IS 1893 and IS 13920 Codal Changes

Reading between the lines…

Alpa Sheth

IS 1893 -2016

भारतीय मानक
Indian Standard

IS 1893 (Part 1) : 2016

संरचनाओं के भूकम्परोधी डिजाइन के मानदंड

भाग 1 सामान्य प्रावधान और भवन

(छठा पुनरीक्षण)

Criteria for Earthquake Resistant Design of Structures

Part 1 General Provisions and Buildings

(Sixth Revision)

ICS 91.120.25

भारतीय मानक ब्यूरो
BUREAU OF INDIAN STANDARDS
मानक भवन, 9 बहादुरशाह ज़फर मार्ग, नई दिल्ली-110002
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December 2016 Price Group 12

Changes In Estimation Of The Hazard

a) Design spectra extended up to natural period up of 6 s;

b) Same design response spectra for all buildings, irrespective of the material of construction (for steel and concrete);

c) New method for arriving at the approximate natural period of buildings

d) Response Reduction Factors Revised; Buildings with flat slabs included;

e) **Minimum design lateral force**

Changes In Estimation Of The Hazard

f) Load combinations consistent with other codes;

g) Temporary structures included

h) Importance factor provisions to acknowledge the density and business continuity;

i) Design Vertical Acceleration Coefficient *Av*

II. Changes in Estimation of Resistance Capacity

j) How to handle different types of irregularity of structural system;

k) Effect of masonry infill;

l) Use of Cracked Section Properties Ieff

m) Torsional provisions revised;

n) Simplified method for liquefaction potential analysis.

o) Open Ground Storey structures requirements revised

BACKDROP


"Then we are agreed nine to one that we will say our previous vote was unanimous!"

- Code is a consensus document
- Collective wisdom of the drafting group, modified by larger committee and public comments
- Open to modifications in next revision
- Code need extensive usage for evaluating its ease of applicability and limitations

Design Horizontal Earthquake Force

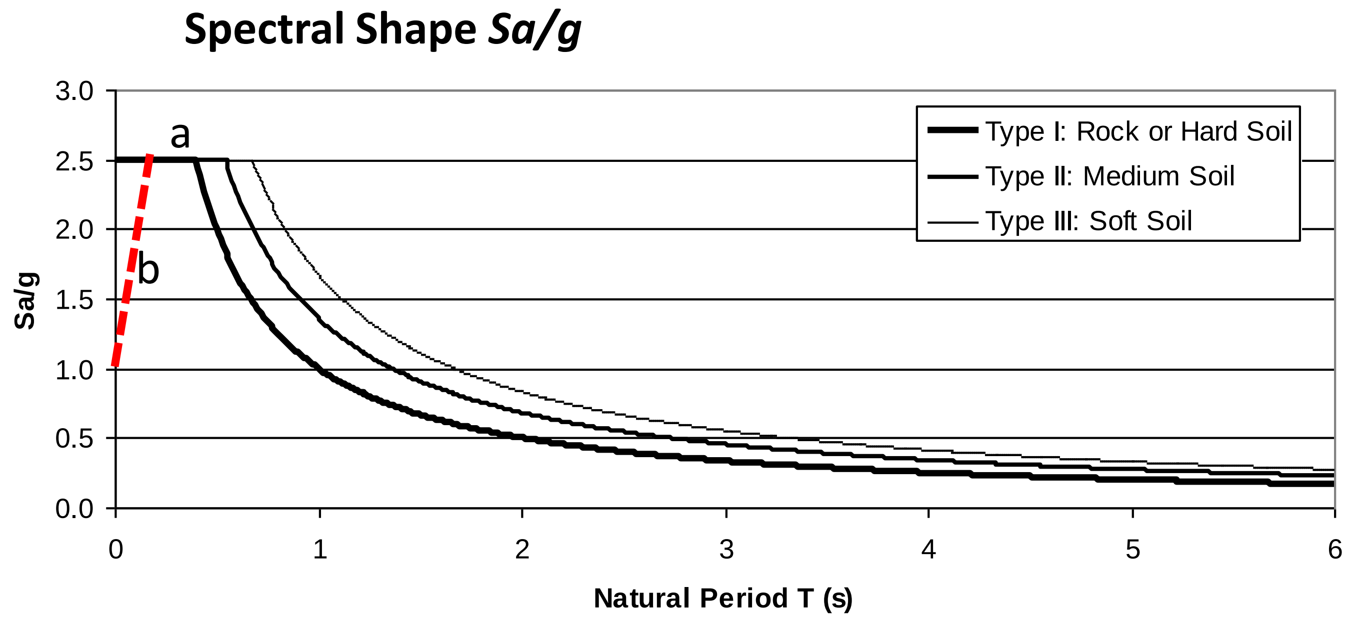


Figure 2: Design Acceleration Coefficient (S_a /g) (corresponding to 5% damping) for use in (a) Equivalent Static Method, and (b) Response Spectrum Method

Design Horizontal Earthquake Force

5% damping for all structures

– *Design force independent of material of construction*

- **Steel, RC and Masonry**

– *Earlier 2% for Steel, 5% for RC, 7% for Masonry*

Note: Damping for Wind Conditions different!!!

Steel Structures now less punished

- New Code is more attractive for Steel Structures.
  - Damping increase from 2% to 5% and R factor from 4 to 5. Thus reduction in seismic coefficient by 1.4*5/4 =1.75 times

Design Horizontal Earthquake Force-New Natural Period Empirical Formula

– *Bare MRF buildings (without any masonry infills)*

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

– *Buildings with RC Structural Walls*

$$T_a = \frac{0.075h^{0.75}}{\sqrt{A_w}} \geq \frac{0.09h}{\sqrt{d}} \qquad A_w = \sum_{i=1}^{N_w}\left[A_{wi}\left\{0.2+\left(\frac{L_{wi}}{h}\right)\right\}^2\right]$$

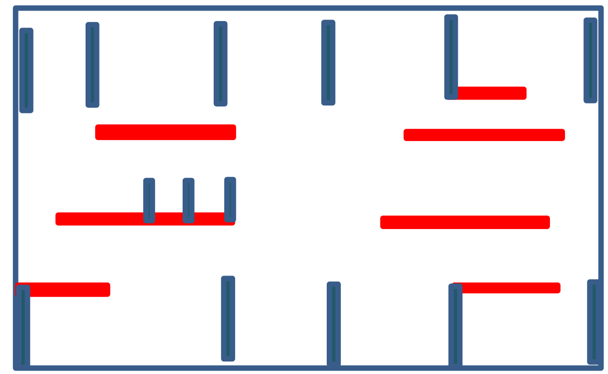
– *All other Buildings*

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

New Natural Period Empirical Formula for Shear Wall Buildings

Gives realistic, higher Natural Period for building with walls structures of lesser stiffness

Building ht h	45 m	15 storeys		
Building depth d	30m	Building Width b	20m	
Tx (s)	=0.09h/sqrt(d) =	0.74	(equation for non-frame structures)	
Tx (S)	=0.1n=	1.5	(not applicable for non-frame structures)	
Wall Thk (m)	Length (m) L_{wi}	Area (m^2)A_{wi}	L_{wi}/h	$A_{wi}(0.2+L_{wi}/h)$^2
0.3	5	1.5	0.111	0.145
0.25	6	1.5	0.133	0.167
0.3	3	0.9	0.067	0.064
0.25	7	1.75	0.156	0.221
0.25	4	1	0.089	0.083
0.3	3.5	1.05	0.078	0.081
0.3	6.5	1.95	0.144	0.231
	ΣA_{wi}=	9.65	A_w=	0.9929
Tx	=0.075h^0.75/sqrt(A_w) =			1.31s
Tx=			**1.31s**	New Alternative equation T for wall structures



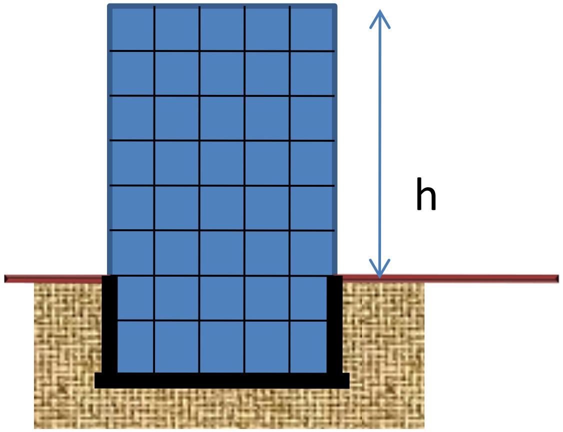
As per **7.6.2 c)**
Ty=0.09h/sqrt(b) = 0.9s
As per **7.6.2 b)**
Ty=0.99s (ΣA_{wi}=18.3m^2, A_w=1.72)

T (s)	As per 7.6.2c	As per 7.6.2b	% walls ($\Sigma \alpha_{wi}$/ (bxd)
Tx (s)	0.74	1.31	0.71
Ty (s)	0.9	0.99	1.4

As ΣA_{wi}>2%, equations merge, and b) will give T less than that from c)

Defining Building height h

- *h* = Height (in m) of building. 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.



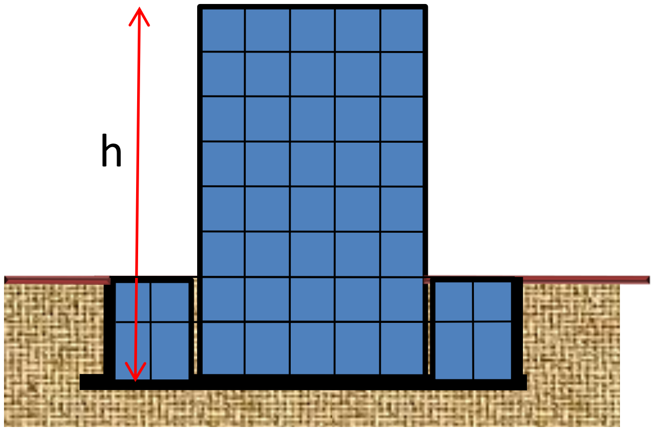


Table 9 Response Reduction Factor *R* for Building Systems

(*Clause* 7.2.6)

Sl No. (1)	Lateral Load Resisting System (2)	R (3)
i)	**Moment Frame Systems**	
	a) RC buildings with ordinary moment resisting frame (OMRF)[1)]	3.0
	b) RC buildings with special moment resisting frame (SMRF)	5.0
	c) Steel buildings with ordinary moment resisting frame (OMRF)[1)]	3.0
	d) Steel buildings with special moment resisting frame (SMRF)	5.0
ii)	**Braced Frame Systems**[2)]	
	a) Buildings with ordinary braced frame (OBF) having concentric braces	4.0
	b) Buildings with special braced frame (SBF) having concentric braces	4.5
	c) Buildings with special braced frame (SBF) having eccentric braces	5.0
iii)	**Structural Wall Systems**[3)]	
	a) Load bearing masonry buildings	
	1) Unreinforced masonry (designed as per IS 1905) without horizontal RC seismic bands[1]	1.5
	2) Unreinforced masonry (designed as per IS 1905) with horizontal RC seismic bands	2.0
	3) Unreinforced masonry (designed as per IS 1905) with horizontal RC seismic bands and vertical reinforcing bars at corners of rooms and jambs of openings (with reinforcement as per IS 4326)	2.5
	4) Reinforced masonry [refer SP 7 (Part 6) Section 4]	3.0
	5) Confined masonry	3.0
	b) Buildings with ordinary RC structural walls[1)]	3.0
	c) Buildings with ductile RC structural walls	4.0
iv)	**Dual Systems**[3)]	
	a) Buildings with ordinary RC structural walls and RC OMRFs[1]	3.0
	b) Buildings with ordinary RC structural walls and RC SMRFs[1]	4.0
	c) Buildings with ductile RC structural walls with RC OMRFs[1]	4.0
	d) Buildings with ductile RC structural walls with RC SMRFs	5.0

R - Same for Steel & RC Buildings

Special Provision for Flat Slab Structures

(Response Reduction Factor R)

v) **Flat Slab – Structural Wall Systems**[4)]

RC building with the three features given below: a) Ductile RC structural walls (which are designed to resist 100 percent of the design lateral force), b) Perimeter RC SMRFs (which are designed to independently resist 25 percent of the design lateral force), and preferably c) An outrigger and belt truss system connecting the core ductile RC structural walls and the perimeter RC SMRFs[1].	3.0

4 In these buildings, (a) punching shear failure shall be avoided, and (b) lateral drift at the roof under design lateral force shall not exceed 0.1 percent.

Minimum Design Lateral Force

- Min design lateral force ~ 25% of the applicable A_h in the zone (at PGA)

Table 7 Minimum Design Earthquake Horizontal Lateral Force for Buildings
(*Clause* 7.2.2)

Sl No. (1)	*Seismic Zone* (2)	ρ Percent (3)
i)	II	0.7
ii)	III	1.1
iii)	IV	1.6
iv)	V	2.4

Table 3 Seismic Zone Factor *Z*
(*Clause* 6.4.2)

Seismic Zone Factor (1)	II (2)	III (3)	IV (4)	V (5)
Z	0.10	0.16	0.24	0.36

$$A_h = \frac{\left(\frac{Z}{2}\right)\left(\frac{S_a}{g}\right)}{\left(\frac{R}{I}\right)}$$

IMPORTANCE FACTOR

- Increased to 1.2 for buildings >200 persons occupancy.(That's buildings with more than 50 apartments)
- Will affect PMAY buildings which are high density, low cost
- Ironically low cost housing has lesser loads (1.5kN/m2 Live load as vs 2 kN/m2 in non low cost housing)

A_v Vertical seismic coeff

Not same as A_h
2/3 of A_h at PGA (at T_a=0)

$$A_v = \begin{cases} \dfrac{\left(\dfrac{2}{3}\times\dfrac{Z}{2}\right)(2.5)}{\left(\dfrac{R}{I}\right)} & \text{For buildings governend by IS1893 (Part 1)} \\ \dfrac{\left(\dfrac{2}{3}\times\dfrac{Z}{2}\right)(2.5)}{\left(\dfrac{R}{I}\right)} & \text{For liquid retaining tanks governend by IS1893 (Part 2)} \\ \dfrac{\left(\dfrac{2}{3}\times\dfrac{Z}{2}\right)\left(\dfrac{S_a}{g}\right)}{\left(\dfrac{R}{I}\right)} & \text{For bridges governend by IS1893 (Part 3)} \\ \dfrac{\left(\dfrac{2}{3}\times\dfrac{Z}{2}\right)\left(\dfrac{S_a}{g}\right)}{\left(\dfrac{R}{I}\right)} & \text{For industrial structures governend by IS1893 (Part 4)} \end{cases}$$

Modal Mass Participation Factors

In buildings located in Seismic Zones II and III, it shall be ensured that the first three modes together contribute at least 65 percent mass participation factor in each principal plan direction. And, in buildings located in Seismic Zones IV and V, it shall be ensured that,

1) the first three modes together contribute at least 65 percent mass participation factor in each principal plan direction, and
2) the fundamental lateral 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.

May not be possible in Buildings with large podiums!!!

Earthquake- Capacity Side

- Cracked Section Stiffness Properties
- Irregularities
- Code has become more stringent and punitive for stiffness irregularity

Cracked Section Properties Defined…

- Ieff (cracked) =0.7 I_{gross} for columns and walls
- Ieff (cracked) =0.35 I_{gross} for beams

Irregularities

- Torsional Irregularity – Max and min displacements on floor differ by over 50%
- Torsional Irregularity- First mode is in torsion
- Plan Stiffness irregularity- Stiffness on floor below must be more than that on floor above. (In Tall buildings code – relaxed to 70%)
- Plan Mass Irregularity- Mass above >1.5 times mass on floor below.
- Plan Weak Storey – Strength less than floor above.

FLOATING COLUMNS PROHIBITED IN SMRF

Horizontal Irregularity

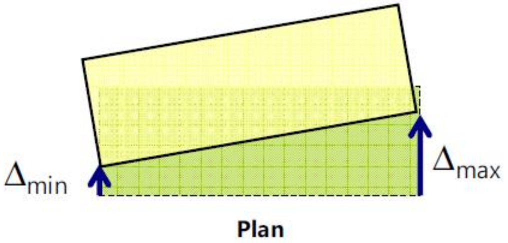
A building is said to be torsionally irregular, when,

1) the maximum horizontal displacement of any floor in the direction of the lateral force at one end of the floor is more than 1.5 times its minimum horizontal displacement at the far end of the same floor in that direction; and
2) 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 directions

In torsionally irregular buildings, when the ratio of maximum horizontal displacement at one end and the minimum horizontal displacement at the other end is in the range,

i) 1.5 – 2.0, (a) 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 then (b) three dimensional dynamic analysis method shall be adopted; and

ii) More than 2.0, the building configuration shall be revised

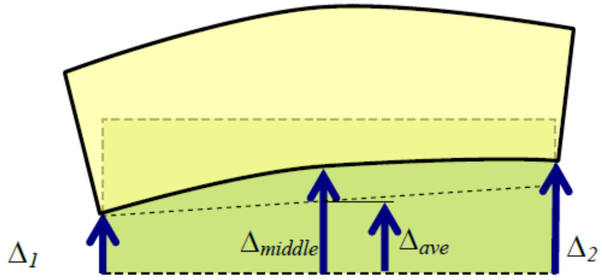


$$\Delta_{max} > 1.5\Delta_{min}$$

Diaphragm Flexibility

7.6.4 *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.
- A floor diaphragm shall be considered to be flexible, if it deforms such that the maximum lateral displacement measured from the chord of the deformed shape at any point of the diaphragm is more than 1.2 times the average displacement of the entire diaphragm (*see Fig. 6).*
- 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 centres) as topping, and of plan aspect ratio less than 3, can be considered to be providing rigid diaphragm action.



$\Delta_{middle} > 1.2\Delta_{ave}$

where $\Delta_{ave} = (\Delta_1 + \Delta_2)/2$

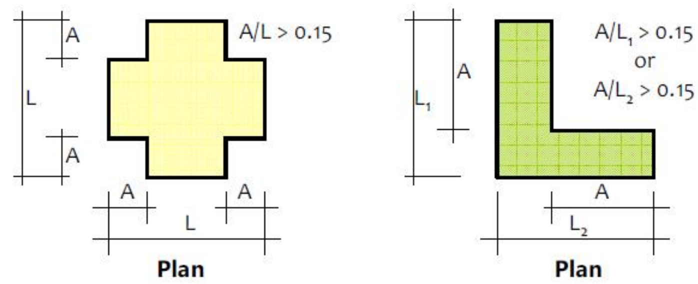
Plan

FIG. 6 DEFINITION OF FLEXIBLE FLOOR DIAPHRAGM

Horizontal Irregularity

•**Re-entrant Corners**

In buildings with re-entrant corners, three-dimensional dynamic analysis method shall be adopted

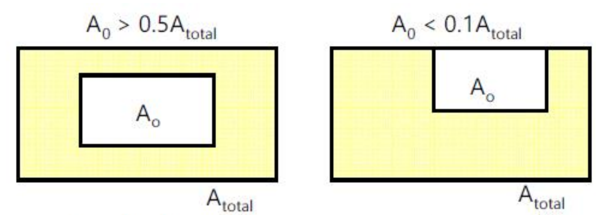


•**Floor Slabs having Excessive Cut-Outs or Openings**

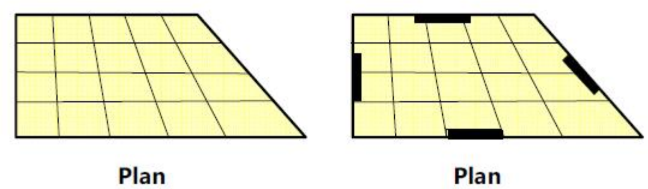
In buildings with discontinuity in their in-plane stiffness, if the area of the geometric cut-out is,

a) less than or equal to 50 percent, the floor slab shall be taken as rigid or flexible depending on the location of and size of openings; and

b) more than 50 percent, the floor slab shall be taken as flexible



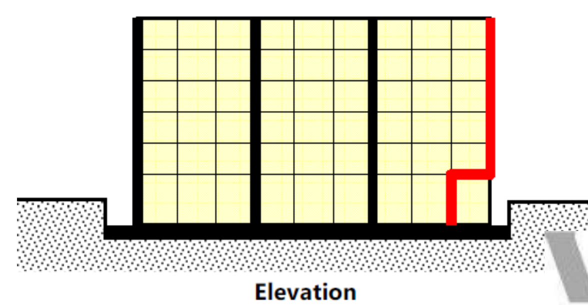
•**Buildings with non-parallel lateral force resisting system** shall be analyzed for load combinations mentioned in 6.3.2.2 or 6.3.4.1**.**



•**In a building with out-of-plane offsets in vertical element following two conditions shall be satisfied, if building is located in Seismic Zone III, IV or V:**

(a) Lateral drift shall be less than 0.2% in the storey having the offset and in the storeys below; and

(b) Specialist literature shall be referred for removing the irregularity arising due to out-of-plane offsets in vertical elements.



Horizontal Irregularity

7.8.2 *Design Eccentricity*

While performing structural analysis by the Seismic Coefficient Method or the Response Spectrum Method, the design eccentricity e_{di} to be used at floor *i* shall be taken as:

$$e_{di} = \begin{cases} 1.5e_{si} + 0.05b_i \\ e_{si} - 0.05b_i \end{cases}$$

whichever gives the more severe effect on lateral force resisting elements.

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.

The factor 1.5 represents dynamic amplification factor, and $0.05b_i$ represents the extent of accidental eccentricity. The above amplification of 1.5 need not be used, when performing structural analysis by the Time History Method.

Vertical Irregularity

i) Stiffness Irregularity (Soft Storey)

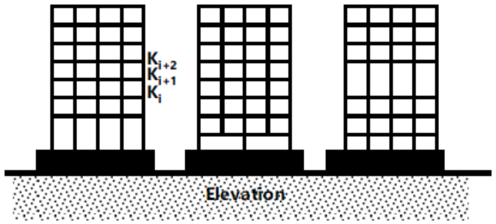
A soft storey is a storey whose lateral stiffness is less than that of the storey above.

The structural plan density (SPD) shall be estimated when unreinforced masonry infills are used. When SPD of masonry infills exceeds 20 percent, the effect of URM infills shall be considered by explicitly modelling the same in structural analysis (as per **7.9**). **The design forces for RC members shall be larger of that obtained from analysis of:**

a) Bare Frame, and

b) Frames with URM Infills,

using 3D modeling of the structure. In buildings designed considering URM infills, the inter-storey drift shall be limited to 0.2 percent in the storey with stiffening and also in all storeys below.



$K_{i+2} > K_{i+1}$

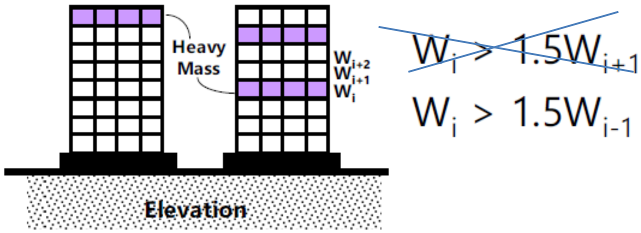
$K_{i+1} > K_i$

Vertical Irregularity

Mass Irregularity

Mass irregularity shall be considered to exist, when the seismic weight *(as per* ***7.7) of any floor is more than 150 percent of that of the floors below.***

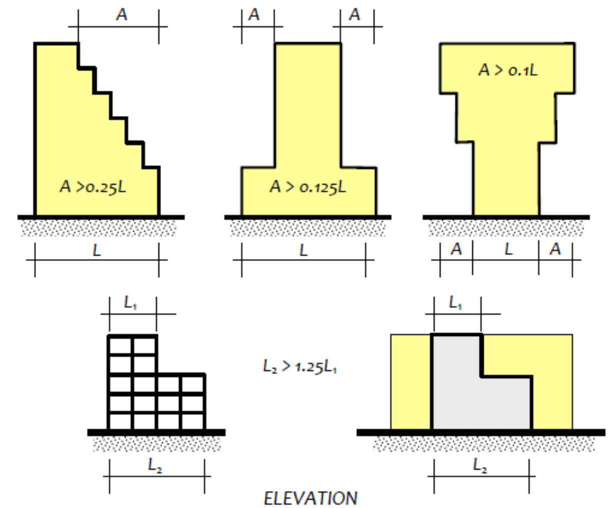
In buildings with mass irregularity and located in Seismic Zones III, IV and V, the earthquake effects shall be estimated by Dynamic Analysis Method (as per ***7.7).***



Vertical Geometric Irregularity

Vertical geometric irregularity shall be considered to exist, when the horizontal dimension of the lateral force resisting system in any storey is more than 125 percent of the storey below.

In buildings with vertical geometric irregularity and located in Seismic Zones III, IV and V, the earthquake effects shall be estimated by Dynamic Analysis Method (as per ***7.7).***

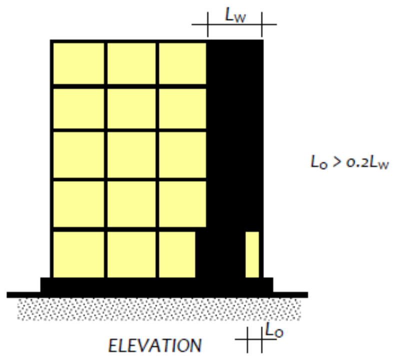


4(C) VERTICAL GEOMETRIC IRREGULARITY

Vertical Irregularity

In-Plane Discontinuity in Vertical Elements Resisting Lateral Force

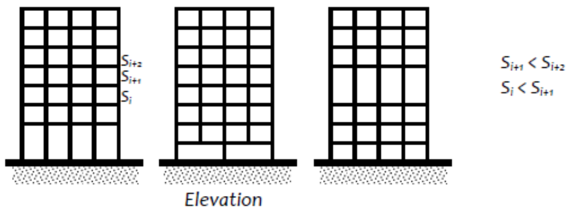
In-plane discontinuity in vertical elements which are resisting lateral force shall be considered to exist, when in-plane offset of the lateral force resisting elements is greater than 20 percent of the plan length of those elements, …. *in Seismic Zones III, IV and V, buildings with in-plane discontinuity shall not be permitted.*



4(D) IN-PLANE DISCONTINUITY IN VERTICAL ELEMENTS RESISTING LATERAL FORCE

v) **Strength Irregularity (Weak Storey)**

A weak storey is a storey whose lateral strength is less than that of the storey above. *In such a case, buildings in Seismic Zones III, IV and V shall be designed such that safety of the building is not jeopardized; also, provisions of* ***7.10 shall be followed***



4(E) STRENGTH IRREGULARITY (WEAK STOREY)

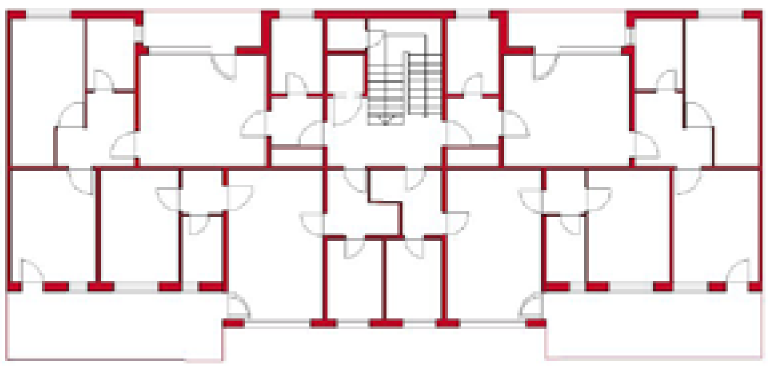
How much wall area in Shear Wall Structures ?

Country	Wall Density (Total)	Min in each direction
Kyrgyzstan	15%	6.5%
Turkey	4-12%	2-6%
Chile	3-6%	1.5-3%
Romania	12-14%	6-7%
Colombia	3-5%	1.5%
India	1-4%?	0.5-3%?

Min 1.5%-2% in each direction is desirable for Zones IV and V

Typical Layouts…

Chile



Romania

Ref: CONCRETE SHEAR WALL CONSTRUCTION

M. Ofelia Moroni, University of Chile, Santiago, Chile (eeri.org)

When to consider URM panels in analysis and design?

- Structural Plan Density of unreinforced masonry infill walls > 20%, URM walls to be considered in design

- Not usually the case!!!

Golcuk, 2000

How to include URM infill wall stiffness

7.9.2.2 URM infill walls shall be modeled by using equivalent diagonal struts as below:

a) Ends of diagonal struts shall be considered to be pin-jointed to *RC* frame;

b) For URM infill walls without any opening, width w_{ds} of equivalent diagonal strut (*See* Fig. 7) shall be taken as:

$$w_{ds} = 0.175\alpha_h^{-0.4} L_{ds}$$

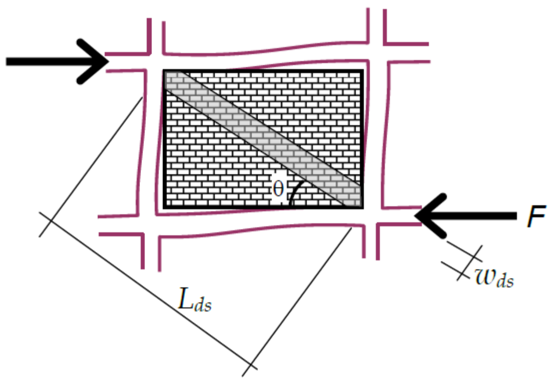
where

$$\alpha_h = h\left(\sqrt[4]{\frac{E_m t \sin 2\theta}{4 E_f I_c h}}\right)$$

where E_m and E_f are the modulii of elasticity of the materials of the URM infill and RC MRF, I_c the moment of inertia of the adjoining column, t the thickness of the infill wall, and θ the angle of the diagonal strut with the horizontal;

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

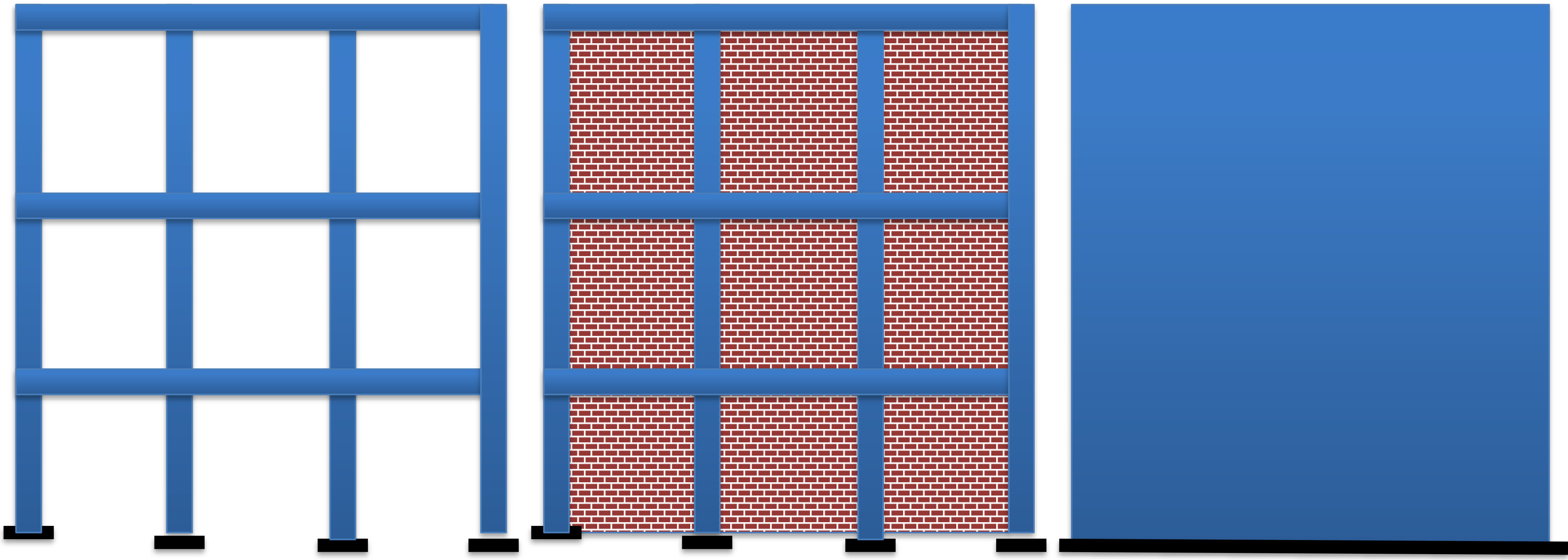
d) Thickness of the equivalent diagonal strut shall be taken as thickness t of original URM infill wall, provided $h/t < 12$ and $l/t < 12$, where h is clear height of URM infill wall between the top beam and bottom floor slab, and l clear length of the URM infill wall between the vertical RC elements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



. 7 EQUIVALENT DIAGONAL STRUT OF URM INFILL WALL

Effect of URM infill wall stiffness

500x500 columns, 300x500 beams, 230 thk masonry wall/230 thick RC wall



T=0.18s

k=62

T=0.142s

k=100

T=0.041s

k=1200

RC Frame Buildings with Open Storeys

In such buildings measures shall be adopted, which increase both stiffness and strength like**:**

a) RC structural walls, or b) Braced frames, in select bays

When the RC structural walls are provided, they shall:

a) Be founded on properly designed foundations;

b) Be continuous preferably over full height of building; and

c) Be connected preferably to the moment resisting frame of building.

d) Not cause additional torsional irregularity in plan than already present

RC Frame Buildings with Open Storeys

e) Lateral stiffness in the open storey(s) >80 percent of that in the storey above; and

f) Lateral strength in the open storey(s) > 90 percent of that in the storey above.

g) Have at least 2 percent (SPD) along each principal direction in Seismic Zones III, IV and V and well distributed in the plan of the building along each direction.

IS:13920 – 2016

- **Major Changes**

– *Scope*

- **Ductile Design and Detailing of RC Structures**

– *Collapse Mechanism*

- **Column-Beam Strength Ratio b**

– *Shear Design of Beam-Column Joints*

- **Minimum Column Size Max[*300 mm; 20db]***

– *Flexural Strength of Structural Walls*

- **Principle of Superposition**

– *Mechanical Couplers*

IS 13920-2016

भारतीय मानक
Indian Standard

IS 13920 : 2016

भूकंपीय बल के प्रभाव के अंर्तगत प्रबलित कंकरीट संरचनाओं का तन्य विस्तार — रीति संहिता

(पहला पुनरीक्षण)

Ductile Design and Detailing of Reinforced Concrete Structures Subjected to Seismic Forces — Code of Practice

(First Revision)

ICS 47.020.99; 93.140

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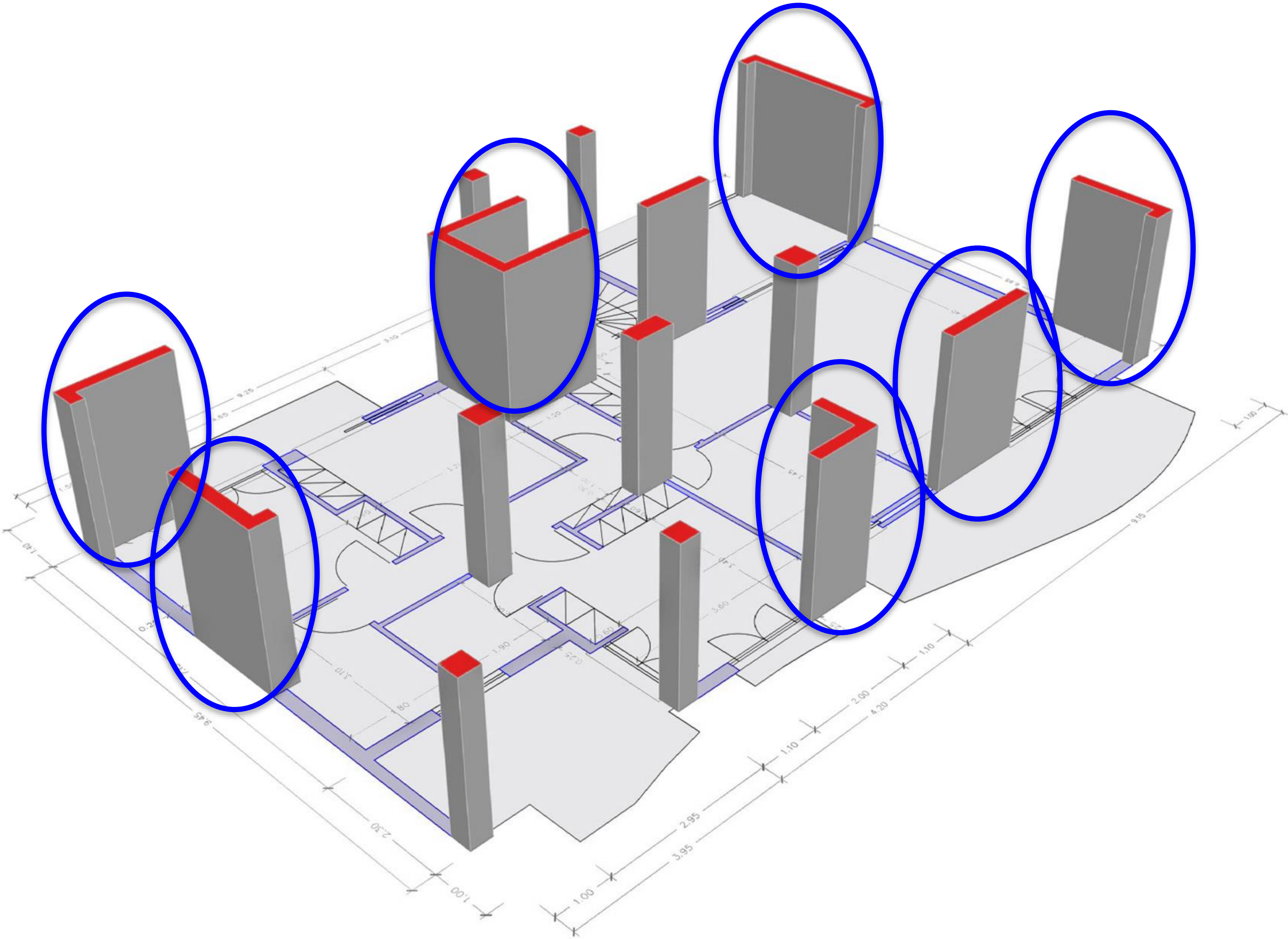
SCOPE

1.1.3 All RC frames, RC walls and their elements in a structure need ***not*** be designed to resist lateral loads and the designer can judiciously identify the lateral load resisting system based on relative stiffness and location in the building and design those members for full lateral force.

How to identify LRFS

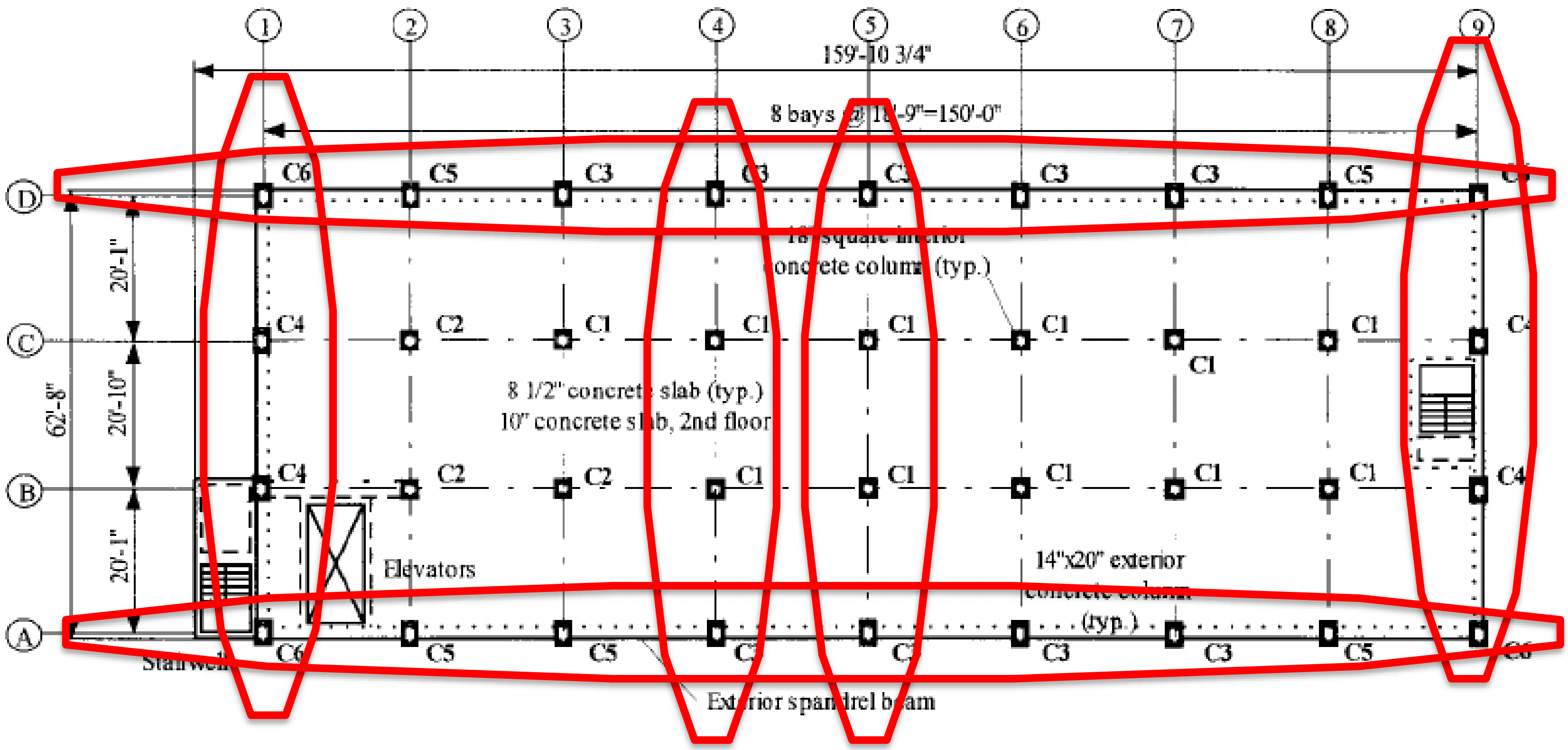
- Select only those frames or shear walls as part of LFRS (in a direction) which participate significantly (Base Shear distribution is a good indicator for this).
- Identify what % (x) of lateral load these frames these together carry (Should be in range of 90%+).
- Scale lateral force by 100/x - to ensure the selected system is capable of carrying at least 100% of force in considered direction

SELECT APPROPRIATE SYSTEM…



SELECT APPROPRIATE SYSTEM…

- Proportion selected frames accordingly so as to take ~85-90+% of base shear.



Also….

- **1.1.3** …….RC monolithic members assumed not to participate in the lateral force resisting system (*see* **3.7**) shall be permitted provided that their effect on the seismic response of the system is accounted for. *Consequence of failure of structural and non-structural members not part of the lateral force resisting system shall also be considered in design.*

Gravity Columns…

- Design gravity columns as per Section 11.
  - Gravity columns in buildings shall be detailed according to **11.1** and **11.2** for bending moments induced when subjected to "R" times the design lateral displacement under the factored equivalent static design seismic loads given by IS 1893 (Part 1).

HOW TO….

- Run the model for R times lateral loads and check   gravity columns for these bending moments. Its almost always still an insignificant stress ratio and more economical to design for, than providing ductile detailing.

What's not new but has been ignored all along…

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

What IS 456 say about walls..

32.2.5 *Design Axial Strength of Wall*

The design axial strength P_{uw} per unit length of a braced wall in compression may be calculated from the following equation:

$$P_{uw} = 0.3\,(t - 1.2\,e - 2e_a)\,f_{ck}$$

where

t = thickness of the wall,

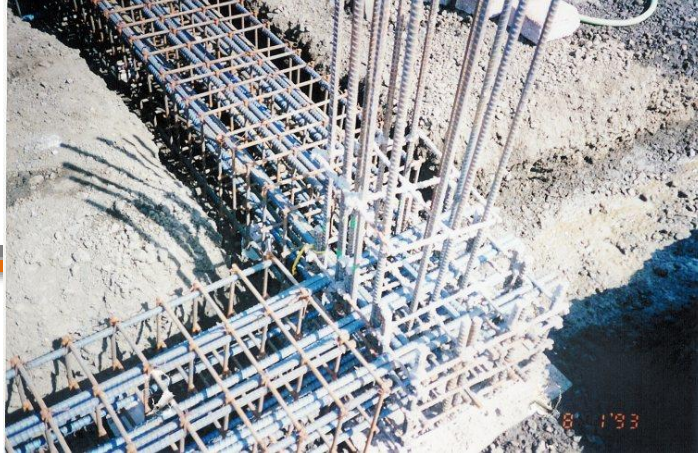
e = eccentricity of load measured at right angles to the plane of the wall determined in accordance with **32.2.2**, and

e_a = additional eccentricity due to slenderness effect taken as $H_{we}^2 / 2\,500\,t$.

WALLS or COLUMN?

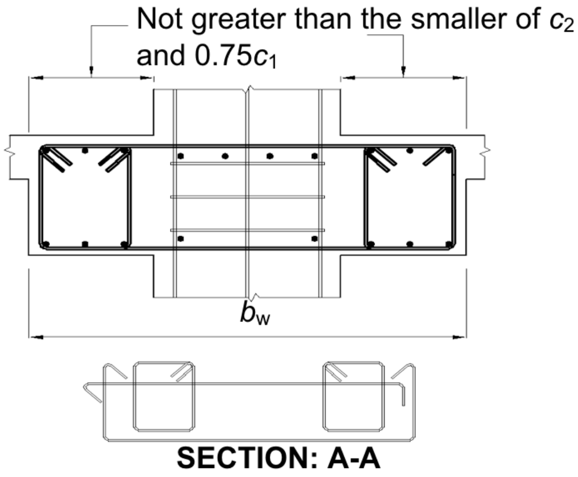
- WHAT THIS MEANS
  - Any member that is desired to be defined as a wall as per IS 456 may not have axial stress ratio greater than say $0.28f_{ck}$
  - that is to say it **cannot** be governed by min reinforcement of ρ_v =0.12% and ρ_h=0.2% if the above axial stress ratio is exceeded
  - In such a case all requirements of Columns as per IS 456 are applicable (min $\rho_{v\,=}$0.8% ....)???

Beam Width



For the first time, explicitly acknowledged that beam may be wider than column as long as transverse reinforcement in continued into beam column joint.

Great relief for some situations and to ease congestion in beam-column joint.



Mechanical Couplers

- Use of mechanical couplers allows for modular and mechanised construction-
- Lapping bars anywhere and all together

Min Link Diameter

- Link min dia for beams and columns to be 8 mm.
- Encouraging better shear capacity
- Acknowledging the corrosion effect on shear capacity of members

Columns

Probably the most important change in the code

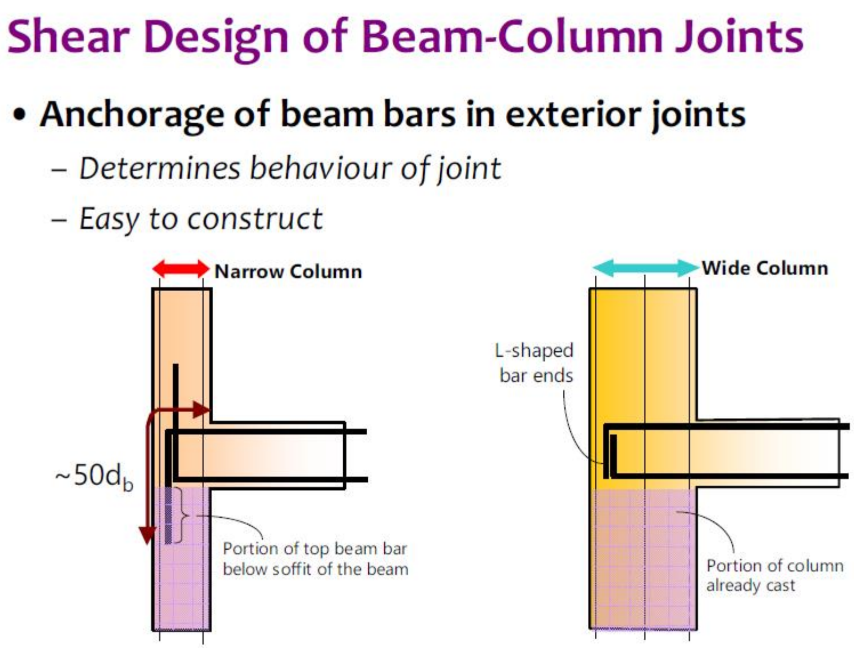
- The factored axial compressive stress considering all load combinations relating to seismic loads shall be limited to 0.40 f_{ck} in all such members, except in those covered under **10**.
- This will perforce require larger sectional size of column or use of higher mix of concrete!

Limiting Column Axial Stress Ratio

- Lesser axial stress ratio will result in lesser percentage reinforcement demand even in case of higher bending stress and will improve column behaviour.
- Special Confining reinforcement- Max spacing of links = 100mm

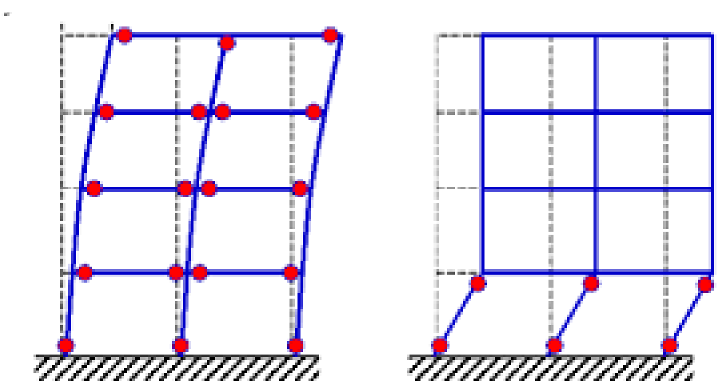
Ensuring Development of Beam Reinforcement

- Min dimension of column =20 d_b, db = max beam bar dia for exterior columns
- E.g. Beam Bar dia=20, Column Size=400



Stress on Collapse Mechanism

- Distributed damage in all storeys   All damage in one storey



(a) Global failure mechanism   (b) Local failure mechanism

Collapse Mechanism

- **Flexural Strength Ratio**

$$\Sigma M_{uc} > \mathbf{1.4}\ \Sigma M_{ub}$$

SHEAR WALLS

- MIN thickness =300 for coupled walls to avoid congestion due to coupling beam steel.
- Special Shear walls must be founded on foundations- i.e. floating special shear walls are not allowed!!!

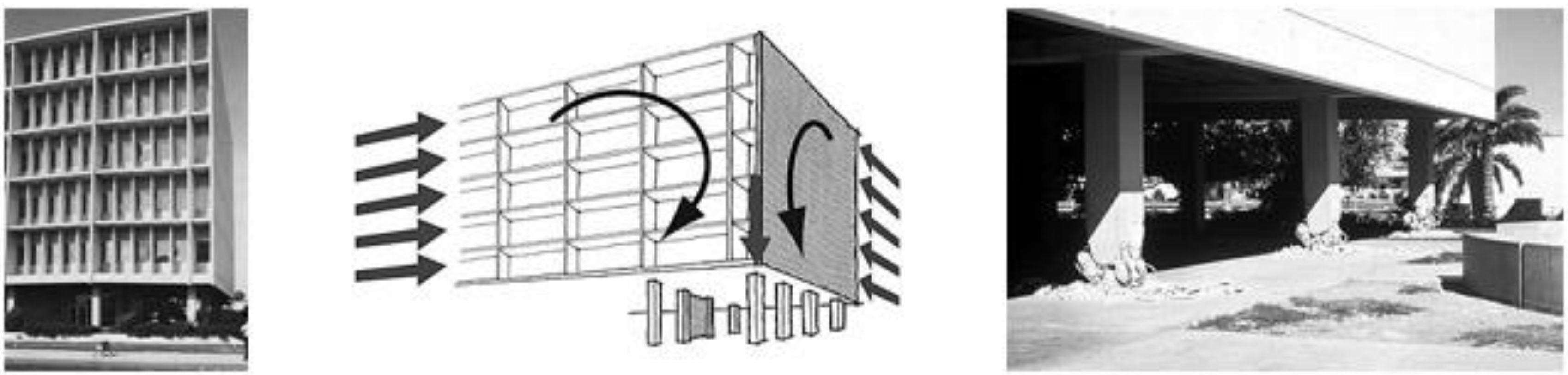


Figure 4.3. Left: corner column; center: Arnold, Ch. (1982) and V. Bertero (1997); right: (Photos: V. Bertero).

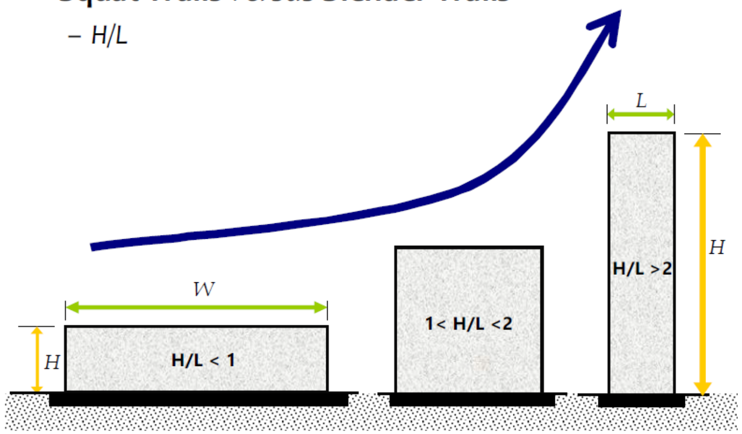
Shear walls

Wall

Sl. No.	Type of Wall	Reinforcement Details
i)	Squat walls	$(\rho_h)_{min} = 0.0025$ $(\rho_v)_{min} = 0.0025 + 0.5\left(1 - \frac{h_w}{t_w}\right)(\rho_h - 0.0025)$ L_w $(\rho_{v,net}) = (\rho_{v,web}) + \left(\frac{t_w}{L_w}\right).[0.02 - 2.5(\rho_{v,web})]$ $(\rho_v)_{provided} < (\rho_h)_{provided}$
ii)	Intermediate walls	$(\rho_h)_{min} = 0.0025$ $(\rho_{v,be})_{min} = 0.0080$ $(\rho_{v,web})_{min} = 0.0025$ $(\rho_{v,net})_{min} = 0.0025 + 0.01375\left(\frac{t_w}{L_w}\right)$.
iii)	Slender walls	$(\rho_h)_{min} = 0.0025 + 0.5\left(\frac{h_w}{L_w} - 2\right)(\rho_h - 0.0025)$ $\rho_{v,web}$ $(\rho_{v,be})_{min} = 0.0080$ $(\rho_{v,web})_{min} = 0.0025$ $(\rho_{v,net})_{min} = 0.0025 + 0.01375\left(\frac{t_w}{L_w}\right)$.

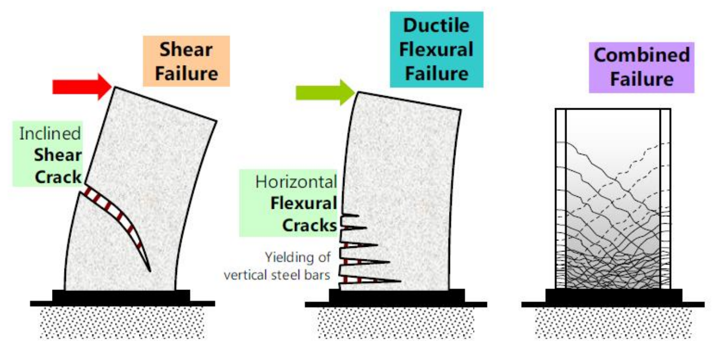
RC Structural Walls

- **Squat Walls** *versus* **Slender Walls**
  - H/L



RC Structural Wall

- **Reinforcement detailing**



Shear Wall Boundary Elements

Much relief in Boundary Elements design. More relaxed than SMRF columns

ii) in rectangular links:

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

For columns

10.4.4 Boundary elements, where required as per **10.4.1**, shall be provided with special confining reinforcement throughout their height, given by

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

and have a spacing not more than

i) 1/3 of minimum member dimension of the boundary element ,
ii) 6 times diameter of the smallest longitudinal reinforcement bars,
iii) 100 mm but may be relaxed to 150 mm if maximum distance between cross-ties/parallel legs of links or ties is limited to 200 mm,

but need not be less than 100 mm.

Shear Wall Boundary Elements

Much relief in Boundary Elements design. More relaxed than SMRF columns

ii) in rectangular links:

$$A_{sh} = Max \begin{bmatrix} 0.18\, s_v\, h\, \dfrac{f_{ck}}{f_y} \left(\dfrac{A_g}{A_k} - 1 \right) & 1 \\ 0.05\, s_v\, h\, \dfrac{f_{ck}}{f_y} & 2 \end{bmatrix}$$

For columns

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

For walls boundary elements

sv	100	mm	
h	160	mm	
fck	30	N/mm2	
fy	500	N/mm2	
Ag	2800	cm2	40x70
Ak	1800	cm2	30x60
Ash	96	mm	1
	48	mm	2

Gravity Columns confinement and shear capacity requirements

- **11.1** The provisions in **11.1.1** and **11.1.2** shall be satisfied, when induced bending moments and horizontal shear forces under the said lateral displacement combined with factored gravity bending moment and shear force do not exceed the design moment of resistance and design lateral shear capacity of the column.
  - **11.1.1** Gravity columns shall satisfy **7.3.2**, **7.4.1** and **7.4.2.** But, spacing of links along the full column height shall not exceed 6 times diameter of smallest longitudinal bar or 150 mm.
  - **11.1.2** Gravity columns with factored gravity axial stress exceeding $0.4f_{ck}$ shall satisfy **11.1.1** and shall have transverse reinforcement at least one half of special confining reinforcement required by **8**.

Acknowledging confinement effect of joints

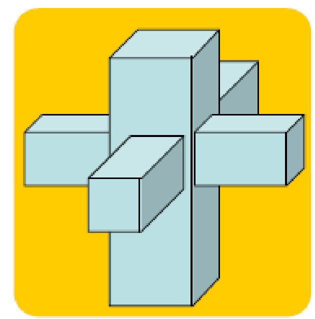
Shear Design of Beam-Column Joints

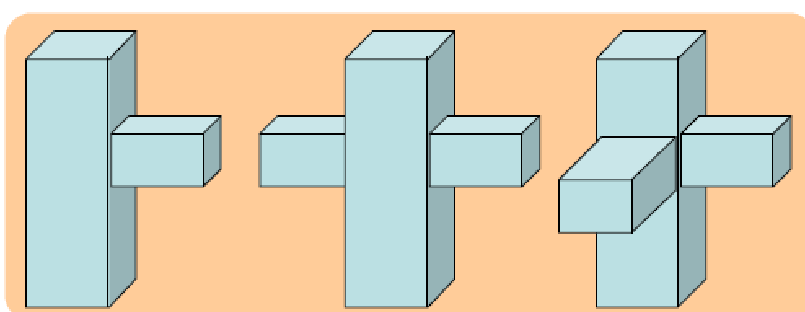
- **Nominal Shear Stress τ_{jv}**
  - *Within Shear Strength τ_{jc} of Concrete in a Joint*

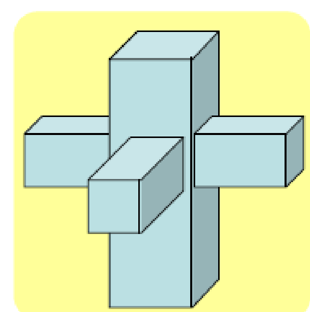
9.1.1 *Shear Strength of Concrete in a Joint*

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

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







Graphics of slides 49, 52, 56 courtesy CVR Murty