taken as per Table 1.

5.3.2.2 *Material*

The modulus of elasticity *E* for concrete, steel and masonry members shall be taken as per IS 456, IS 800 and IS 1905.

5.3.2.3 *Effect of unreinforced masonry infill walls*

The effect of in-plane stiffness and strength of Unreinforced Masonry Infill (URM) walls shall be considered as per **6.3** of Section 2 of this standard.

5.4 Miscellaneous

5.4.1 *Buildings on Sloping Ground*

a) RC buildings on sloping ground (Fig. 12) shall be provided with RC walls as suggested hereunder, which are oriented along both principal plan directions, to minimize torsional irregularity.

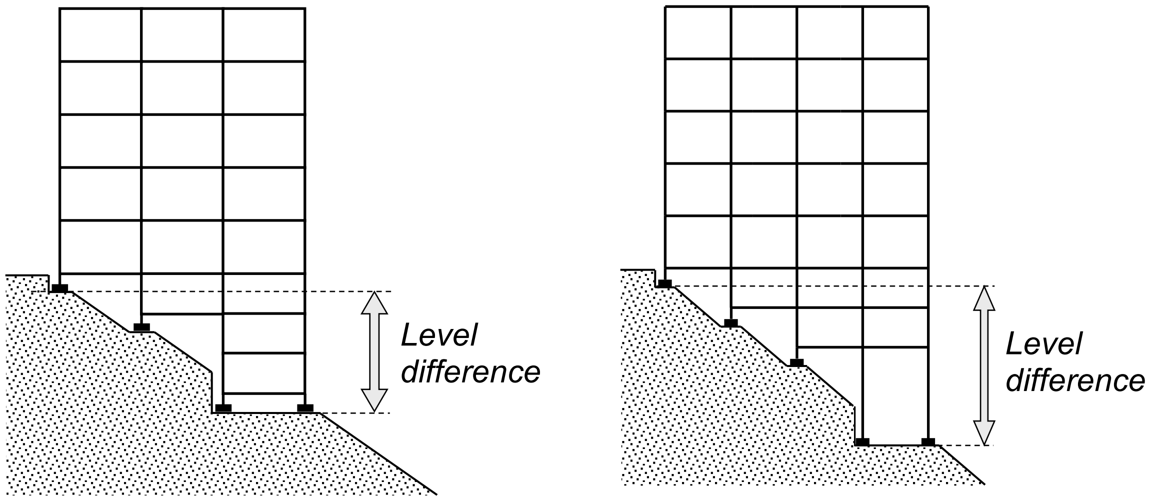
b) RC walls shall be provided (Fig. 13):

   1) perpendicular to the road, in the tallest bay on the downhill side of the two exterior frames along the entire height of the building; and

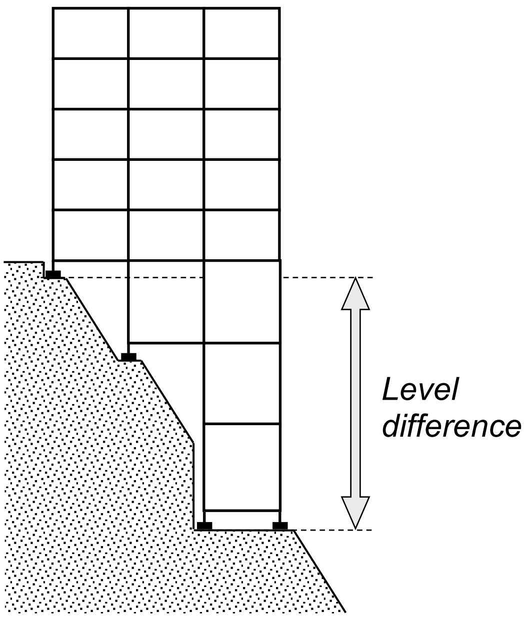
   2) parallel to the road, in the tallest frame on the down-hill side from the lowest column base level up to the highest column base level, and meet the following:

      i) These RC walls shall have configuration A, B or C (Table 7); and
      ii) These RC walls have a total cross-sectional area, such that the maximum horizontal displacement of the tallest frame at any floor, including roof, under the action of design horizontal force in the direction parallel to the road, is not more than 1.05 times that at the same floor in the shortest frame.

c) The design and detailing of the RC walls shall conform to the provisions of special shear walls in IS 13920.

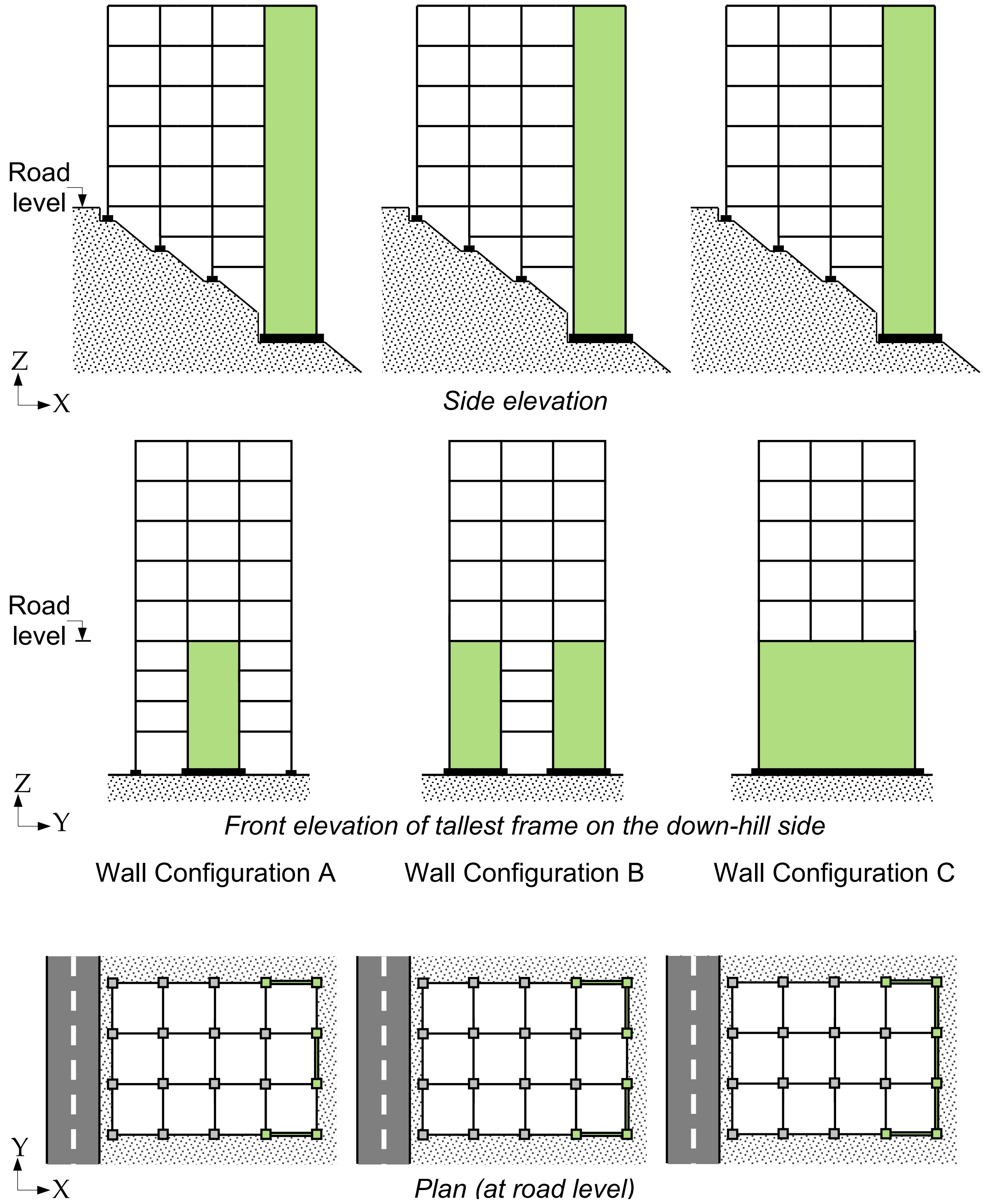


12A BUILDING ON SMALL SLOPE



12B BUILDING ON LARGE SLOPE

FIG. 12 TYPICAL ELEVATION OF STEP-BACK BUILDINGS: LEVEL DIFFERENCE BETWEEN TOP-MOST AND BOTTOM-MOST COLUMN BASE CAN BE CONSIDERED A QUANTITATIVE MEASURE OF TORSIONAL IRREGULARITY



13A
CONFIGURATION A

13B
CONFIGURATION B

13C
CONFIGURATION C

FIG. 13 SUGGESTED RC WALL CONFIGURATIONS IN BUILDINGS ON HILL SLOPES

Table 7 RC Wall Configurations in Step-Back Buildings on Hill Slopes
[*Clause* 5.4.1(b)]

Sl No.	***Difference between Highest and Lowest Column Base Levels***	***Wall configuration***
(1)	(2)	(3)
i)	≤ 10 m	A
ii)	10-15 m	B
iii)	> 15 m	C

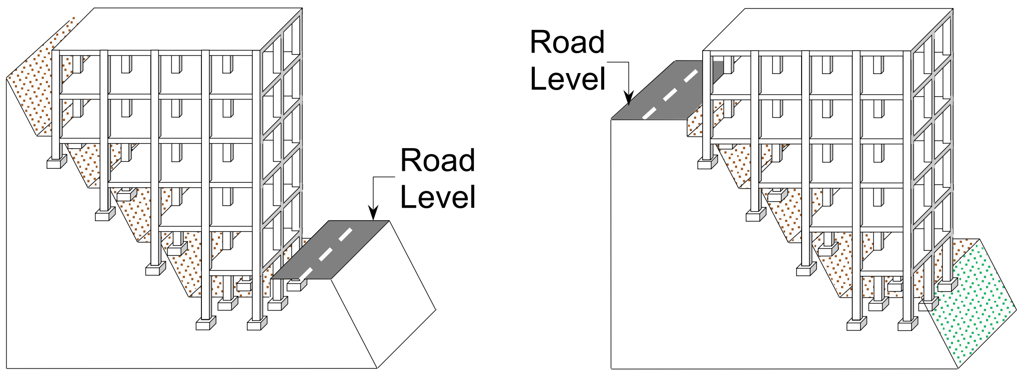
COMMENTARY

C-5.4 Miscellaneous

C-5.4.1 *Buildings on Sloping Ground*

Buildings on slope in hilly regions (Figure C-2) need special structural configuration to minimize the ill effects of stiffness irregularity in the form of having dominant torsional mode of oscillation about a vertical axis.

This standard provides clauses to address the stiffness irregularity through a structural solution for down-slope buildings.



C-2A UP-SLOPE BUILDING C-2B DOWN-SLOPE BUILDING

FIG. C-2 BROAD CLASSIFICATION OF BUILDINGS ON HILL SLOPE

5.4.2 *Foundations*

Isolated RC footings without tie beams or unreinforced strip foundations shall not be adopted in buildings rested on soft soils (Site Category D) in any Earthquake Zone. Use of foundations vulnerable to significant differential settlement due to ground shaking shall be avoided in buildings located in Earthquake Zones III, IV, V and VI.

Individual spread footings or pile caps shall be interconnected with ties (as per Section 3 of this standard, except when individual spread footings are directly supported on rock, in buildings located in Earthquake Zones IV and V. All ties shall be capable of carrying, in tension and in compression, an axial force equal to $A_{\mathrm{HD}}/4$ times the larger of the column or pile cap load, in addition to the otherwise computed forces, subject to a minimum of 5 percent of larger of column or pile cap loads, where A_{HD} is as per **6.2.4.4** of IS 1893 (Part 1).

Piles shall be designed and constructed to withstand maximum curvature imposed (structural response) by earthquake ground shaking. Design of anchorage of piles into the pile cap shall consider combined effects, including that of axial forces due to uplift and bending moments due to fixity to pile cap.

5.4.3 *Cantilever Projections*

Only the projecting parts and their connections with the main structures (and not the main building) shall be designed for the design forces based on increased Elastic Maximum PSA specified hereunder.

5.4.3.1 *Vertical projections*

Small-sized facilities (like towers, tanks, parapets, smoke stacks/chimneys) and other vertical cantilever projections attached to buildings and projecting vertically above the roof, but not a part of the structural system of the building, shall be designed and checked for stability for five times the Design Horizontal Elastic PSA A_{HD} (as per **6.2.4.4** of IS 1893 (Part 1)) for that building. In the analysis of the building, weights of these projecting elements shall be lumped with the roof weight.

5.4.3.2 *Horizontal projections*

All horizontal projections of buildings (like cantilever structural members at the porch level or higher) or attached to buildings (like brackets, cornices and balconies) shall be designed for five times the Design Vertical Elastic PSA A_{VD} (as per **6.2.4.4** of IS 1893 (Part 1)) for that building.

5.4.4 *Compound Walls*

Compound walls shall be designed for the design horizontal elastic PSA A_{HD} of $2.5Z$, where Z is design Earthquake Zone Factor taken as per **6.2.2.3** of IS 1893 (Part 1).

5.4.5 *Architectural elements and utilities*

The Design Relative Displacements Δ_X and Δ_Y along plan directions X and Y, respectively, at the two ends of a *D–AEU* estimated in **8.5.2(c)** and **(d)** shall not be less than the following:

a) *D–AEU* supported consecutively at two levels of the same structure, one at height h_1 and the other at height h_2 from base of the structure, shall be estimated as:

1) Equivalent Static Linear Analysis

$\Delta_{X,\min} = Max[1.2R_X(\Delta_{X1} - \Delta_{X2});\ 1.2R_X\delta(h_{X1} - h_{X2})]$, and
$\Delta_{Y,\min} = Max[1.2R_Y(\Delta_{Y1} - \Delta_{Y2});\ 1.2R_Y\delta(h_{Y1} - h_{Y2})]$, and

2) Linear Dynamic Analysis

$\Delta_{X,\min} = Max[R_X(\Delta_{X1} - \Delta_{X2});\ 1.2R_X\delta(h_{X1} - h_{X2})]$, and
$\Delta_{Y,\min} = Max[R_Y(\Delta_{Y1} - \Delta_{Y2});\ 1.2R_Y\delta(h_{Y1} - h_{Y2})]$
where

Δ_{X1}, Δ_{Y1} and Δ_{Z1} = Design displacements along X, Y and Z directions, respectively, of the structure at height h_1 at level 1 from its base,
Δ_{X2}, Δ_{Y2} and Δ_{Z2} = Design displacements along X, Y and Z directions, respectively, of the structure at height h_2 at level 2 from its base, or of the ground,
R_X = Elastic Force Reduction Factor of building for the lateral load resisting structural system employed along X-direction,
R_Y = Elastic Force Reduction Factor of building for the lateral load resisting structural system employed along Y-direction, and
δ = Drift limit specified for buildings in Table 2 of **5.2.1.1**; and

b) *D–AEU* supported:

1) At two levels of two different structures (A and B, say, even if one of them is an electric pole, or a communication antenna tower),
2) At two levels of two adjoining parts (A and B, say) of the same structure separated by a construction joint on which the *AEU* is supported, that is, at height h_1 on Structure A and at height h_2 on Structure B from bases of the respective structures, and
3) One end at a level on a structure and another on adjoining ground,

i) Equivalent Static Linear Analysis

$\Delta_{X,\min} = Max[|1.2R_{XA}\Delta_{XA1}| + |1.2R_{XB}\Delta_{XB2}|;\ 1.2R_{XA}\delta_A h_{X1} + 1.2R_{XB}\delta_B h_{X1}]$
, and
$\Delta_{Y,\min} = Max[|1.2R_{YA}\Delta_{YA1}| + |1.2R_{YB}\Delta_{YB2}|;\ 1.2R_{YA}\delta_A h_{Y1} + 1.2R_{YB}\delta_B h_{Y1}]$

ii) Linear Dynamic Analysis

$\Delta_{X,\min} = Max[R_{XA}|\Delta_{XA1}| + R_{XB}|\Delta_{XA2}|;\ 1.2R_{XA}\delta_A h_{X1} + 1.2R_{XB}\delta_B h_{X1}]$
, and
$\Delta_{Y,\min} = Max[R_{YA}|\Delta_{YA1}| + R_{YB}|\Delta_{YA2}|;\ 1.2R_{YA}\delta_A h_{Y1} + 1.2R_{YB}\delta_B h_{Y1}]$,

where

Δ_{XA1}, Δ_{YA1} and Δ_{ZA1} = Design displacements along X, Y and Z directions, respectively, of Structure A at height h_1 at level 1 from its base,

Δ_{XB2}, Δ_{YB2} and Δ_{ZB2} = Design displacements along X, Y and Z directions, respectively, of Structure B at height h_2 at level 2 from its base, or of the ground,

R_{XA} and R_{XB} = Elastic Force Reduction Factor of building for the lateral load resisting structural system employed along X-direction of buildings A and B, respectively,

R_{YA} and R_{YB} = Elastic Force Reduction Factor of building for the lateral load resisting structural system employed along Y-direction of buildings A and B, respectively;

δ = Drift limit specified for buildings in Table 2 of **5.2.1.1**, and

h_{X1} and h_{Y1} = Elevations of level 1 from the base of the building along X and Y directions, respectively,

SECTION 2

ADDITIONAL CRITERIA FOR MASONRY BUILDINGS

6 EARTHQUAKE DEMAND ON MASONRY BUILDINGS

6.1 Structural Systems

The following structural systems in load-bearing masonry walls shall be admissible to resist effects of earthquake shaking:

a) Masonry walls with:

1) Bands (MWB), and
2) Bands, and Horizontal and Vertical Reinforcements (MWBR),

b) Confined masonry walls (CMW), and
c) Reinforced masonry walls (RMW).

Table 8 indicates the Earthquake Zones in which these constructions are admissible:

6.1.1 *Masonry Walls with Bands* (*MWB*)

This structural system comprises of unreinforced masonry conforming to geometrical requirements specified in IS 1905, not designed for stress-resultant and deformation demands based on principles of structural engineering, but provided with horizontal bands only.

6.1.2 *Masonry Walls with Bands and Horizontal and Vertical Reinforcements* (*MWBR*)

These structural system comprises of unreinforced masonry conforming to geometrical requirements specified in IS 1905, not designed for stress-resultant and deformation demands based on principles of structural engineering, but provided with horizontal bands and horizontal and vertical reinforcements prescribed in this standard.

6.1.3 *Confined Masonry Walls (CMW)*

This structural system comprises of unreinforced masonry walls conforming to geometrical requirements specified in IS 1905, with the load-bearing walls having reinforced concrete horizontal (that is, tie-beams) and vertical (that is, tie-columns) confining concrete members built on all four edges of the masonry wall panel, and conforming to requirements of IS 17848.

6.1.4 *Reinforced Masonry Walls* (*RMW*)

This category comprises of masonry walls conforming to geometrical requirements specified in IS 1905, with the load-bearing masonry walls provided with vertical and horizontal reinforcements as per structural design requirements of Section 2 of IS

13920 (Part 2).

Table 8 Structural Systems Admissible in Different Earthquake Zones for Construction of Buildings of Different Categories
(*Clause* 6.1)

Sl No.	Earthquake Zone	Building Category		
		Normal	Important	Critical and Lifeline
(1)	(2)	(3)	(4)	(5)
i)	II	MWB MWBR CMW RMW	MWBR CMW RMW	CMW RMW
ii)	III			
iii)	IV	MWBR CMW RMW	CMW RMW	RMW
iv)	V			
v)	VI			

6.2 Design Earthquake Force

6.2.1 *Design Base Shear*

The total design horizontal earthquake base shear force at the base of the building shall be estimated as per **5.2.2.1**(a) and **5.2.2.1**(b) of Section 1 of this standard for strength and serviceability designs of buildings.

The following shall be used in the estimation of the design horizontal and vertical earthquake forces of masonry buildings:

a) Seismic weight shall be taken as per **5.2.2.2** of Section 1**;**
b) Importance Factor shall be taken as per **5.2.2.3** of Section 1**;**.
c) Damping ratio shall be taken as 5 percent of the critical damping as per **5.2.2.4** of Section 1; and
d) Elastic force reduction factor *R* shall be taken as per Table 3 of Section 1.

6.2.2 *Distribution of Design Base Shear*

6.2.2.1 *Distribution in elevation*

The design base shear shall be distributed along the height of the building as per **5.2.4.1** of Section 1.

6.2.2.2 *Distribution in plan*

The in-plan distribution of design lateral force at floor *i* to different walls shall be estimated based on their relative stiffness.

COMMENTARY

C-6.2.2 *Distribution of Design Base Shear*

C-6.2.2.1 *Distribution in plan*

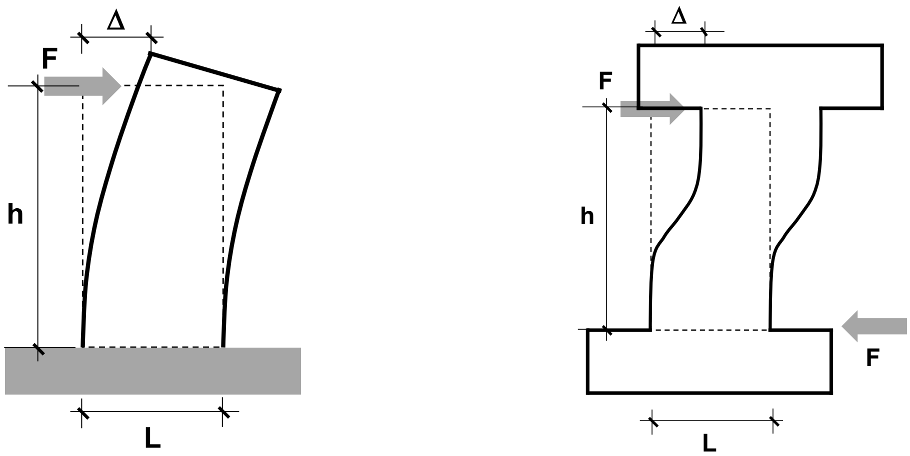
Masonry walls are often perforated to make arrangement for windows and doors. The distribution of lateral force in a masonry wall is dependent on the position of the openings and the relative rigidity of the masonry piers created due to the presence of the openings in the masonry wall. The relative rigidity is dependent on the height by length ratio (h/L ratio) of the piers and the end conditions of those masonry piers as the deflection of the masonry piers due to horizontal loading changes due to the end condition of the piers (Fig. C-3). Here, a simple process is described which can be used to distribute the lateral force in a wall, which can be considered to consist of some piers with some specific arrangements.

In any kind of placing of opening, the wall can be represented as a horizontal and vertical combination of piers with their respective end condition, which will be used to find out their rigidities. Where large openings occur, it is difficult to obtain effective coupling of the wall segments or piers. If the wall is analyzed as a horizontal combination of piers as shown in Fig. C-4 and the combined rigidity.

$$R = R_{c1} + R_{c2} + R_{c3},$$

where

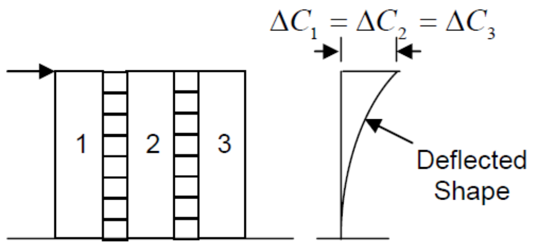
R_{c1}, R_{c2} and R_{c3} are the rigidities of the piers 1, 2 and 3, respectively.



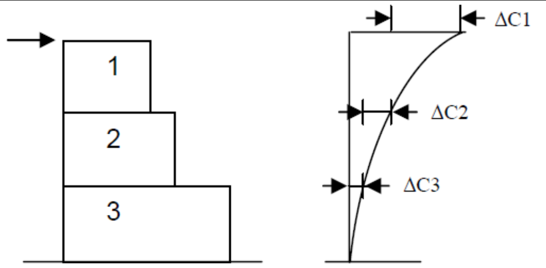
C-3A CANTILEVER WALL OR PIER (TOP FREE TO ROTATE)

C-3B PIER BETWEEN OPENINGS (TOP RESTRAINED FROM ROTATION)

FIG. C-3 DEFLECTIONS DUE TO END CONDITIONS OF PIERS



C-4A HORIZONTAL COMBINATION OF WALL SEGMENTS



C-4B VERTICAL COMBINATIONS OF WALL SEGMENTS

FIG. C-4 WALL COMBINATIONS FOR ESTIMATING RIGIDITIES OF WALLS WITH OPENINGS

If the segments are combined vertically (Fig. C-3**)**, the combined rigidity can be calculated as;

$$R = \frac{1}{\left(\frac{1}{R_{c1}} + \frac{1}{R_{c2}} + \frac{1}{R_{c3}}\right)},$$

where

R_{c1}, R_{c2} and R_{c3} are the rigidities of the piers 1, 2 and 3, respectively. Combination of these two types can be used to find the effective relative stiffness of a masonry wall. The expression ignores the rotations that occur at the tops of Segments 2 and 3. Therefore, overestimates the rigidity of the wall. Along with this, it is valid only for the application of loads at the top level of the building.

The rigidity *R* of the pier is dependent on its dimensions, Modulus of Elasticity *E*, Modulus of Rigidity *G* and the support conditions. For a cantilever pier the displacement due to combined action of bending and shear is.

$$\Delta_c = \frac{Fh^3}{3EI} + \frac{1.2Fh}{GA},$$

where

F = Horizontal force applied to the pier,
h = Height of the pier,
I = Moment of inertia of pier = $tL^3/12$,
G = Modulus of Rigidity = $0.4E_m$,
A = Area of the pier = *tL,*
L = Length of the pier, and
t = Thickness of pier.

Therefore, displacement of the cantilever pier is:

$$\Delta_c = \frac{1}{Et}\left[4\left(\frac{h}{l}\right)^3 + 3\left(\frac{h}{l}\right)\right].$$

The rigidity of a wall is proportional to the inverse of the deflection.

For cantilever walls, the rigidity is estimated:

$$R_c = \frac{1}{\Delta_c} = \frac{Et}{\left[4\left(\frac{h}{l}\right)^3 + 3\left(\frac{h}{l}\right)\right]}, \text{ and}$$

$$\frac{R_c}{Et} = \frac{1}{\left[4\left(\frac{h}{l}\right)^3 + 3\left(\frac{h}{l}\right)\right]}.$$

For a pier with both ends fixed against rotation, the deflection due to combined action of bending and shear is estimated as:

$$\Delta_f = \frac{Fh^3}{12EI} + \frac{1.2Fh}{GA} = \frac{F}{Et}\left[\left(\frac{h}{l}\right)^3 + 3\left(\frac{h}{l}\right)\right], \text{ and}$$

$$\frac{R_f}{Et} = \frac{1}{\left[\left(\frac{h}{l}\right)^3 + 3\left(\frac{h}{l}\right)\right]}.$$

From the above equations, the relative contributions of the bending and shear deformation depend on the wall aspect ratio (*h/L*). Therefore, the rigidity varies over the height of the building. For high *h/L* ratios, the effect of shear deformation is small and calculation of pier rigidities based on flexural stiffness is relatively accurate. For very squat walls (with $h/L<0.25$), rigidities based on shear deformation are reasonably accurate, but for intermediate walls (with $0.25<h/L<4.0$), both components of relative rigidity should be considered. The rigidities (*R/Et*) of walls with different *h/L* can be estimated from Fig. C-5.

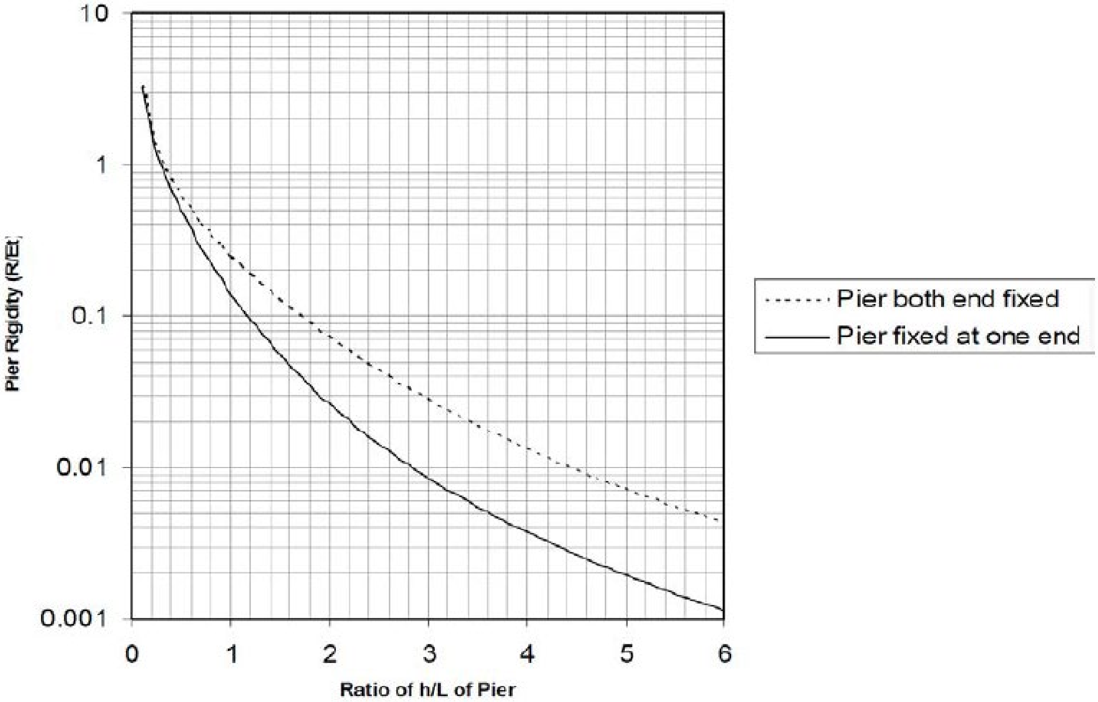


FIG. C-5 ESTIMATION OF RIGIDITIES OF MASONRY WALL

6.2.3 *Torsion*

6.2.3.1 *Design eccentricity*

The design eccentricity e_{di} to be used at floor *i* shall be taken as the one which produces the most severe effect in the masonry walls:

a) *Equivalent Static Method*:

$$e_{di} = \begin{cases} 1.8e_{si} + 0.05b_i \\ e_{si} - 0.05b_i \end{cases},$$

b) *Response Spectrum Method*:

$$e_{di} = \begin{cases} e_{si} + 0.05b_i \\ e_{si} - 0.05b_i \end{cases},$$

c) *Response History Method*:

$$e_{di} = \begin{cases} +0.05b_i \\ -0.05b_i \end{cases},$$

where

e_{si} = Static eccentricity at floor *i,* that is, distance between centre of mass and centre of stiffness, and

b_i = Floor plan dimension of floor *i*, perpendicular to the direction of force. **5.2.5.2** of Section 1 of this standard shall not be applicable for masonry buildings.

COMMENTARY

C-6.2.3 *Torsion*

C-6.2.3.1 Design eccentricity

Masonry buildings typically have large eccentricities, *e*/*B*, owing to distribution of walls along both directions, unlike in reinforced concrete and steel buildings. Hence, the dynamic amplification of 1.8 is not applied to masonry buildings.

6.3 Structural Analysis

To perform structural analysis, obtain demands on masonry walls and to perform demand-to-capacity verification under a combination of gravity and earthquake actions of the masonry building being designed, simple structural models based on hand calculations or more sophisticated computer models shall be developed.

6.3.1 *Modelling*

Structural analysis by hand calculations is feasible for single-storied simple configurations that are regular in plan and in elevation with minimum torsional eccentricities. For more complex configurations of masonry buildings, alternative approaches shall be adopted such as:

a) Equivalent Frame modelling, or
b) Finite Element modelling.

6.3.1.1 *Idealization of masonry buildings*

Linear elastic models with homogenized or smeared material properties for idealizing the composite masonry (that is, masonry unit and mortar) shall be adopted. Attention must be paid to the modelling of the floor and roof diaphragms, providing diaphragm constraints only where the floor and roof slabs can be classified as rigid.

a) *Equivalent Frame modelling*

The masonry walls shall be discretized into equivalent frame elements representing the gravity load carrying lateral load resisting vertical masonry elements, referred to as masonry piers, and those representing the lateral load resisting horizontal masonry elements, referred to as masonry spandrels.

In a perforated masonry wall (that is, with window and door openings), the masonry piers and masonry spandrels, the deformable elements, shall be connected through rigid masonry nodes. A masonry wall without openings shall be discretized only with a masonry pier.

The masonry piers and spandrels are idealized as beam-column frame elements capable of resisting axial loads, flexure and shear, and these frame elements are assigned section properties and appropriate boundary conditions. The effective height of the masonry piers can vary depending on the adjacent connecting elements and size of openings. The effective length of the masonry spandrels is determined by the width of the openings. Appropriate boundary conditions shall be assigned to the masonry piers, paying attention to whether rotational restraints are available at the top of the pier.

b) *Finite Element modelling*

All masonry elements are discretized with planar or three-dimensional elements, and these elements shall be assigned material properties.

6.3.1.2 *Section properties*

In case of equivalent frame modelling, the masonry piers and spandrels, which are treated as beam-column frame elements, shall be assigned elastic section properties, namely area of cross section and second moment of area, apart from elastic material properties, namely density, modulus of elasticity and Poisson's ratio for the homogenized masonry composite.

6.3.1.3 *Material properties*

Both in case of equivalent frame modelling and finite element modelling, elastic material properties, namely density, Modulus of Elasticity and Poisson's ratio shall be assigned to the homogenized masonry composite.

The Modulus of Elasticity E_m (in MPa) of clay brick masonry shall be taken as:

$$E_{\mathrm{m}} = 550 f_{\mathrm{m}},$$

where

f_{m} is compressive strength of masonry prism (in MPa) obtained as per IS 1905.

6.4 Special Categories of Buildings

6.4.1 *Small Buildings*

In single-storied residential buildings with less than 50 m^2 of built-up area, with load-bearing walls oriented along both principal plan directions to minimize torsional irregularity, structural plan density in orthogonal directions not less than 15 percent, and laterally supported wall lengths not more than 3.0 m, structural analysis is not required in:

a) *Earthquake Zones III*

If masonry walls are provided with bands (MWB).

b) *Earthquake Zones IV, V and VI*

If masonry walls are provided with bands, and horizontal and vertical reinforcements (MWBR).

SECTION 3

ADDITIONAL CRITERIA FOR CONCRETE BUILDINGS

7 EARTHQUAKE DEMAND ON CONCRETE BUILDINGS

7.1 Structural Systems

RC Buildings required to resist earthquake ground shaking shall have one of the structural systems listed in Table 9 depending on the Earthquake Zone in which the building is located.

Along any principal plan direction, the same type of lateral load resisting system shall be used.

7.1.1 *Ordinary Moment Resisting Frames*

Ordinary MRF buildings and their members shall be designed and detailed as per IS 456.

7.1.2 *Special Moment Resisting Frames*

Special MRF buildings and their members shall be designed and detailed as per IS 13920.

7.1.3 *Special Moment Resisting Frames with Special Structural Walls*

Special MRF buildings with Structural Walls (*SWs*) and their members shall be designed and detailed as per IS 13920.

Additionally, RC SWs shall be provided as per Table 9, such that:

a) The RC Special SWs shall be well distributed in the plan of the building;
b) The Structural Plan Density ρ_{SW} of the RC Special SWs alone shall be at least as per Table 10 depending on the Earthquake Zone in which the building is located along each principal plan direction; and
c) The RC Special SWs shall be designed and detailed as per IS 13920.

7.1.4 Dual System

In addition to provisions given in **7.1.3**, building is said to have a Dual System, if it consists of:

a) Special SWs and Special MRFs in the same principal plan direction in a single plane, or
b) Special MRFs and bracings in the same principal plan direction in a single plane, such that both of the following conditions are valid:

    1) Two systems are designed to resist total design lateral force in proportion

to their lateral stiffness, considering interaction of two systems at all floor levels; and

2) Special Moment Resisting Frames are designed to resist independently at least 25 percent of the design horizontal base shear.

Table 9 Admissible Structural Systems in RC Buildings
(*Clause* 7.1.3)

Sl No.	Earthquake Zone	Building Category		
		Normal	Important	Critical and Lifeline
(1)	(2)	(3)	(4)	(5)
i)	II	OMRF SMRF SMRF + SSW Dual System	SMRF SMRF + SSW Dual System	SMRF + SSW Dual System
ii)	III	SMRF SMRF + SSW Dual System	SMRF SMRF + SSW Dual System	SMRF + SSW Dual System
iii)	IV	SMRF + SSW Dual System	SMRF + SSW Dual System	Dual System
iv)	V	Dual System	Dual System	Dual System
v)	VI	Dual System	Dual System	Dual System

Table 10 Minimum Structural Plan Density of Structural Walls alone in RC Buildings
(*Clause* 7.1.3)

Sl No.	Earthquake Zone	Structural Plan Density of Structural Walls alone along each principal plan direction
i)	II	1.0 percent
ii)	III	1.5 percent
iii)	IV	2.0 percent
iv)	V	2.5 percent
v)	VI	2.5 percent

7.2 Special Buildings

7.2.1 *Small Buildings*

In residential buildings up to 2-storeys (including basement), and located in:

a) Earthquake Zones III and IV,

1) Equivalent Static Analysis is permitted; and
2) The structural system shall be at least SMRFs with infill walls in at least 90 percent of the bays; and

b) Earthquake Zones IV and V:

1) Equivalent Static Analysis is permitted; and
2) The structural system shall be at least SMRFs (with infills) with SSWs with at least 1.5 percent structural plan density in each principal plan direction.

7.2.2 *RC Frame Buildings with Open Storeys*

RC moment resisting frame buildings, which have open storey(s) at any level, such as due to discontinuation of unreinforced masonry (URM) infill walls or of structural walls, are known to have flexible and weak storeys as per **5.1** of Section 1. In such buildings, measures, such as RC Structural Walls or Braced frames (in select bays of the building) shall be provided along both plan directions as per the requirements specified hereunder, shall be adopted to increase both stiffness and strength to the required level in the open storey and the storeys below.

This provision shall be applicable to buildings in all earthquake zones.

a) Structural Walls

When RC structural walls are provided:

1) They shall be:
   i) founded on properly designed foundations;
   ii) continuous preferably over the full height of the building; and
   iii) connected preferably to the moment resisting frame of the building.

2) They shall be designed such that the building does NOT have:

   i) Additional torsional irregularity in plan than that already present in the building. In assessing this, lateral stiffness shall be included of all elements that resist lateral actions at all levels of the building;
   ii) Lateral stiffness in the open storey(s) is less than 80 percent of that in the storey above; and
   iii) Lateral strength in the open storey(s) is less than 90 percent of that in the storey above.

3) RC structural walls of this measure may be adopted even in regular buildings that do not have open storey(s).

b) Braces

When steel braces are provided, provisions of **8.2.2**(a) shall be complied with.

c) Unreinforced Masonry Infill Walls

In RC MRF Buildings and Unreinforced Masonry (URM) infill walls, effect of in-plane stiffness and strength URM infill walls shall be considered on storey stiffness and storey strength of the building (along the height). When variations of storey stiffness and strength along its height render the building to be irregular as per **5.1** of Section 1, the irregularity shall be corrected especially in

Earthquake Zones III, IV, V and VI.

Members of RC MRFs shall be designed for the more severe of the combinations of stress resultants arising from two structural analyses of the building, with:

a) Bare RC MRF building alone, and
b) RC MRF building with URM infills.

Further, the estimation of in-plane stiffness and strength of URM infill walls shall be based on the following:

1) The modulus of elasticity E_m (in MPa) of masonry infill wall shall be taken as:

$$E_{\mathrm{m}} = 550 f_{\mathrm{m}},$$

where

f_{m} is compressive strength of masonry prism (in MPa) obtained as per IS 1905. Alternately, it may be estimated as:

$$f_{\mathrm{m}} = 0.63 f_{\mathrm{b}}^{0.49} f_{\mathrm{mo}}^{0.32}$$

where

f_{b} = Compressive strength (in MPa) of brick, and
f_{mo} = Compressive strength (in MPa) of mortar.

2) URM infill walls shall be modeled by using equivalent diagonal struts as below:
   i) Ends of diagonal struts shall be considered to be pin-jointed to RC frame;
   ii) For URM infill walls without any opening, width w_{ds} of equivalent diagonal strut (Fig. 14) shall be taken as:

$$w_{\mathrm{ds}} = 0.175 \alpha_{\mathrm{h}}^{-0.4} L_{\mathrm{ds}}$$

where

$$\alpha_{\mathrm{h}} = h \left(\sqrt[4]{\frac{E_{\mathrm{m}} t \sin 2\theta}{4 E_{\mathrm{c}} I_{\mathrm{c}} h}} \right)$$

where

E_{m} and E_{c} are the moduli of elasticity of URM infill and concrete, I_{c} the second moment of area of the adjoining column, t the thickness of the infill wall, and θ the angle of the diagonal strut with the horizontal;

3) For URM infill walls with openings, no reduction in strut width is required; and

4) Thickness of the equivalent diagonal strut shall be taken as thickness t of original URM infill wall, provided $h/t<12$ and $l/t<12$, where h is clear height of URM infill wall between the top beam and bottom floor slab, and l clear length of the URM infill wall between the vertical RC elements (columns, walls or a combination thereof) between which it spans.

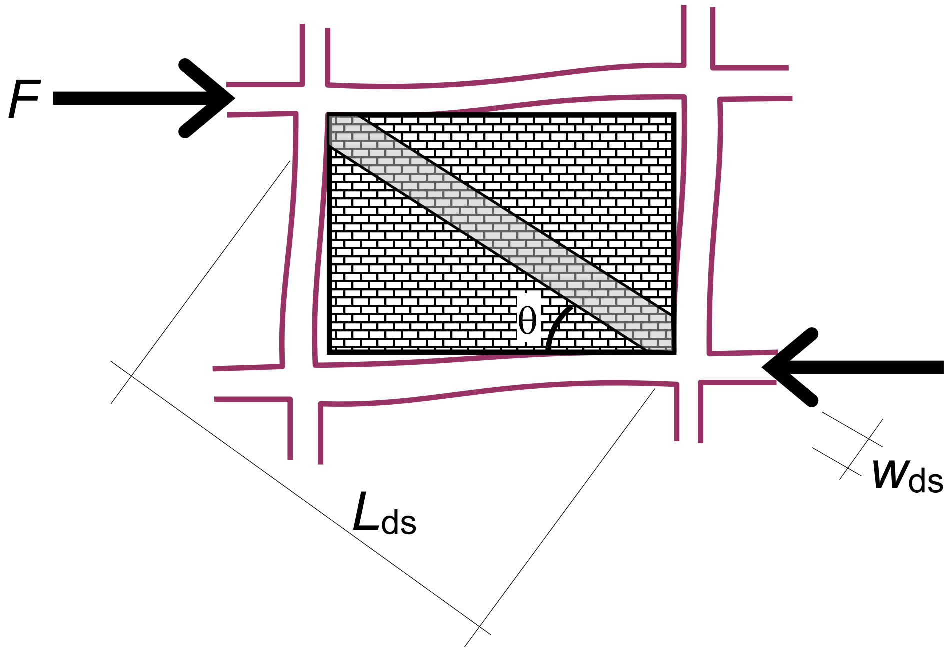


FIG. 14 EQUIVALENT DIAGONAL STRUT OF THE URM INFILL WALL

SECTION 4

ADDITIONAL CRITERIA FOR STEEL BUILDINGS

8 EARTHQUAKE DEMAND ON STEEL BUILDINGS

8.1 Structural Systems

Steel Buildings required to resist earthquake ground shaking shall have one of the structural systems listed in Table 11 depending on the Earthquake Zone in which the building is located.

Along any principal plan direction, the same type of lateral load resisting system shall be used.

Table 11 Admissible Structural Systems in Steel Buildings
(*Clause* 8.1)

Sl No.	Earthquake Zone	Building Category		
		Normal	Important	Critical and Lifeline
(1)	(2)	(3)	(4)	(5)
i)	II	OMRF OCBF SCBF SMRF EBF	SCBF SMRF EBF	SCBF SMRF EBF
ii)	III	SCBF SMRF EBF	SCBF SMRF EBF	SCBF SMRF EBF
iii)	IV	SMRF EBF	SMRF EBF	SMRF EBF
iv)	V	EBF	EBF	EBF
v)	VI	EBF	EBF	EBF

8.1.1 *Ordinary Moment Resisting Frame*

Ordinary MRF buildings and their members shall be designed and detailed as per IS 800.

8.1.2 *Ordinary Concentrically Braced Frame*

Ordinary Concentrically Braced Frame buildings and their members shall be designed and detailed as per IS 800.

8.1.3 *Special Concentrically Braced Frames*

Special Concentrically Braced Frame buildings and their members shall be designed and detailed as per CED 39(18640).

8.1.4 *Special Moment Resisting Frames*

Special MRF buildings and their members shall be designed and detailed as per CED 39(18640).

8.1.5 *Eccentrically Braced Frames*

Steel eccentrically braced frame buildings and their members shall be designed and detailed as per CED 39(18640).

8.2 Special Buildings

8.2.1 *Small Buildings*

In residential buildings up to 2-storeys (including basement), and located in:

a) Earthquake Zones III and IV,

    1) Equivalent Static Analysis is permitted; and
    2) The structural system shall be at least SMRFs with infill walls in at least 90 percent of the bays; and

b) Earthquake Zones V and VI,

    1) Equivalent Static Analysis is permitted; and
    2) The structural system shall be at least SMRFs (with infills), SCBFs and EBFs in each principal plan direction as per **5.1**.

8.2.2 *Steel Buildings with Open Storeys*

Steel moment resisting frame buildings, which have open storey(s) at any level, such as due to discontinuation of unreinforced masonry (URM) infill walls or of braces, are known to have flexible and weak storeys as per **5.1** of Section 1. In such buildings, measures, such as Steel Braces or RC structural Walls in select bays of the building, shall be provided along both plan directions as per the requirements specified hereunder, shall be adopted to increase both stiffness and strength to the required level in the open storey and the storeys below.

This provision shall be applicable to buildings in all earthquake zones.

a) Braces

When steel braces are provided:

1) They shall be:

    i) continuous over the full height of the building; and
    ii) connected to the moment resisting frame of the building.

2) They shall be designed such that the building does NOT have:

   i) Additional torsional irregularity in plan than that already present in the building. In assessing this, lateral stiffness shall be included of all elements that resist lateral actions at all levels of the building;
   ii) Lateral stiffness in the open storey(s) is less than 80 percent of that in the storey above; and
   iii) Lateral strength in the open storey(s) is less than 90 percent of that in the storey above.

3) Steel braces of this measure may be adopted even in regular buildings that do not have open storey(s).

b) Structural Walls

When RC structural walls are provided, provisions of **7.2.2**(a) shall be complied with.

c) Unreinforced Masonry Infill Walls

In steel buildings with Unreinforced Masonry (URM) infill walls, effect of in-plane stiffness of URM infill walls shall be considered on storey stiffness of the building (along the height), as per **6.3** of Section 2.

Annex A

LIST OF CROSS REFERRED INDIAN STANDARDS

IS Number	*Title*
IS 456 : 2000	Plain and reinforced concrete – Code of practice (*Fourth Revision*)
IS 800 : 2007	General construction in steel – Code of practice (*Second Revision*)
IS 1893 (Part 1: XXXX) CED 39(22343)	Criteria for Earthquake Resistant Design of Structures : Part 1 General Provisions – Code of Practice (*Seventh Revision*)
IS 1905 : 1987	Structural Use of Unreinforced Masonry – Code of Practice (*Third Revision*)
IS 13920 : 2016	Ductile design and detailing of reinforced concrete structures subjected to seismic forces - Code of practice (*First Revision*)
IS 17848 : 2022	Confined masonry for earthquake resistance – Code of practice
CED 39(18640)	Earthquake resistant design and detailing of steel buildings – Code of practice
IS 13826: 1993	Improving earthquake resistance of earthen buildings — Guidelines
IS 13827: 1993	Improving earthquake resistance of low strength masonry buildings — Guidelines

Annex 2

(Committee composition will be added after finalization of the draft)

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