CHAPTER 18

Water Tank

CONTENTS.

18. Water Tank.

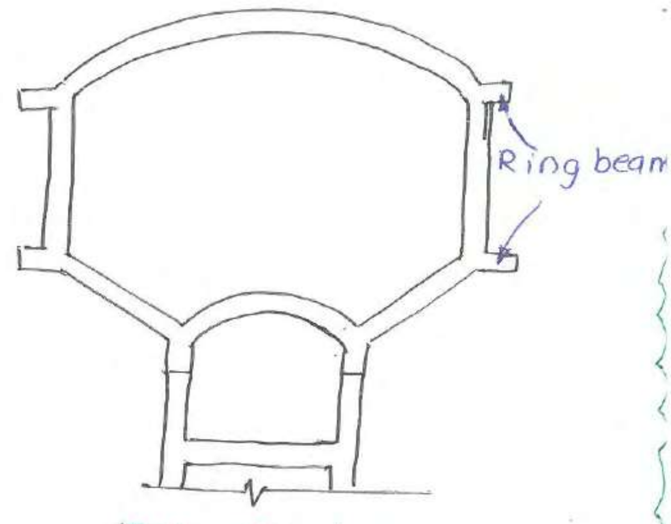
18.1 Classification of Water Tank:

A) Based on Elevation:

1. Under ground
2. Resting on ground.
3. Elevated.

B) Based on Shape

1. Circular/ Cylindrical
2. Rectangular.
3. Intze Tank

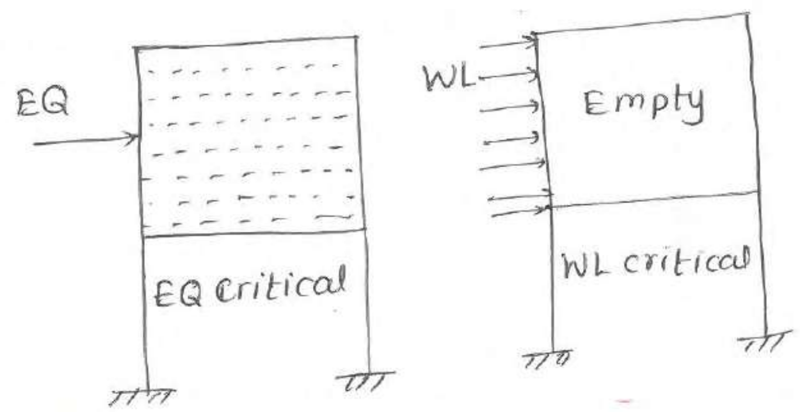


18.2 Codal Provisions of IS 3370:

1. Permeability of concrete must be least so use lesser value of w/c ratio.
2. No porous aggregate should be used.
3. Part of structure retainging liquid and enclosing space above liquid should be taken under severe exposure condition.
4. Maximum cement content is 400 kg/m³ to take care of shrinkage effect
5. Minimum cement content for RCC tank is 320kg/m³
6. Minimum grade of concrete is M30
7. Maximum w/c ratio is 0.45
8. Minimum nominal cover is 45mm
9. Maximum allowed crack width is 0.2mm in LSM design
10. To reduce cracking due to temperature, shrinkage and moisture loss at early stage of concrete, curing should be done for not less than 14 days.

11. Cracking of concrete can be controlled to some extent by maintaining slope filling rate of 1 m in 24 hrs, at the first time of filling.

12. All structures required to retain liquid shall be designed for both empty and full condition.



13. Permissible Stress:

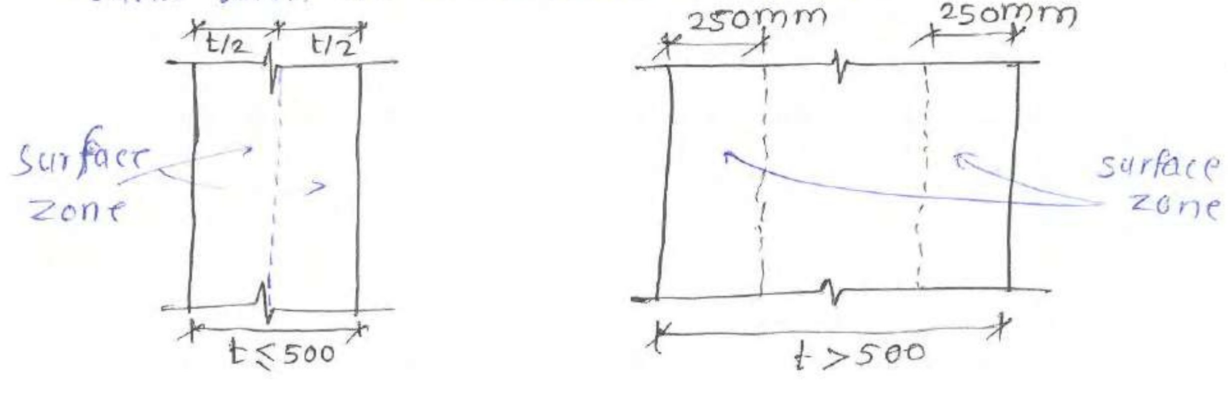
- Steel
  - Mild – 115 N/mm^2
  - HYSD – 130 N/mm^2
- Concrete

Grade	direct tension	bending tension
M25	$1.3\ N/mm^2$	1.8
M30	1.5	2.0

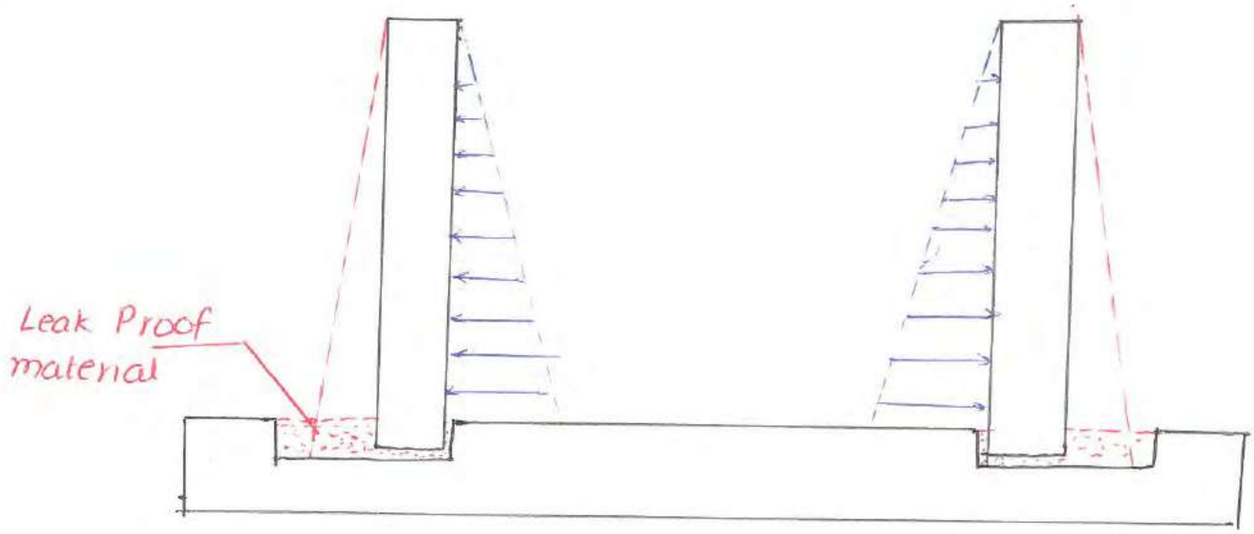
14. If thickness is more than 200 mm the reinforcement is provided in 2 layers, One on each face.

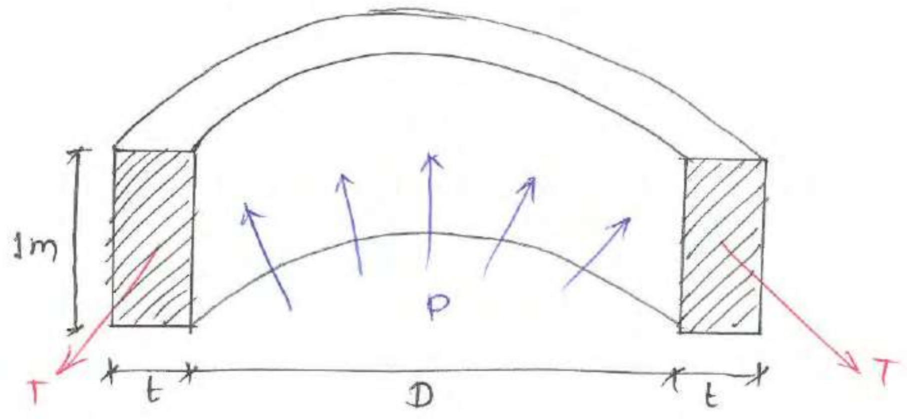
15. Minimum steel is 0.64 % and 0.4 % of surface zone for mild steel and HYSD respectively

It can be reduced to 0.35 % and 0.24 % for tank with no dimension more than 15 m.



18.3 Design of Circular Tank with Flexible joint between Base slab and Wall:



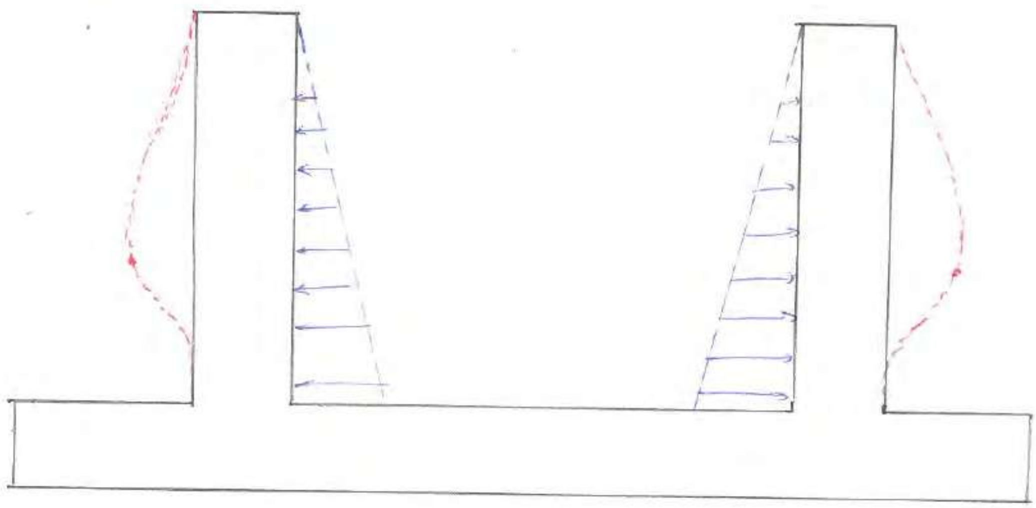


$$2T = p\,(D \times 1)$$

$$T = \frac{pD}{2}$$

- Wall is subjected to hoop tension only, so it is designed only for axial tension.

18.4 Design of Circular Tank with Rigid Joint between Base slab and wall:



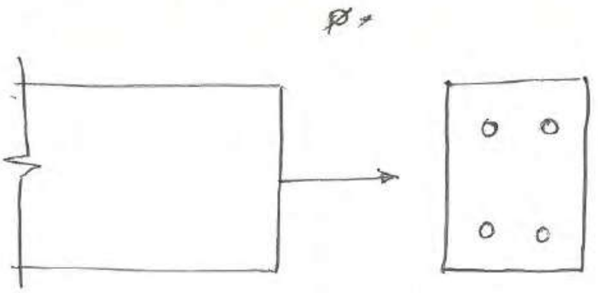
- Upperportion is primarily subjected to hoop tension and bottom portion behaves like cantilever
- Wall is designed for BM and axial tension both.

18.5 Design of Member Subjected to Axial Tension

Member is designed in such a way entire force is assumed to be transfered through steel only.

$$A_{st} = \frac{P}{\sigma_{st}}$$

Section size should be such that tensile stress of concrete should not exceed its permissible stress.



$$P = P_c + P_s$$
$$= f_{ct} \cdot A_c + f_{st} \cdot A_{st}$$
$$P = f_{ct}(A_g - A_{st}) + (mf_c)A_{st}$$

$$f_{ck} = \frac{P}{A_g + (m-1)A_{st}} \leq \sigma_{ct}$$

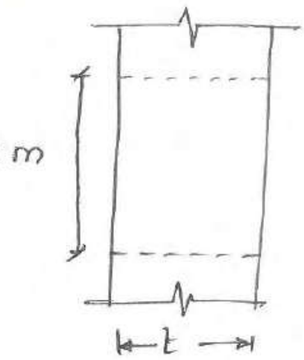
Ex. Calculate thickness of wall and area of steel required for axial tension 250 kN/m. M30, Fe415, $m = 9.33$

$\Rightarrow$

$$A_{st} = \frac{P}{\sigma_{st}}$$

$$= \frac{250 \times 10^3}{130}$$

$$A_{st} = 1923.07 \text{ mm}^2$$



Assuming thickness $= t = 100$ mm

$$f_{ct} = \frac{P}{A_g + (m-1)A_{st}}$$

$$= \frac{250 \times 10^3}{(100 \times 1000) + (9.33 - 1) \times 1923.07}$$

$$= 2.15 \text{ N/mm}^2 > \sigma_{ct} \ (1.5 \text{ N/mm}^2)$$

Safe but unserviceable.

Now, assuming $t = 250$ mm

$$f_{ct} = \frac{P}{A_g + (m-1)A_{st}}$$

$$= \frac{250 \times 10^3}{(250 \times 1000) + (9.33 - 1) \times 1923.07}$$

$$f_{ct} = 0.93 \text{ N/mm}^2 < \sigma_{ct} \ (1.5 \text{ N/mm}^2)$$

Safe and serviceable.

.... Chapter 18 Ends Here....

STAIRCASE

- Width – 1 to 2m,   Steps – 3 to 12 nos., T+2R = 500,   T*R = 40000 to 42000
- Residential
  - T = 250 to 300
  - R = 150 to 180
- Public
  - T = 250 to 300
  - R = 120 to 150

Width:

1m

2.0m → Public building

Minimum Steps = 3 Nos

Maximum Steps = 12 Nos.

Tread

250mm

300mm

Riser:

≥150mm Residential

≤150mm Public

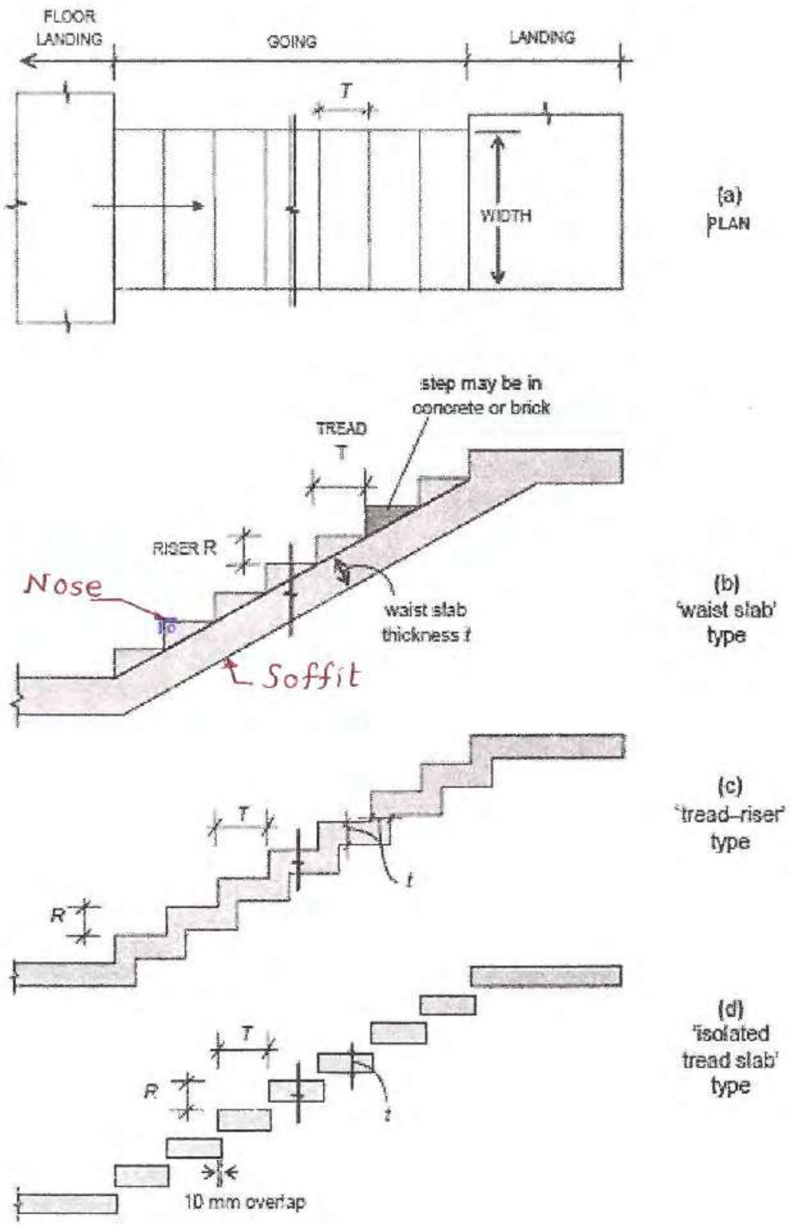


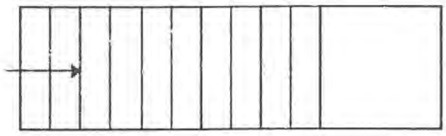
Fig. 12.1 A typical flight in a staircase

12.2 TYPES OF STAIRCASES

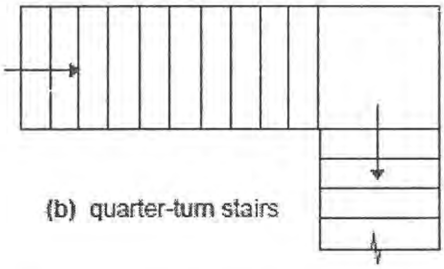
12.2.1 Geometrical Configurations

A wide variety of staircases are met with in practice. Some of the more common geometrical configurations are depicted in Fig. 12.2. These include:

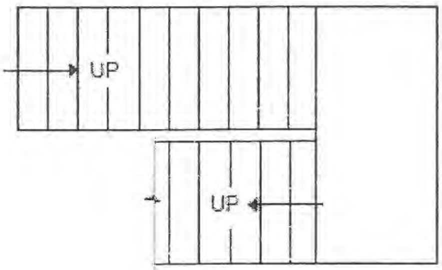
- straight stairs (with or without intermediate landing) [Fig. 12.2(a)]
- quarter-turn stairs [Fig. 12.2(b)]
- dog-legged stairs [Fig. 12.2(c)]
- open well stairs [Fig. 12.2(d)]
- spiral stairs [Fig. 12.2(e)]
- helicoidal stairs [Fig. 12.2(f)]



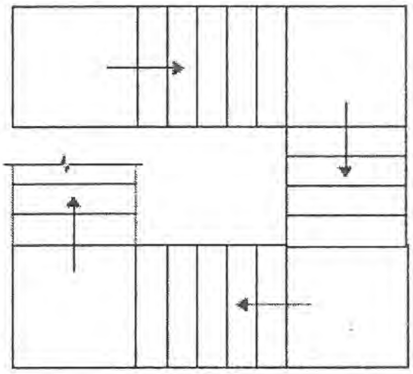
(a) straight stairs



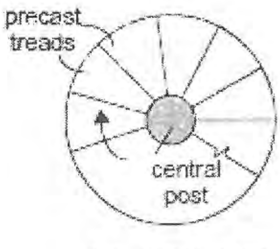
(b) quarter-turn stairs



(c) dog-legged stairs

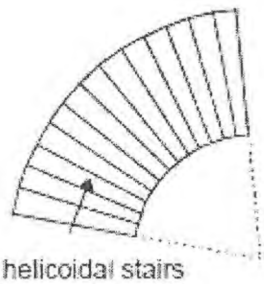


(d) open-well stairs

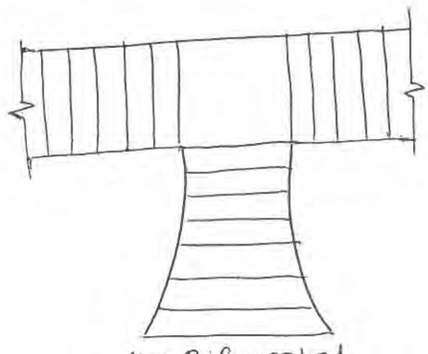


(e) spiral stairs

PLAN VIEWS



(f) helicoidal stairs



(g) Bifurcated

12.2.2 Structural Classification

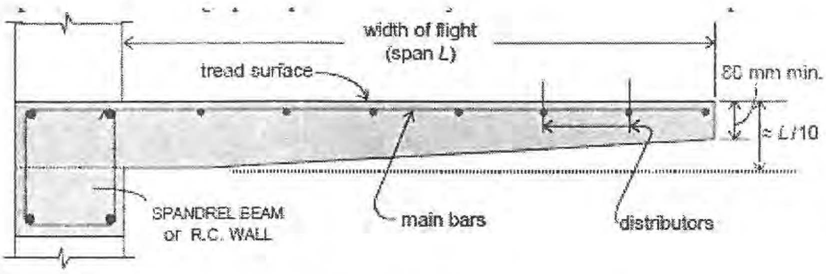
Structurally, staircases may be classified largely into two categories, depending on the predominant direction in which the slab component of the stair undergoes flexure:

1. stair slab spanning transversely (stair widthwise);
2. stair slab spanning longitudinally (along the incline).

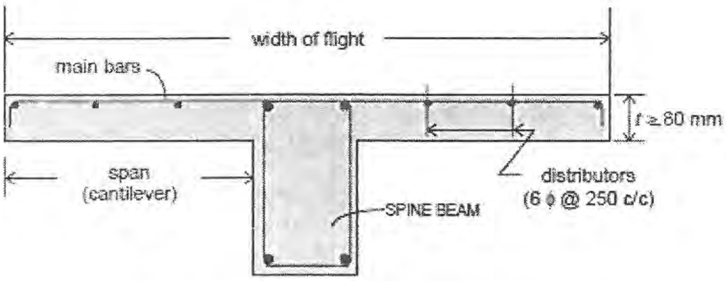
Stair Slab Spanning Transversely

This category generally includes:

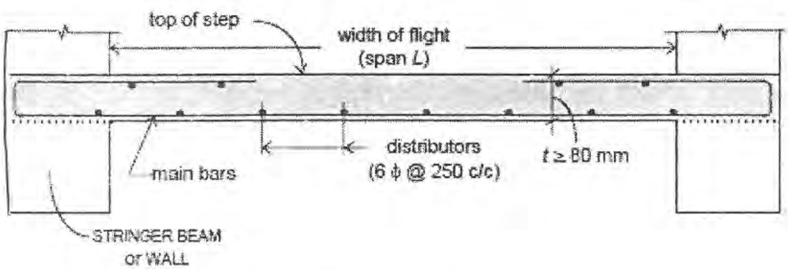
1. slab cantilevered from a spandrel beam or wall [Fig. 12.3(a)];
2. slab doubly cantilevered from a central spine beam [Fig. 12.3(b)];
3. slab supported between two stringer beams or walls [Fig. 12.3(c)].



(a) slab cantilevered from a spandrel beam or wall

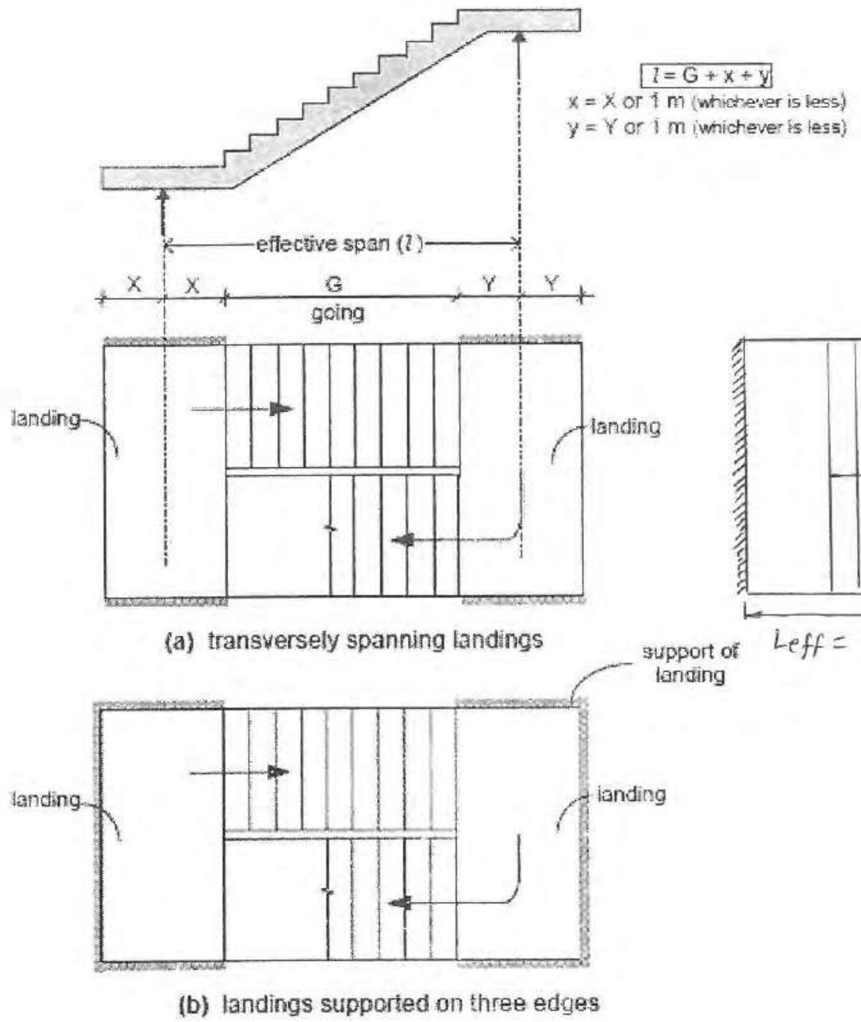


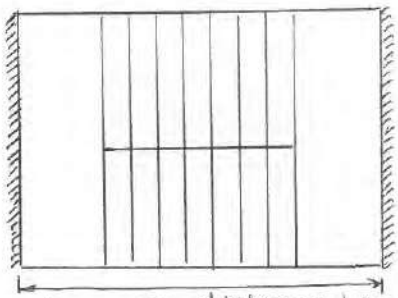
(b) slab doubly cantilevered from a central spine beam



(c) slab supported between two stringer beam or walls

STAIRCASE SPANNING LONGITUDINALLY

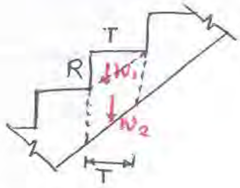


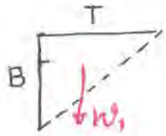


L_{eff} = c/c distance b/w supports

Fig. 12.5 Special support conditions for longitudinally spanning stair slabs

For W_1:





$$W_1 = \frac{\frac{1}{2} \times R \times T \times B \times 25}{T} \text{ kN/m}$$

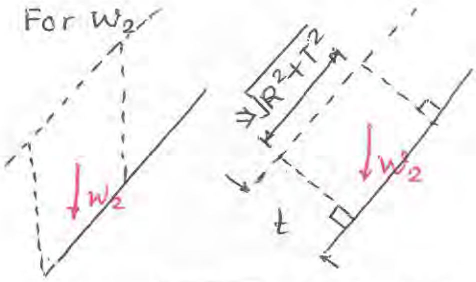
12.3.1 Dead Loads

The components of the dead load to be considered comprise:

- self-weight of stair slab (tread/tread-riser slab/waist slab);
- self-weight of step (in case of 'waist slab' type stairs);
- self-weight of tread finish (usually 0.5 – 1.0 kN/m^2)

The unit weight of reinforced concrete for the slab and step may be taken as 25kN/m^3 as specified in the Code (Cl. 19.2.1).

For W_2:



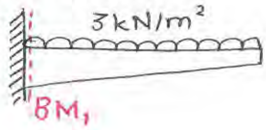
$$W_2 = \frac{\sqrt{R^2+T^2} \times t \times B \times 25}{T} \text{ kN/m}$$

$t =$

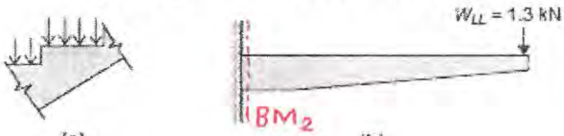
$B =$

12.3.2 Live Loads

Live loads are generally assumed to act as uniformly distributed loads on the horizontal projection of the flight, i.e., on the 'going'. The Loading Code [IS 875 : 1987 (Part II)] recommends a uniformly distributed load of 5.0 kN/m^2 in general, on the going, as well as the landing. However, in buildings (such as residences) where the specified floor live loads do not exceed 2.0 kN/m^2, and the staircases are not liable to be overcrowded, the Loading Code recommends a lower live load of 3.0 kN/m^2 [Fig. 12.6(a)].



$$w_{LL} = \begin{cases} 5.0 \text{ kN/m}^2 & \text{in general} \\ 3.0 \text{ kN/m}^2 & \text{when overcrowding is unlikely} \end{cases}$$



$$\text{Design BM} = \text{Maximum} \begin{cases} BM_1 \\ BM_2 \end{cases}$$

Fig. 12.6 Code specifications for live loads on stair slabs

Further, in the case of *structurally independent cantilever steps*, the Loading Code requires the tread slab to be capable of safely resisting a concentrated live load of 1.3 kN applied to the free end of each cantilevered tread [Fig. 12.6(b)].

It may be noted that the specified live loads are *characteristic* loads; these loads as well as the characteristic dead loads should be multiplied by the appropriate *load factors* in order to provide the *factored loads* required for 'limit state design'.

In the case of stairs with open wells, where spans partly crossing at right angles occur, the load on areas common to any two such spans may be taken as one-half in each direction as shown in Fig. 18. Where flights

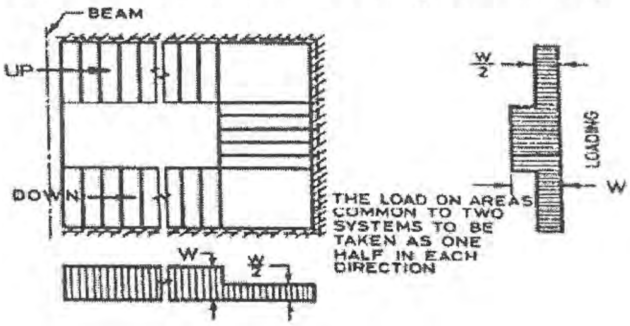
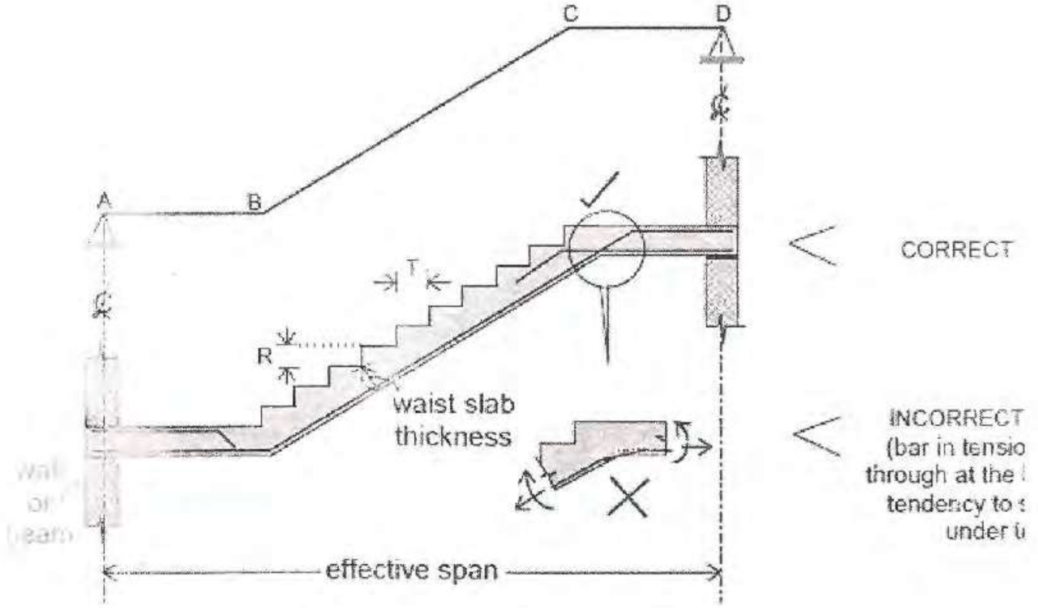


FIG. 18 LOADING ON STAIRS WITH OPEN WELLS



EXAMPLE 12.5

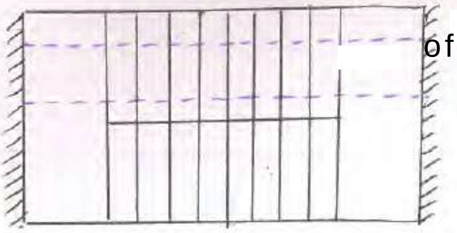
Design a ('waist slab' type) dog-legged staircase for an office building, given the following data:

- height between floor = 3.2 m;
- riser = 160 mm, tread = 270 mm;
- width of flight = landing width = 1.25 m
- live load = 5.0 kN/m^2
- finishes load = 0.6 kN/m^2

Assume the stairs to be supported on 230 mm thick masonry walls at the outer edges of the landing, parallel to the risers [Fig. 12.13(a)]. Use M 20 concrete and Fe 415 steel. Assume *mild* exposure conditions.

SOLUTION

- Given: $R = 160$ mm, $T = 270$ mm $\Rightarrow \sqrt{R^2 + T^2} = 314$ mm
  Effective span = c/c distance between supports = 5.16 m [Fig. 12.13(a)].
- Assume a waist slab thickness $\approx l/20$ = 5160/20 = 258 → 260 mm.
  based on experience
  Assuming 20 mm clear cover (*mild* exposure) and 12 φ main bars,
  effective depth $d = 260 - 20 - 12/2 = 234$ mm.
  The slab thickness in the landing regions may be taken as 200 mm as the bending moments are relatively low here.
  D



1m A strip of unit width is designed

- *Loads on going* [Ref. 12.13(b)] on projected plan area:

(1)	self-weight of waist slab @ $25 \times 0.26 \times 314/270$		$= 7.56$ kN/m^2
(2)	self-weight of steps @ $25 \times \left(\frac{1}{2} \times 0.16\right)$		$= 2.00$ "
(3)	finishes	(given)	$= 0.60$ "
(4)	live load	(given)	$= 5.00$ "
			15.16 kN/m^2

$\Rightarrow$ Factored load $= 15.16 \times 1.5 = 22.74$ kN/m^2

- *Loads on landing*

(1)	self-weight of slab @ $25 \times 0.20 = 5.00$ kN/m^2	
(2)	finishes	@ 0.6 "
(3)	live loads	@ 5.0 "
		10.60 kN/m^2

$\Rightarrow$ Factored load $= 10.60 \times 1.5 = 15.90$ kN/m^2

- *Design Moment* [refer Fig. 12.13(b)]

Reaction $R = (15.90 \times 1.365) + (22.74 \times 2.43)/2 = 49.33$ kN/m

Maximum moment at midspan:

$$M_u = (49.33 \times 2.58) - (15.90 \times 1.365) \times (2.58 - 1.365/2) - (22.74) \times (2.58 - 1.365)^2/2$$

$$= 69.30 \text{ kNm/m}$$

- *Main reinforcement*

$M_{u,lim} = 0.138 f_{ck} bd^2$

$= 0.138 \times 20 \times 1000 \times 234^2$

$M_{u,lim} = 151.12$ kNm $\Rightarrow$ So, $M_u < M_{u,lim}$

$R = \frac{M_u}{bd^2} = \frac{69.30 \times 10^6}{10^3 \times 234^2} = 1.265$ MPa

Assuming $f_{ck} = 20$ MPa, $f_y = 415$ MPa,

$\frac{p_t}{100} = \frac{A_{st}}{bd} = \frac{20}{2 \times 415}\left[1 - \sqrt{1 - 4.598 \times 1.265/20}\right] = 0.381 \times 10^{-2}$

[This may also be obtained from design aids Table 3(a)].

$\Rightarrow (A_{st})_{reqd} = (0.381 \times 10^{-2}) \times 10^3 \times 234 = 892$ mm^2/m

$A_{st} = \frac{0.5 f_{ck} bd^2}{f_y}\left[1 - \sqrt{1 - \frac{4.6 B M_u}{f_{ck} b d^2}}\right]$

$= \frac{0.5 \times 20 \times 1000 \times 234}{415} \times \left[1 - \sqrt{1 - \frac{4.6 \times 69.3 \times 10^6}{20 \times 1000 \times 234^2}}\right]$

$A_{st} = 892$ mm^2

Required spacing of 12 ϕ bars $= \frac{113 \times 10^3}{892} = 127$ mm

Required spacing of 16 ϕ bars $= \frac{201 \times 10^3}{892} = 225$ mm (to be reduced slightly to account for reduced effective depth)

Provide 16 ϕ @ 220c/c

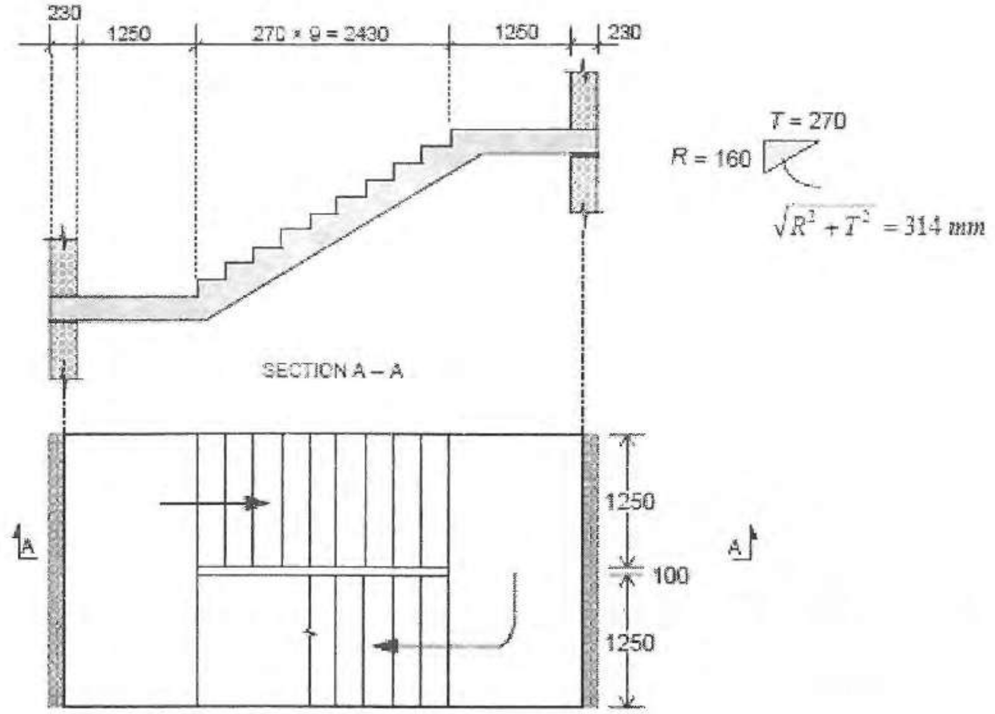
- *Distributors*

$(A_{st})_{reqd} = 0.0012\, bt$ (for Fe 415 bars)

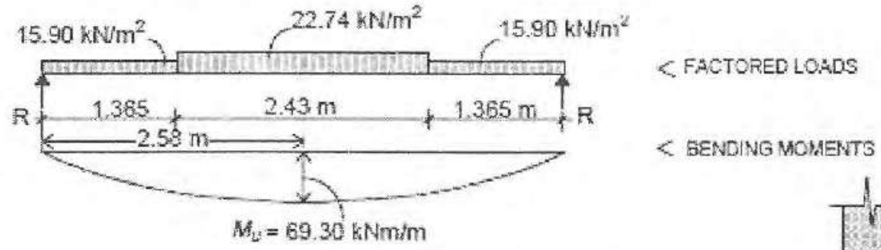
$= 0.0012 \times 10^3 \times 260 = 312$ mm^2/m

spacing 10 ϕ bars $= 78.5 \times 10^3 / 312 = 251$ mm

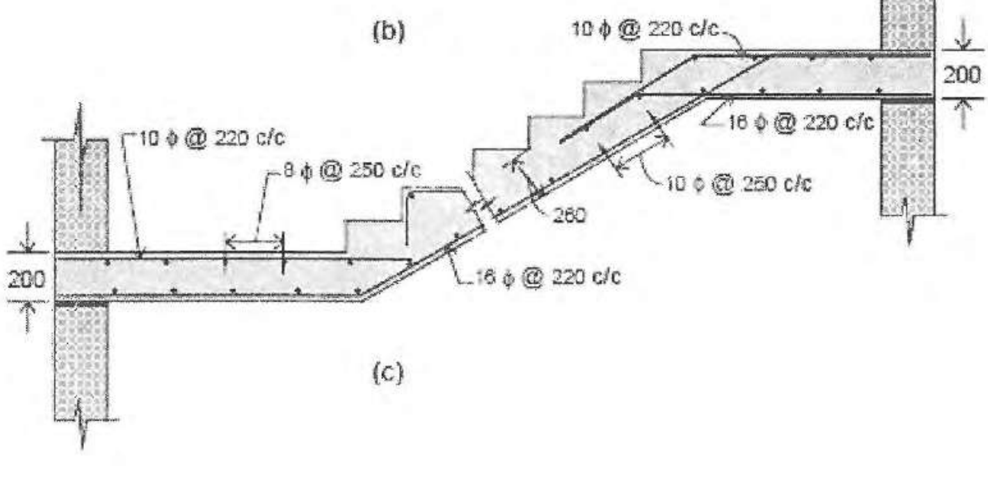
Provide 10 ϕ @ 250c/c as distributors


230
1250
270 × 9 = 2430
1250
230
T = 270
R = 160
$\sqrt{R^2 + T^2} = 314\ mm$
SECTION A – A
A
A
1250
100
1250

(a)


15.90 kN/m²
22.74 kN/m²
15.90 kN/m²
< FACTORED LOADS
R
1.365
2.43 m
1.365 m
R
2.58 m
< BENDING MOMENTS
M_u = 69.30 kNm/m

(b)


10 φ @ 220 c/c
200
16 φ @ 220 c/c
10 φ @ 220 c/c
8 φ @ 250 c/c
10 φ @ 250 c/c
260
200
16 φ @ 220 c/c

(c)

Fig. 12.13 Example 12.5

The detailing of bars for the first flight is shown in Fig. 12.13(c). Some nominal reinforcement (10 ϕ @ 220c/c) is provided in the landing slabs near the support at top to resist possible 'negative' moments on account of partial fixity; 8 ϕ @ 250 c/c distributors are also provided.

EXAMPLE 12.6

Repeat the problem of the dog-legged staircase in Example 12.5, considering the landings to be supported only on two edges perpendicular to the risers [Fig. 12.14(a)].

SOLUTION

- The prevailing IS Code recommendations are adopted here for determination of the design moments[†].
- Given: $R = 160$ mm, $T = 270$ mm $\Rightarrow \sqrt{R^2 + T^2} = 314$ mm
  As the flight is supported on the landings (whose length is less than 2.0 m), the effective span (as per Code) is given by the c/c distance between landings.
  $l = 2.43 + 1.25 = 3.68$ m
- Assume a waist slab thickness $\approx 3680/20 = 184 \rightarrow 185$ mm.
  Let thickness of the landing slabs also be 185 mm
  Assuming 20 mm cover and 12 ϕ bars, $d = 185 - 20 - 12/2 = 159$ mm
- *Loads on going* [Ref. 12.14(b)] on projected plan area:

(1)	self-weight of waist slab @ $25 \times 0.185 \times 314/270$		$= 5.38$ kN/m^2
(2)	self-weight of steps @ $25 \times \left(\frac{1}{2} \times 0.16\right)$		$= 2.00$ "
(3)	finishes	(given)	$= 0.60$ "
(4)	live load	(given)	$= 5.00$ "
			12.98 kN/m^2

  $\Rightarrow$ Factored load $= 12.98 \times 1.5 = 19.47$ kN/m^2
- *Loads on landing*

(1)	self-weight of slab @ $25 \times 0.185 = 4.63$ kN/m^2	
(2)	finishes	@ 0.60 "
(3)	live loads	@ 5.00 "
		10.23 kN/m^2

  $\Rightarrow$ Factored load $= 10.23 \times 1.5 = 15.35$ kN/m^2
  50% of this load may be assumed to be acting longitudinally, i.e., $15.35 \times 1/2 = 7.68$ kN/m^2 [Fig. 12.14(b)].
  Design of waist slab [refer Fig. 12.14(b)]

Reaction on landing $R = (7.68 \times 0.625) + (19.47 \times 2.43/2) = 28.46$ kN/m

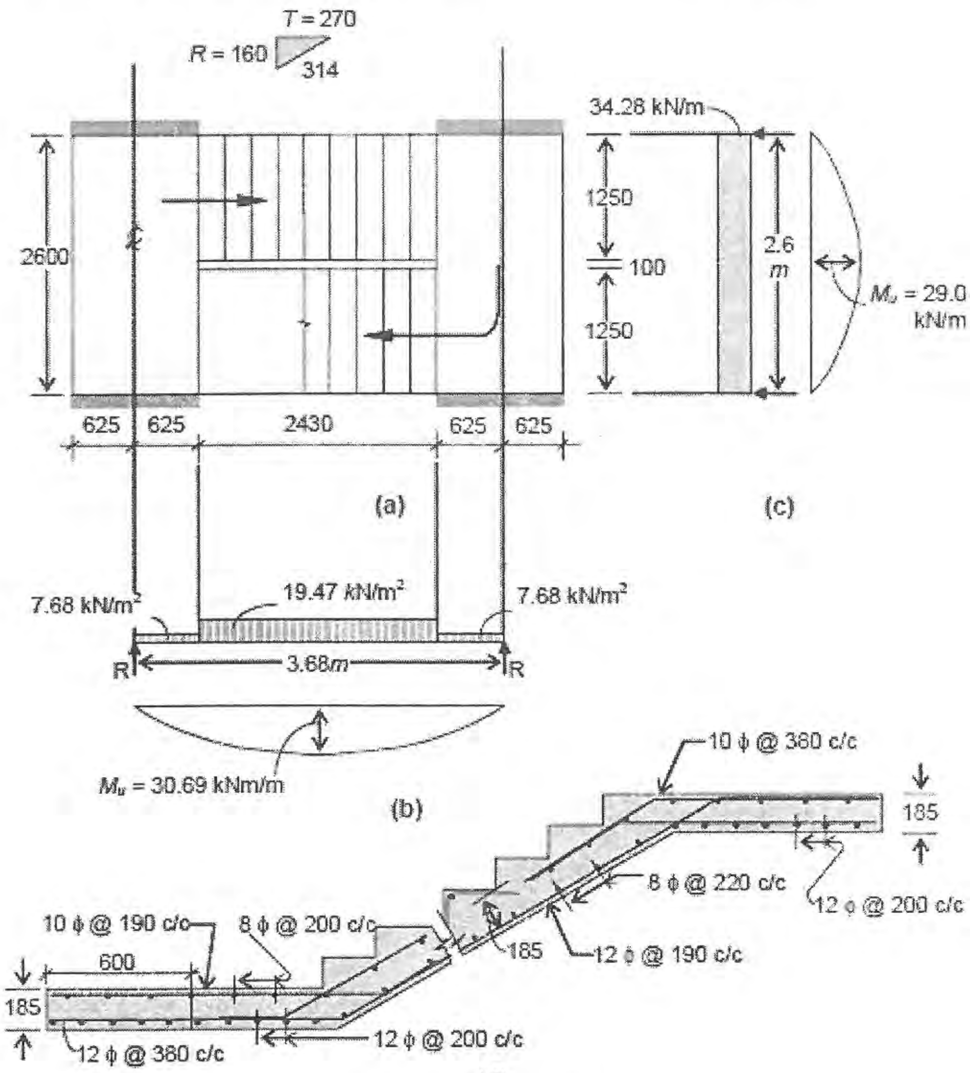


Fig. 12.14 Example 12.6

- *Design Moment at midspan:*
  $M_u = (28.46 \times 3.68/2) - (7.68 \times 0.625) \times (1.84 - 0.625/2) - 19.47 \times 1.215^2/2$
  $= 30.69$ kN/m
- *Main reinforcement*
  $\Rightarrow R \equiv \dfrac{M_u}{bd^2} = \dfrac{30.69 \times 10^6}{10^3 \times 159^2} = 1.214$ MPa
  Assuming M 20 concrete and Fe 415 steel,

$$\frac{p_t}{100} \equiv \frac{A_{st}}{bd} = \frac{20}{2 \times 415}\left[1 - \sqrt{1 - 4.598 \times 1.214/20}\right] = 0.364 \times 10^{-2}$$

$$\Rightarrow (A_{st})_{reqd} = (0.364 \times 10^{-2}) \times 10^3 \times 159 = 579\ \text{mm}^2/\text{m}$$

$$\text{Required spacing of } 12\ \phi \text{ bars} = \frac{113 \times 10^3}{579} = 195\ \text{mm.}$$

Provide 12 ϕ @ 190c/c main bars in the waist slab; these bars are continued into the landing slab, as shown in Fig. 12.14(c). Nominal top steel 10 ϕ @ 190c/c is also provided at top at the junction of the waist slab with the landing slab to resist possible 'negative' moments.

- *Distributors*:

$(A_{st})_{min} = 0.0012 \times 1000 \times 185 = 222\ \text{mm}^2/\text{m}$

$$\text{Required spacing } 8\ \phi \text{ bars} = \frac{503 \times 10^3}{222} = 226\ \text{mm}$$

Provide 8 ϕ @ 220c/c distributors in the waist slab.

Design of landing slabs [refer Fig. 12.14(c)].

The entire loading on the staircase is transmitted to the supporting edges by the bending of the landing slab in a direction parallel to the risers.

- *Loads* (assumed to be uniformly distributed):

(considering the full width of landing of 1.25 m)

(i) directly on landing: 15.35×1.25	$= 19.19$ kN/m
(ii) from going: $19.47 \times 2.43/2$	$= 23.66$ "
	42.85 kN/m

$\Rightarrow$ Loading on 1 m wide strip $= 42.85/1.25 = 34.28$ kN/m

Effective span = 2.60 m

- *Design Moment* (at midspan):

$M_u = 34.28 \times 2.60^2/8 = 29.0$ kNm/m

$$\Rightarrow \frac{M_u}{bd^2} = \frac{29.0 \times 10^6}{10^3 \times 159^2} = 1.147\ \text{MPa}$$

$$\frac{p_t}{100} \equiv \frac{A_{st}}{bd} = \frac{20}{2 \times 415}\left[1 - \sqrt{1 - 4.598 \times 1.147/20}\right] = 0.342 \times 10^{-2}$$

$$\Rightarrow (A_{st})_{reqd} = (0.342 \times 10^{-2}) \times 10^3 \times 159 = 544\ \text{mm}^2/\text{m}$$

$$\text{Required spacing of } 12\ \phi \text{ bars} = \frac{113 \times 10^3}{544} = 207\ \text{mm.}$$

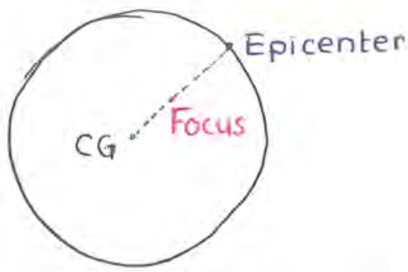
Provide 12 ϕ @ 200 c/c at bottom in a direction parallel to the risers.

- The detailing of the staircase (one typical flight) is depicted in Fig. 12.14(d). Note that the bars from the waist slab are kept above the main bars of the landing slab so that the desired maximum effective depth is obtained for the main bars in the landing slab. This arrangement is *essential* all the more because the waist slab is supported by the landing, and to facilitate effective load transfer, the waist slab

bars must be placed *above* the main bars in the landing. Nominal bars 8 ϕ @ 200 c/c are also provided at top in the landing slabs.

Earthquake Design

- Ground motion is in all three directions. Two orthogonal horizontal directions and one vertical direction.
- Origin point of earthquake inside the earth is called focus and radialy outward point (vertical) on earth surface is called Epicenter.



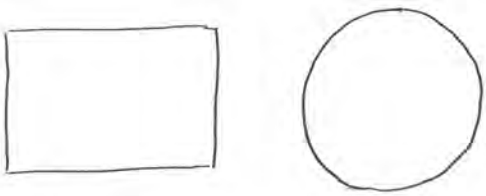
- Regular structures behave better in earthquake than irregular structures. Three types of irregularities exist.
  - Mass irregularity
  - Vertical irregularity
  - Torsional irregularity.

Mass Irregularity

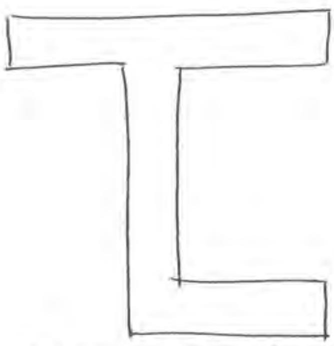
Mass irregularity shall be considered to exist where the seismic weight of any storey is more than 200 percent of that of its adjacent storeys. The irregularity need not be considered in case of roofs

Vertical Geometric Irregularity

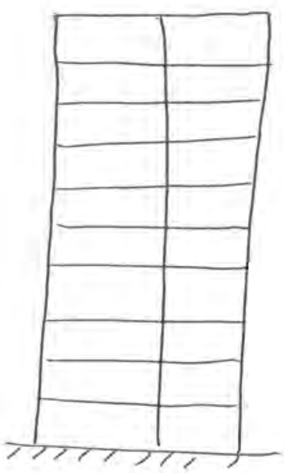
Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral force resisting system in any storey is more than 150 percent of that in its adjacent storey



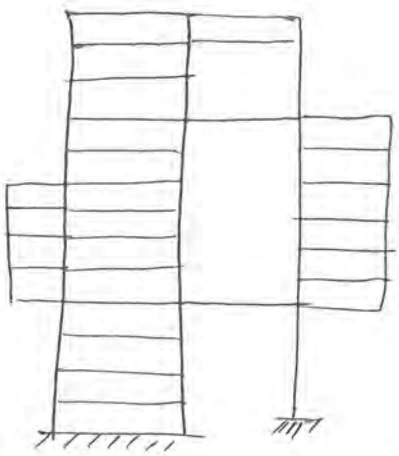
Regular in Plan



Irregular in plan



Regular in Elevation

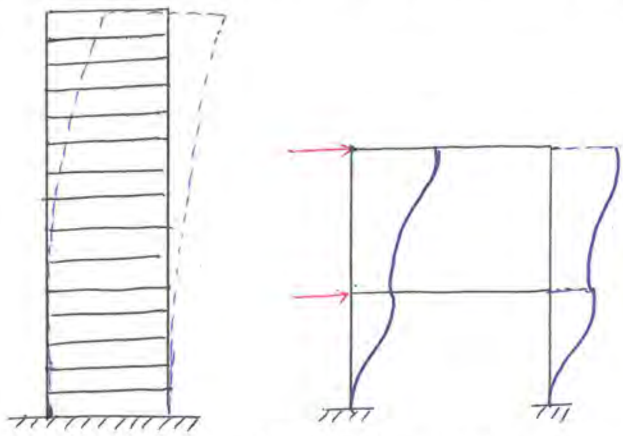


Irregular in Elevation

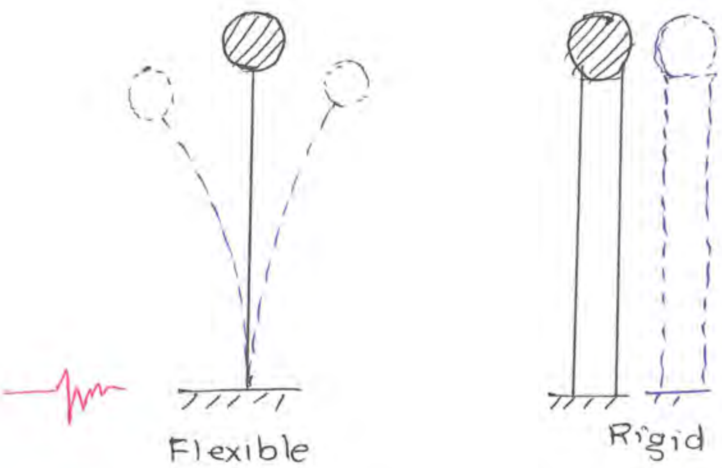
- Magnitude: Earthquake size is a *quantitative* measure of the size of the earthquake at its source.The Richter Magnitude Scale measures the amount of seismic energy released by an earthquake.

- Intensity: It is measured corresponding to damage in any area for given earthquake. The severity of earthquake shaking is assessed using a Mercalli Intensity Scale. When an earthquake occurs, its magnitude can be given a single numerical value on the Richter Magnitude Scale. However the intensity is variable over the area affected by the earthquake, with high intensities near the epicentre and lower values further away. These are allocated a value depending on the effects of the shaking according to the Modified Mercalli Intensity Scale.

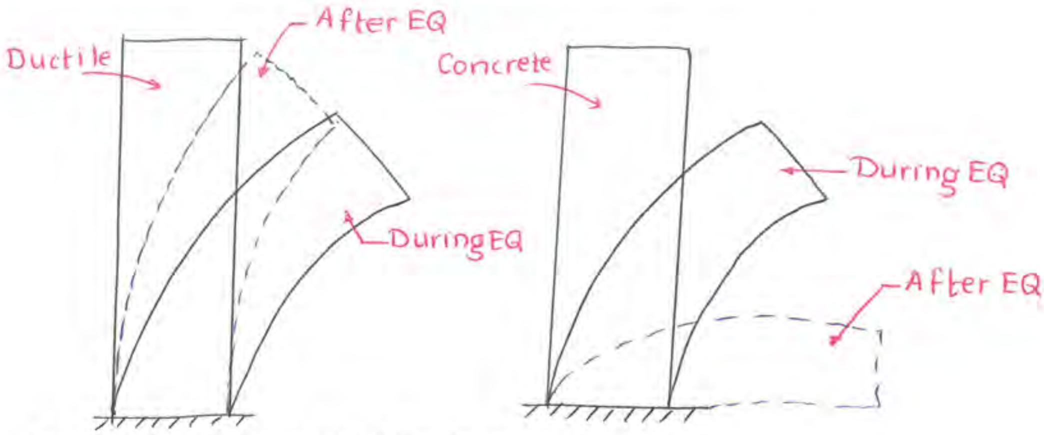
- Tall building behaves as cantilever however short buildings behave as portal frame.



- Flexible structures attract less earthquake force than rigid structure.

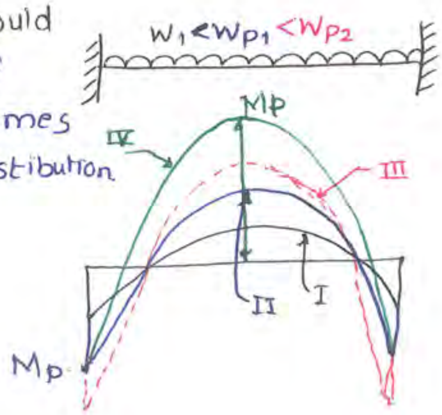


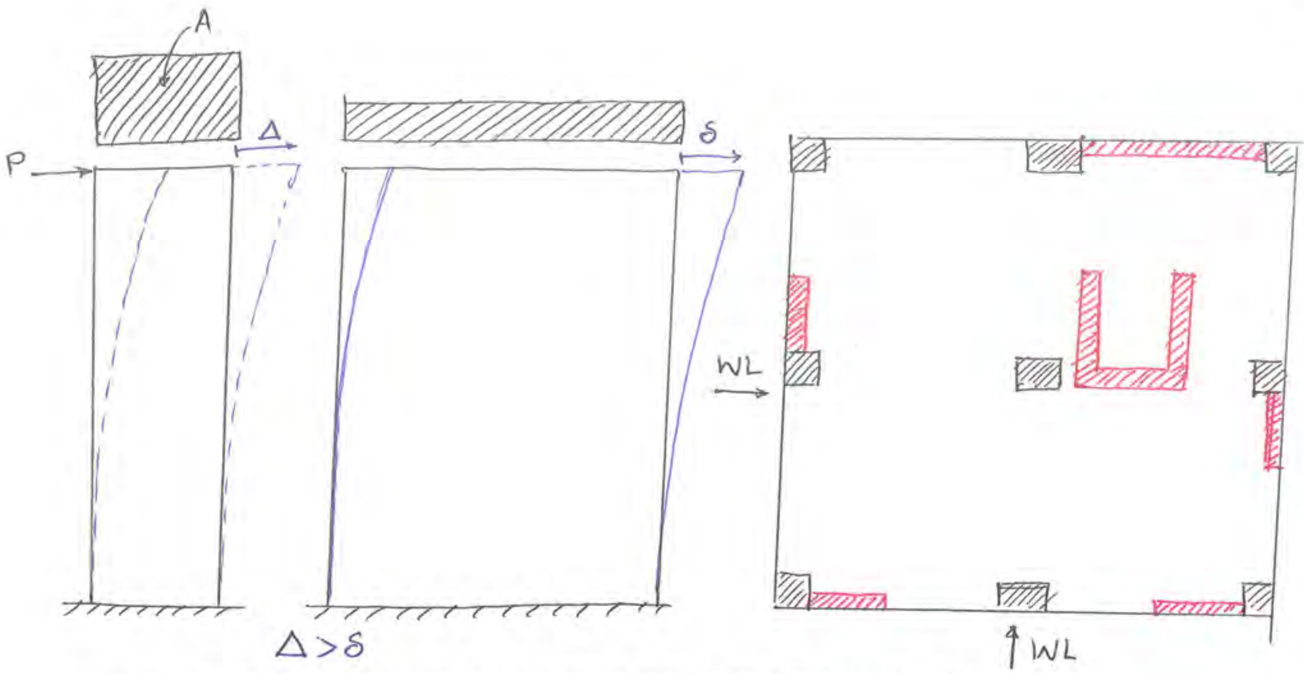
- Ductile structures behave better in earthquake than brittle structure.



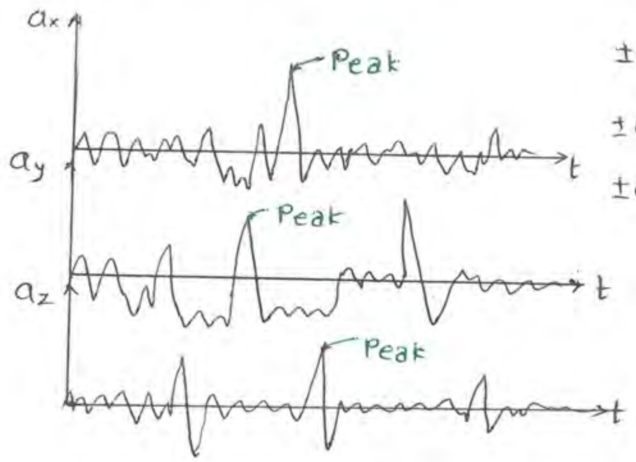
- Ductility can be enhanced by following ways.
    - Decrease in % of tension steel (under reinforced section)
    - Increase in compression steel.
    - Use of mild steel.
    - Use of shear stirrup.
- To maintain overall ductility behavior of structure with minimum damage, it is necessary to achieve
    - Strong foundation and weak superstructure.
    - Strong column and weak beam.
    - Member strong in shear than flexure because shear failure is brittle.
- Maximum 30% moment redistribution is permitted in RCC structure.
- Shear wall is provided to resist lateral load in its own plane only. It is designed for BM, SF and axial force. Thickness should not be less than 150mm.

As load increases from W_{p1} to W_{p2}, BMD should change from II to III. But due to formation of plastic hinges at fixed supports BMD becomes IV instead of III This is called moment redistibution.





- Peak value of earthquake is not taken in all three directions simultaneously.



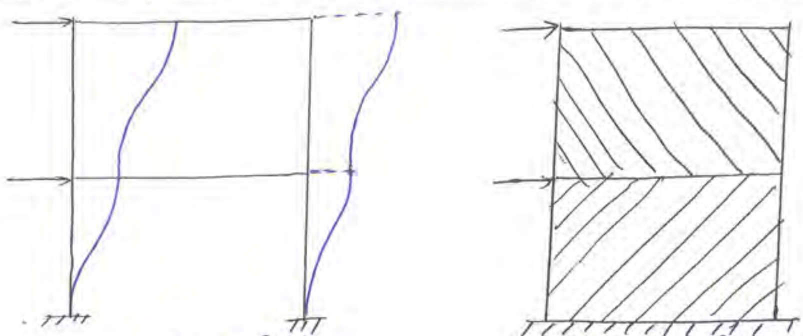
$\pm EL_x \pm 0.3 EL_y \pm 0.3 EL_z$

$\pm 0.3 EL_x \pm EL_y \pm 0.3 EL_z$

$\pm 0.3 EL_x \pm 0.3 EL_y \pm EL_z$

- India is divided in four earthquake zones. Zone II, III, IV & V. zone V is most severe.

- Infill frames provide more lateral resistant than without infill frames.



without infill　　　　with infill

- Soft storey is one in which the sum of lateral stiffness is less than 70% of that in the storey above or less than 80% of the average lateral stiffness of three storey above.

- Extreme soft storey is one in which the sum of lateral stiffness is less than 60% of that in the storey above or less than 70% of the average lateral stiffness of three storey above.

- Damping beyond which vibration is no longer oscillatory is called critical damping.
  - Under damped – vibrates
  - Critically damped – Just no vibration
  - Over damped – Never vibrates
- Criteria adopted by code for fixing level of design seismic load are given below.
  - Structure should be able to resist minor earthquake.
  - Structure should be able to resist moderate earthquake without significant structural damage, though some non structural damage are allowed.
  - Structure should be able to resist major earthquake without collapse while structural and non structural damage are allowed.
- Design Basic Earthquake (DBE), which can reasonably be expected to occur atleast once during design life of structure.

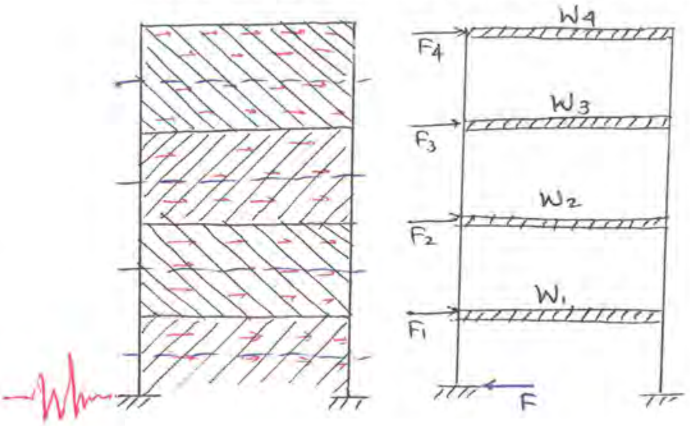
Structure	Life
Residential	50 yrs
Bridges	100 yrs
Dams	300 yrs
Taj Mahal	500 yrs

Magnitude of EQ	Return Period
6 →	30 yrs
7 →	50 yrs
8 →	100 yrs
8.5 →	300 yrs
9 →	500 yrs
9.5 →	1000 yrs

Structure	DBE
Residential →	7
Bridge →	8
Dam →	8.5
Taj Mahal →	9

- Maximum considered Earthquake (MCE) is the most severe earthquake effect considered by IS 1893: 2002
- The design approach adopted in this standard is to ensure that structures possess at least a minimum strength to withstand minor earthquakes (<DBE), which occur frequently, without damage; resist moderate earthquakes (DBE) without significant structural damage though some non-structural damage may occur and aims that structures withstand a major earthquake (MCE) without collapse.

DESIGN OF BUILDING FRAME FOR EARTHQUAKE



F (Base Shear) $= W A_h$

$W = W_1 + W_2 + W_3 + W_4$

A_h = Acceleration Coefficient

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

-While calculating W, partial LL is considered.

LL	considered.
upto $3 kN/m^2$	25%
beyond $3 kN/m^2$	50%

Z= zone factor, depends on earthquake zone.
I = importance factor, depends on functional use of structure

Table 2 Zone Factor, *Z*
(*Clause* 6.4.2)

Seismic Zone	II	III	IV	V
Seismic Intensity	Low	Moderate	Severe	Very Severe
Z	0.10	0.16	0.24	0.36

Z is corresponding to MCE and $\frac{Z}{2}$ represents DBE.

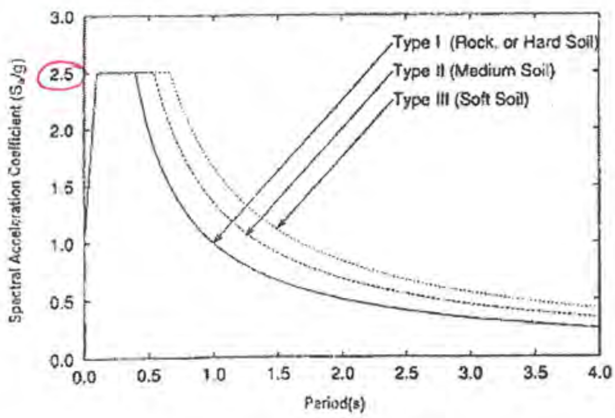
Table 6 Importance Factors, *I*
(*Clause* 6.4.2)

Sl No.	Structure	Importance Factor
(1)	(2)	(3)
i)	Important service and community buildings, such as hospitals; schools; monumental structures; emergency buildings like telephone exchange, television stations, radio stations, railway stations, fire station buildings; large community halls like cinemas, assembly halls and subway stations, power stations	1.5
ii)	All other buildings	1.0

NOTES

1 The design engineer may choose values of importance factor I greater than those mentioned above.

R= response reduction factor, depends on ductility of structure. Its value is higher for more ductile structure. (1 to 5)
Note: Value of I/R should not be more than 1.
Sa/g = Average response acceleration coefficient.



For rocky, or hard soil sites

$$\frac{S_a}{g} = \begin{cases} 1+15T; & 0.00 \le T \le 0.10 \\ 2.50 & 0.10 \le T \le 0.40 \\ 1.00/T & 0.40 \le T \le 4.00 \end{cases}$$

For medium soil sites

$$\frac{S_a}{g} = \begin{cases} 1+15T; & 0.00 \le T \le 0.10 \\ 2.50 & 0.10 \le T \le 0.55 \\ 1.36/T & 0.55 \le T \le 4.00 \end{cases}$$

For soft soil sites

$$\frac{S_a}{g} = \begin{cases} 1+15T; & 0.00 \le T \le 0.10 \\ 2.50 & 0.10 \le T \le 0.67 \\ 1.67/T & 0.67 \le T \le 4.00 \end{cases}$$

Time period of structure

7.6 Fundamental Natural Period

7.6.1 The approximate fundamental natural period of vibration (T_a), in seconds, of a moment-resisting frame building without brick infil panels may be estimated by the empirical expression:

$$T_a = 0.075\,h^{0.75} \quad \text{for RC frame building}$$
$$= 0.085\,h^{0.75} \quad \text{for steel frame building}$$

where

h = Height of building, in m. This excludes the basement storeys, where basement walls are connected with the ground floor deck or fitted between the building columns. But, it includes the basement storeys, when they are not so connected.

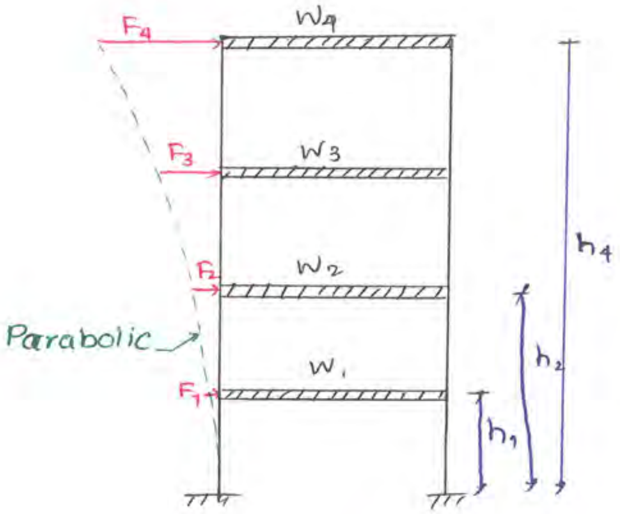
7.6.2 The approximate fundamental natural period of vibration (T_a), in seconds, of all other buildings, including moment-resisting frame buildings with brick infil panels, may be estimated by the empirical expression:

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

where

h = Height of building, in m, as defined in 7.6.1; and

d = Base dimension of the building at the plinth level, in m, along the considered direction of the lateral force.



$$F_i = F \times \frac{W_i h_i^2}{\sum W_i h_i^2}$$

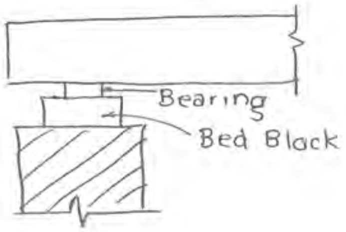
Above procedure to design of building frames is Equivalent Static Load method. This method is applicable for following structures only.

Regular Structure

- zone II & III → 90m height
- zone IV & V → 40m

Irregular Structure

- zone II & III → 40m
- zone IV & V → 12m



Masonry Design (IS1905 & SP20)

2.1 Bed Block — A block bedded on a wall, column or pier to disperse a concentrated load on a masonry element.

2.2 Bond — Arrangement of masonry units in successive courses to tie the masonry together both longitudinally and transversely; the arrangement is usually worked out to ensure that no vertical joint of one course is exactly over the one in the next course above or below it, and there is maximum possible amount of lap.

2.3 Column, Pier and Buttress

2.3.1 *Column* — An isolated vertical load bearing member, width of which does not exceed four times the thickness.

2.3.2 *Pier* — A thickened section forming integral part of a wall placed at intervals along the wall, to increase the stiffness of the wall or to carry a vertical concentrated load. Thickness of a pier is the overall thickness including the thickness of the wall or when bonded into a leaf of a cavity wall, the thickness obtained by treating that leaf as an independent wall (*see* Fig. 1).

2.3.3 *Buttress*—A pier of masonry built as an integral part of wall and projecting from either or both surfaces, decreasing in cross-sectional area from base to top.

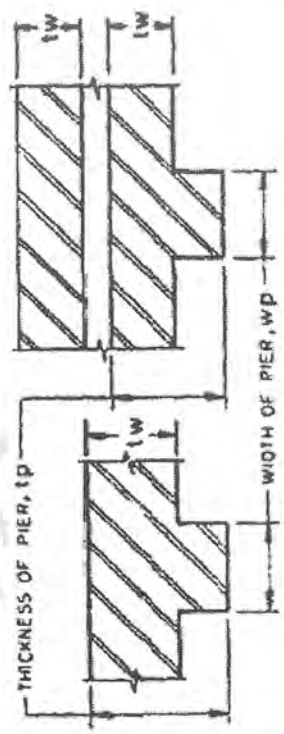


FIG. 1 DEFINITION OF PIER

2.6 Effective Height — The height of a wall or column to be considered for calculating slenderness ratio.

2.7 Effective Length — The length of a wall to be considered for calculating slenderness ratio.

2.8 Effective Thickness — The thickness of a wall or column to be considered for calculating slenderness ratio.

2.11 Joint — A junction of masonry units.

2.11.1 *Bed Joint* — A horizontal mortar joint upon which masonry units are laid.

2.11.2 *Cross Joint* — A vertical joint, normal to the face of the wall.

2.11.3 *Wall Joint* — A vertical joint parallel to the face of the wall.

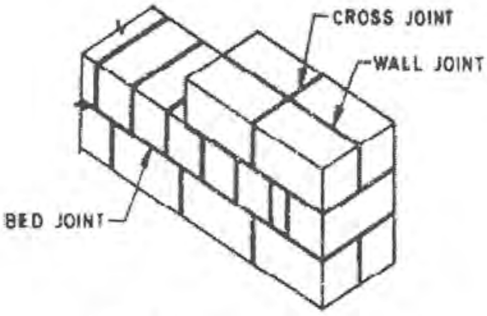


FIG. E-5 JOINTS IN MASONRY

2.17 Partition Wall — An interior non-load bearing wall, one storey or part storey in height.

2.18 Panel Wall — An exterior non-load bearing wall in framed construction, wholly supported at each storey but subjected to lateral loads.

2.19 Shear Wall — A wall designed to carry horizontal forces acting in its plane with or without vertical imposed loads.

2.20 Slenderness Ratio — Ratio of effective height or effective length to effective thickness of a masonry element.

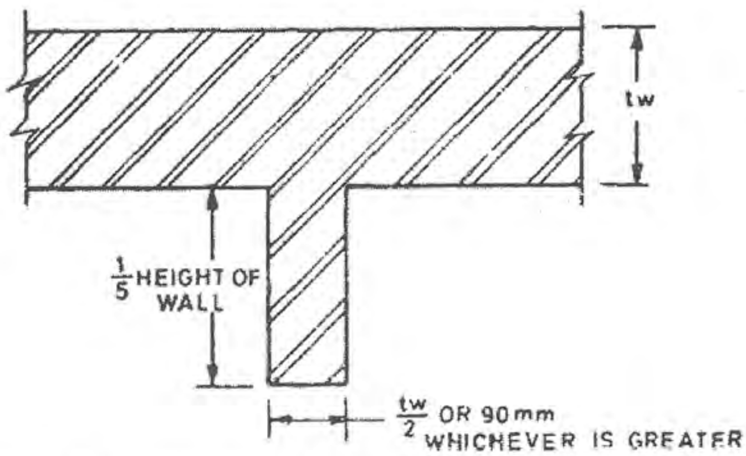
Slenderness ratio = Minimum $\begin{cases} \cdot H_{eff}/t_{eff} \\ \cdot L_{eff}/t_{eff} \end{cases}$

2.21 Types of Walls

2.21.1 *Cavity Wall* — A wall comprising two leaves, each leaf being built of masonry units and separated by a cavity and tied together with metal ties or bonding units to ensure that the two leaves act as one structural unit, the space between the leaves being either left as continuous cavity or filled with a non-load bearing insulating and waterproofing material. (Google → Pic)

2.21.2 *Faced Wall* — A wall in which facing and backing of two different materials are bonded together to ensure common action under load

2.21.3 *Veneered Wall* — A wall in which the facing is attached to the backing but not so bonded as to result in a common action under load.



7 MINIMUM DIMENSIONS FOR MASONRY WALL OR BUTTRESS PROVIDING EFFECTIVE LATERAL SUPPORT

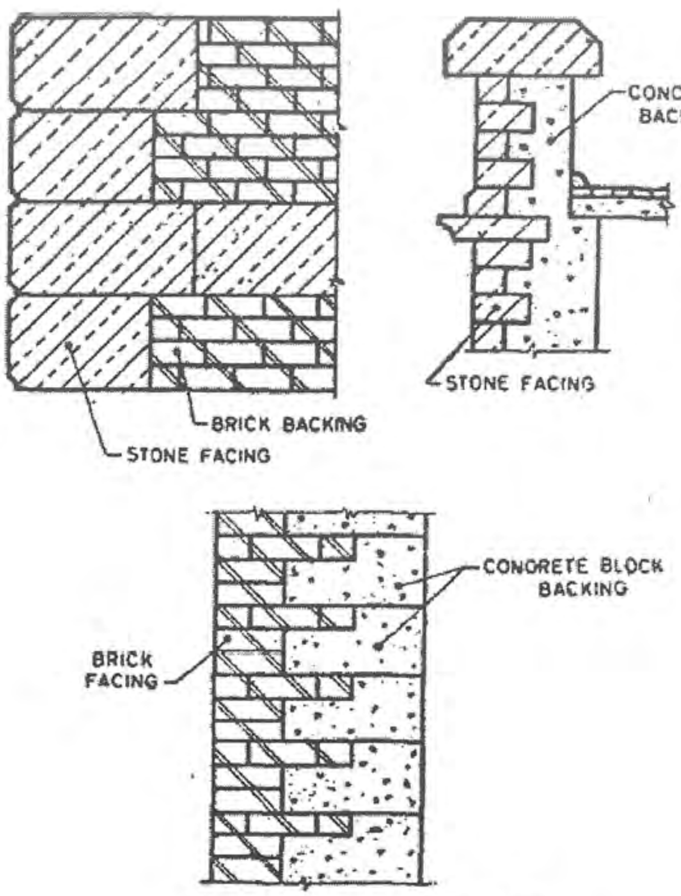


FIG. 3 TYPICAL FACED WALL

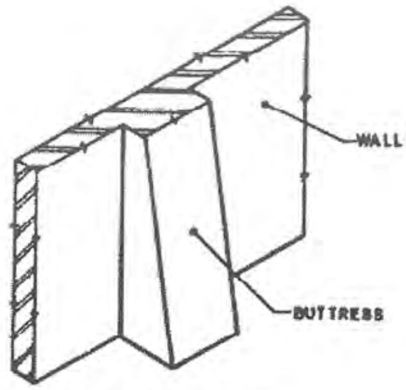
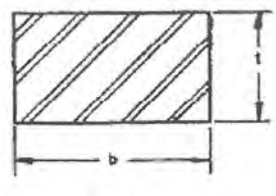
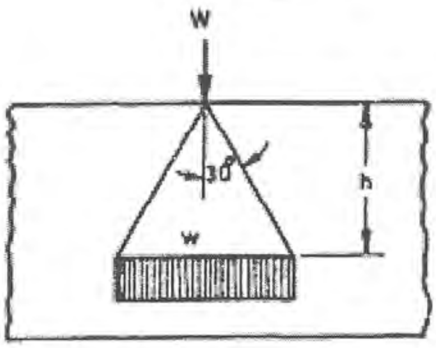


FIG. E-2 BUTTRESS

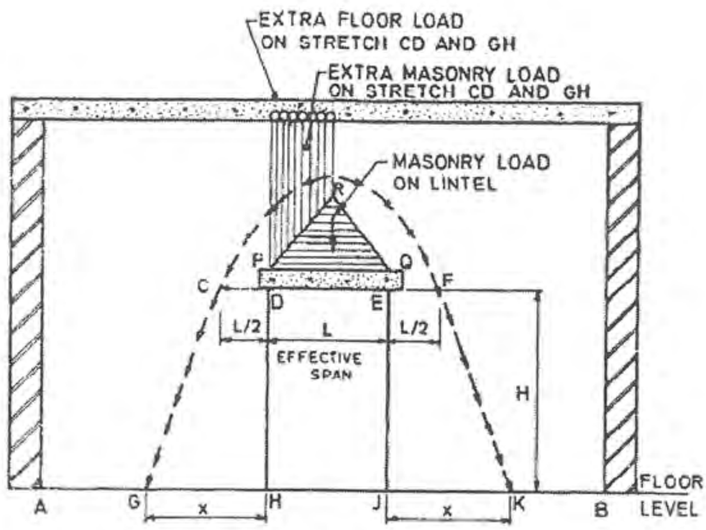


$b \leq 4t$

FIG. E-3 COLUMN



ANGLE OF DISPERSAL 30°



$x = L$ or $\frac{L+H}{2}$, whichever is less.

FIG. E-24 ARCHING ACTION IN MASONRY

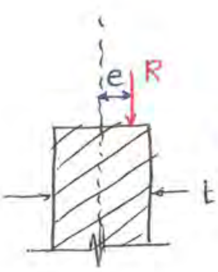
TABLE 1 MIX PROPORTION AND STRENGTH OF MORTARS FOR MASONRY

(*Clause* 3.2.1)

SL No.	GRADE OF MORTAR *	MIX PROPORTIONS (BY LOOSE VOLUME)					MINIMUM * COMPRESSIVE STRENGTH AT 28 DAYS IN N/mm²
		Cement	Lime	Lime Pozzolana Mixture	Pozzolana	Sand	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	H1	1	¼ C or B	0	0	3	10
2(a)	H2	1	½ C or B	0	0	4	7.5
2(b)		1	½ C or B	0	0	4½	6.0
3(a)	M1	1		0	0	5	5·0
3(b)		1	1 C or B	0	0	6	3.0
3(c)		0	0	1 (LP-40)	0	1½	3.0
4(a)	M2	1	0	0	0	6	3·0
4(b)		1	2 B	0	0	9	2·0
4(c)		0	1 A	0	0	2	2·0
4(d)		0	1 B	0	1	1	2·0
4(e)		0	1 C or B	0	2	0	2·0
4(f)		0	0	1 (LP-40)	0	1½	2·0
5(a)	M3	1	0	0	0	7	1.5
5(b)		1	3 B	0	0	12	1.5
5(c)		0	1 A	0	0	3	1.5
5(d)		0	1 B	0	2	1	1.5
5(e)		0	1 C or B	0	3	0	1.5
5(f)		0	0	1 (LP-40)	0	2	1.5
6(a)	L1	1	0	0	0	8	0·7
6(b)		0	1 B	0	1	2	0·7
6(c)		0	1 C or B	0	2	1	0·7
6(d)		0	0	1 (LP-40)	0	2¼	0·7
6(e)		0	0	1 (LP-20)	0	1½	0·7
7(a)	L2	0	1 B	0	0	3	0·5
7(b)		0	1 C or B	0	1	2	0·5
7(c)		0	0	1 (LP-7)	0	1½	0·5

H = High M = Medium L = Low

6.5.3.2 For load bearing walls, depth of vertical and horizontal chases shall not exceed one-third and one-sixth of the wall thickness respectively.



eccentricity ratio = $\frac{e}{t}$

5.4.3 *Permissible Shear Stress* — In case of walls built in mortar not leaner than Grade M1 (*see* Table 1) and resisting horizontal forces in the plane of the wall, permissible shear stress, calculated on the area of bed joint, shall not exceed the value obtained by the formula given below, subject to a maximum of 0·5 N/mm²:

$$f_s = 0{\cdot}1 + f_d/6$$

where

f_s = permissible shear stress in N/mm², and

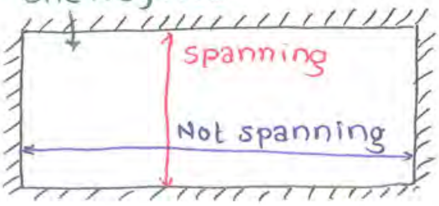
f_d = compressive stress due to dead loads in N/mm².

Eccentricity Ratio:

It is the ratio of eccentricity of resultant load on wall and thickness of wall.

SI. No.	CONDITION OF SUPPORT	EFFECTIVE HEIGHT	
(1)	(2)	(3)	
1.	Lateral as well as rotational restraint (that is, full restraint) at top and bottom. For example, when the floor/roof spans on the walls so that reaction to load of floor/roof is provided by the walls, or when an RCC floor/roof has bearing on the wall (minimum 9 cm), irrespective of the direction of the span (foundation footings of a wall give lateral as well as rotational restraint)	0·75 H	
2.	Lateral as well as rotational restraint (that is, full restraint) at one end and only lateral restraint (that is, partial restraint) at the other. For example, RCC floor/roof at one end spanning or adequately bearing on the wall and timber floor/roof not spanning on wall, but adequately anchored to it, on the other end	0·85 H	
3.	Lateral restraint, without rotational restraint (that is, partial restraint) on both ends. For example, timber floor/roof, not spanning on the wall but adequately anchored to it on both ends of the wall, that is, top and bottom	1·00 H	
4.	Lateral restraint as well as rotational restraint (that is, full restraint) at bottom but have no restraint at the top. For example, parapet walls with RCC roof having adequate bearing on the lower wall, or a compound wall with proper foundation on the soil	1·50 H	

one ways lab



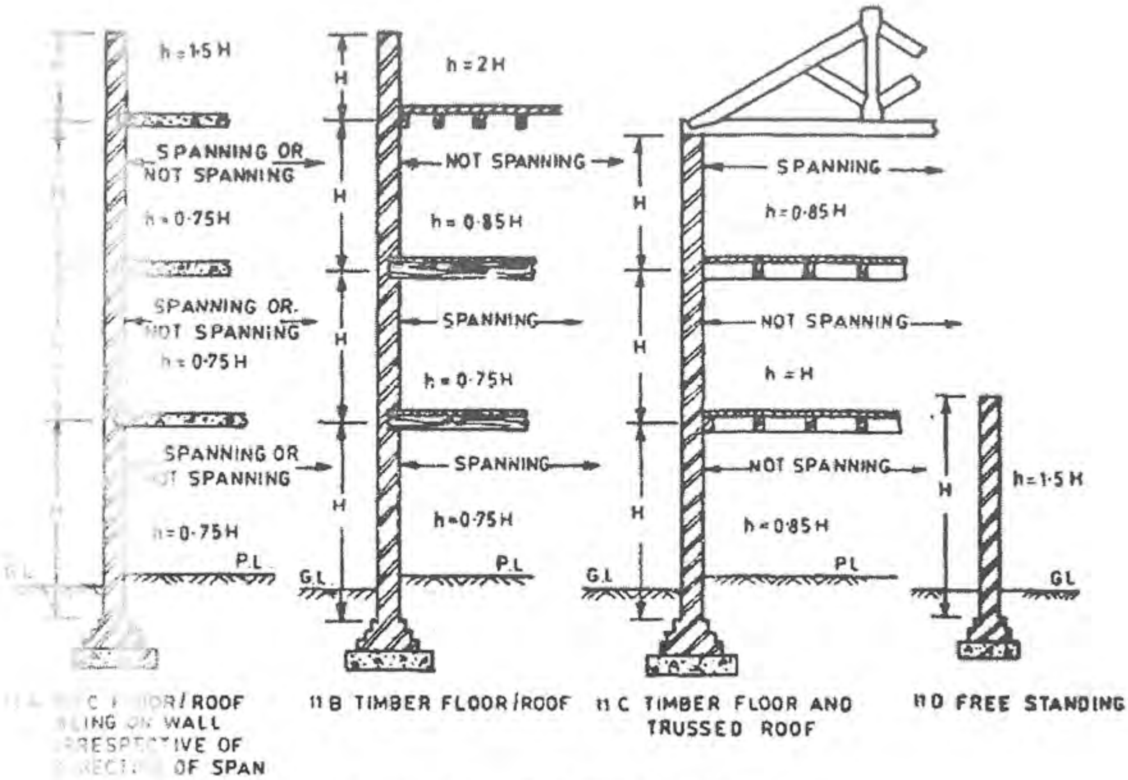


Fig. 11 Effective Height of Wall

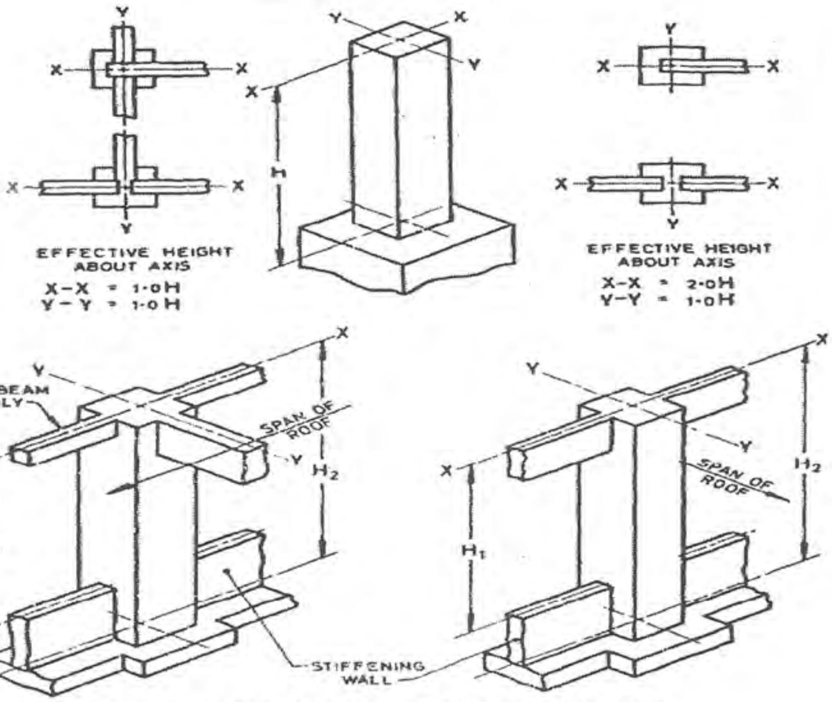


Fig. 12 Examples of Effective Height of Columns

4.4 Effective Length — Effective length of a wall shall be as given in Table 5.

TABLE 5 EFFECTIVE LENGTH OF WALLS

SL No.	CONDITIONS OF SUPPORT (see FIG. 13)	EFFECTIVE LENGTH
(1)	(2)	(3)
1.	Where a wall is continuous and is supported by cross wall, and there is no opening within a distance of $H/8$ from the face of cross wall or Where a wall is continuous and is supported by piers/buttresses conforming to **4.2.1.2** (b)	$0.8\ L$
2.	Where a wall is supported by a cross wall at one end and continuous with cross wall at other end or Where a wall is supported by a pier/buttress at one end and continuous with pier/buttress at other end conforming to **4.2.1.2** (b)	$0.9\ L$
3.	Where a wall is supported at each end by cross wall or Where a wall is supported at each end by a pier/buttress conforming to **4.2.1.2** (b)	$1.0\ L$
4.	Where a wall is free at one end and continuous with a cross wall at the other end or Where a wall is free at one end and continuous with a pier/buttress at the other end conforming to **4.2.1.2** (b)	$1.5\ L$
5.	Where a wall is free at one end and supported at the other end by a cross wall or Where a wall is free at one end and supported at the other end by a pier/buttress conforming to **4.2.1.2** (b)	$2.0\ L$

where

H = actual height of wall between centres of adequate lateral support; and

L = length of wall from or between centres of cross wall, piers or buttresses.

NOTE — In case there is an opening taller than $0.5\ H$ in a wall, ends of the wall at the opening shall be considered as free.

TABLE 7 MAXIMUM SLENDERNESS RATIO FOR A LOAD BEARING WALL

NO. OF STOREYS	MAXIMUM SLENDERNESS RATIO	
	Using Portland Cement or Portland Pozzolana Cement in Mortar	Using Lime Mortar
(1)	(2)	(3)
Not exceeding 2	27	20
Exceeding 2	27	13

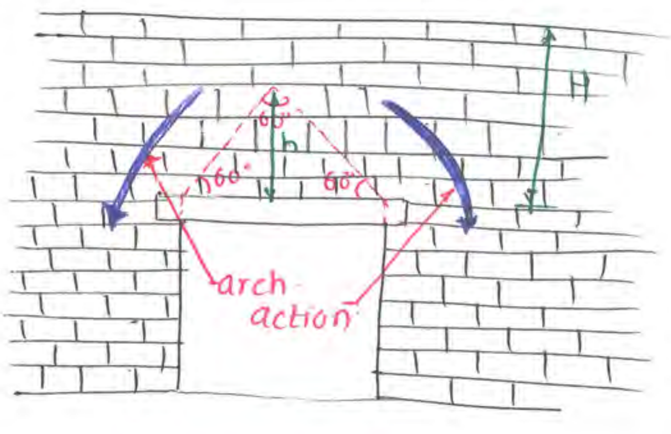
For columns:

- Laterally restrained at ends

  $H_{eff} = 1.0H$

- Laterally not restrained at ends

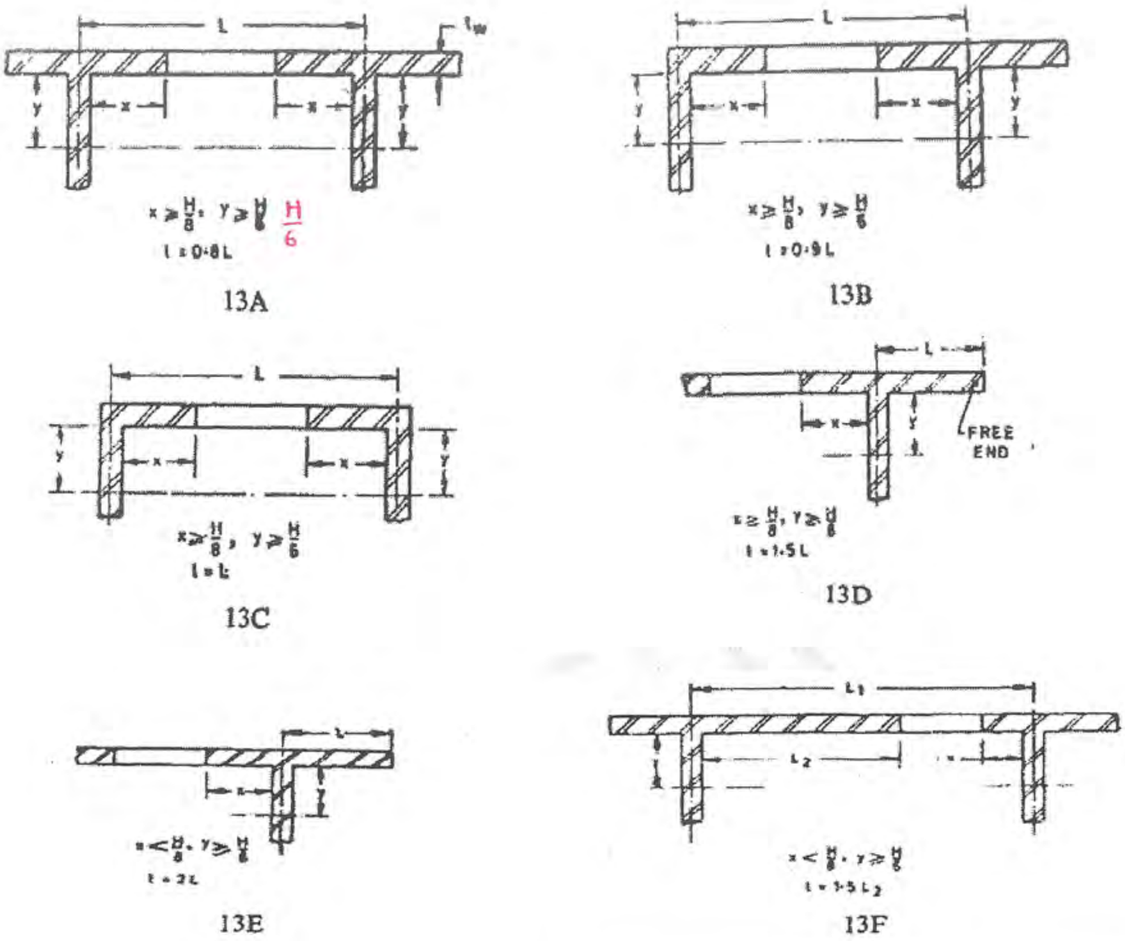
  $H_{eff} = 2.0H$



For arch action

$H > 1.25h$

MD-08



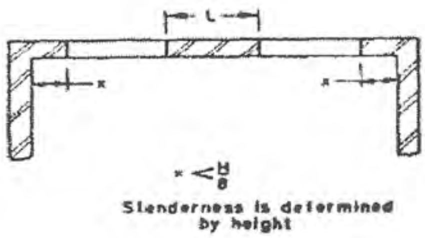


FIG. 13 EFFECTIVE LENGTH OF WALL

TABLE 6 STIFFENING COEFFICIENT FOR WALL STIFFENED BY PIERS, BUTTRESSES OR CROSS WALLS

(*Clauses* 4.5.2 *and* 4.5.3)

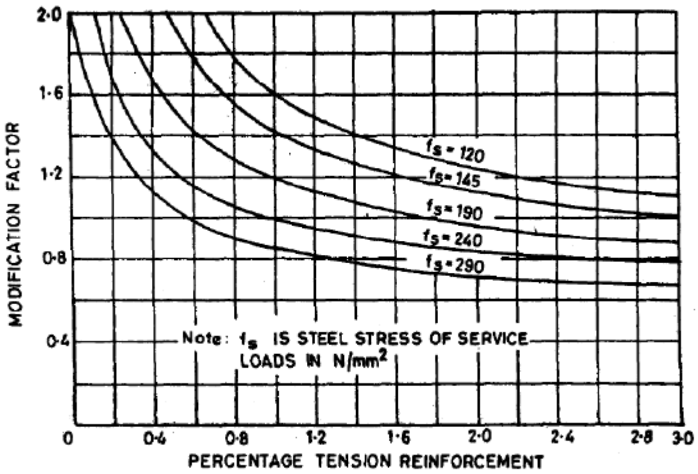
SL No.	$\frac{S_P}{W_P}$ Ratio	Stiffening Coefficient $\frac{t_P}{t_w} = 1$	$\frac{t_P}{t_w} = 2$	$\frac{t_P}{t_w} = 3$ or more
(1)	(2)	(3)	(4)	(5)
1	6	1·0	1·4	2·0
2	8	1·0	1·3	1·7

TABLE A SALIENT POINTS ON THE DESIGN STRESS-STRAIN CURVE FOR COLD-WORKED BARS

(Clause 1.4)

Stress Level	f_y = 415 N/mm²		f_y = 500 N/mm²	
	Strain	Stress	Strain	Stress
(1)	(2)	(3) N/mm²	(4)	(5) N/mm²
0·80 f_{yd}	0·001 44	288·7	0·001 74	347·8
0·85 f_{yd}	0·001 63	306·7	0·001 95	369·6
0·90 f_{yd}	0·001 92	324·8	0·002 26	391·3
0·95 f_{yd}	0·002 41	342·8	0·002 77	413·0
0·975 f_{yd}	0·002 76	351·8	0·003 12	423·9
1·0 f_{yd}	0·003 80	360·9	0·004 17	434·8

NOTE — Linear interpolation may be done for intermediate values.



$$f_s = 0.58\, f_y \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

FIG. 4 MODIFICATION FACTOR FOR TENSION REINFORCEMENT

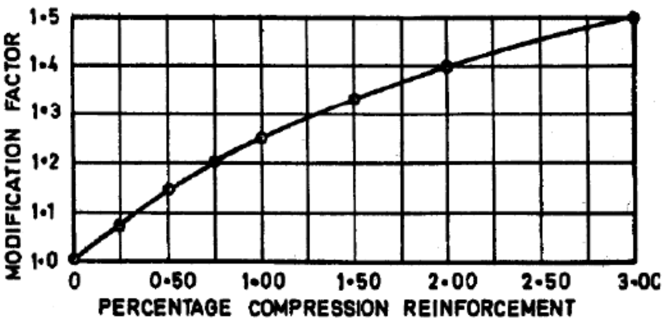


FIG. 5 MODIFICATION FACTOR FOR COMPRESSION REINFORCEMENT

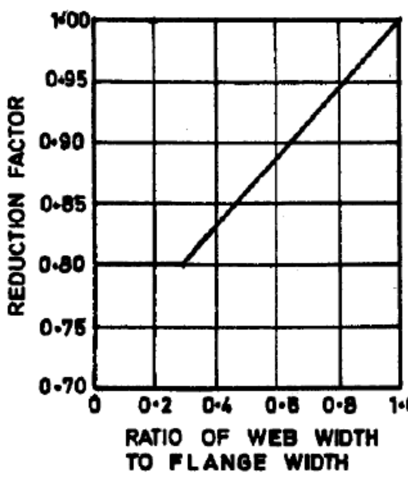


Table 19 Design Shear Strength of Concrete, τ_c, N/mm²

(*Clauses* 40.2.1, 40.2.2, 40.3, 40.4, 40.5.3, 41.3.2, 41.3.3 *and* 41.4.3)

$100\frac{A_s}{bd}$	Concrete Grade					
	M 15	M 20	M 25	M 30	M 35	M 40 and above
(1)	(2)	(3)	(4)	(5)	(6)	(7)
≤ 0.15	0.28	0.28	0.29	0.29	0.29	0.30
0.25	0.35	0.36	0.36	0.37	0.37	0.38
0.50	0.46	0.48	0.49	0.50	0.50	0.51
0.75	0.54	0.56	0.57	0.59	0.59	0.60
1.00	0.60	0.62	0.64	0.66	0.67	0.68
1.25	0.64	0.67	0.70	0.71	0.73	0.74
1.50	0.68	0.72	0.74	0.76	0.78	0.79
1.75	0.71	0.75	0.78	0.80	0.82	0.84
2.00	0.71	0.79	0.82	0.84	0.86	0.88
2.25	0.71	0.81	0.85	0.88	0.90	0.92
2.50	0.71	0.82	0.88	0.91	0.93	0.95
2.75	0.71	0.82	0.90	0.94	0.96	0.98
3.00 and above	0.71	0.82	0.92	0.96	0.99	1.01

NOTE — The term A_s is the area of longitudinal tension reinforcement which continues at least one effective depth beyond the section being considered except at support where the full area of tension reinforcement may be used provided the detailing conforms to 26.2.2 and 26.2.3

Table 20 Maximum Shear Stress, $\tau_{c\,max}$, N/mm²

(*Clauses* 40.2.3, 40.2.3.1, 40.5.1 *and* 41.3.1)

Concrete Grade	M 15	M 20	M 25	M 30	M 35	M 40 and above
$\tau_{c\,max}$, N/mm²	2.5	2.8	3.1	3.5	3.7	4.0

Table 26 Bending Moment Coefficients for Rectangular Panels Supported on Four Sides with Provision for Torsion at Corners

(*Clauses* D-1.1 *and* 24.4.1)

Case No.	Type of Panel and Moments Considered	Short Span Coefficients α_x (Values of l_y/l_x)								Long Span Coefficients α_y for All Values of l_y/l_x
		1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1	*Interior Panels*:									
	Negative moment at continuous edge	0.032	0.037	0.043	0.047	0.051	0.053	0.060	0.065	0.032
	Positive moment at mid-span	0.024	0.028	0.032	0.036	0.039	0.041	0.045	0.049	0.024
2	*One Short Edge Continuous*:									
	Negative moment at continuous edge	0.037	0.043	0.048	0.051	0.055	0.057	0.064	0.068	0.037
	Positive moment at mid-span	0.028	0.032	0.036	0.039	0.041	0.044	0.048	0.052	0.028
3	*One Long Edge Discontinuous*:									
	Negative moment at continuous edge	0.037	0.044	0.052	0.057	0.063	0.067	0.077	0.085	0.037
	Positive moment at mid-span	0.028	0.033	0.039	0.044	0.047	0.051	0.059	0.065	0.028
4	*Two Adjacent Edges Discontinuous*:									
	Negative moment at continuous edge	0.047	0.053	0.060	0.065	0.071	0.075	0.084	0.091	0.047
	Positive moment at mid-span	0.035	0.040	0.045	0.049	0.053	0.056	0.063	0.069	0.035
5	*Two Short Edges Discontinuous*:									
	Negative moment at continuous edge	0.045	0.049	0.052	0.056	0.059	0.060	0.065	0.069	—
	Positive moment at mid-span	0.035	0.037	0.040	0.043	0.044	0.045	0.049	0.052	0.035
6	*Two Long Edges Discontinuous*:									
	Negative moment at continuous edge	—	—	—	—	—	—	—	—	0.045
	Positive moment at mid-span	0.035	0.043	0.051	0.057	0.063	0.068	0.080	0.088	0.035
7	*Three Edges Discontinuous (One Long Edge Continuous)*:									
	Negative moment at continuous edge	0.057	0.064	0.071	0.076	0.080	0.084	0.091	0.097	—
	Positive moment at mid-span	0.043	0.048	0.053	0.057	0.060	0.064	0.069	0.073	0.043
8	*Three Edges Discontinuous (One Short Edge Continuous)*:									
	Negative moment at continuous edge	—	—	—	—	—	—	—	—	0.057
	Positive moment at mid-span	0.043	0.051	0.059	0.065	0.071	0.076	0.087	0.096	0.043
9	*Four Edges Discontinuous*:									
	Positive moment at mid-span	0.056	0.064	0.072	0.079	0.085	0.089	0.100	0.107	0.056

Table 27 Bending Moment Coefficients for Slabs Spanning in Two Directions at Right Angles, Simply Supported on Four Sides

(*Clause* D-2.1)

l_y/l_x	1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	2.5	3.0
α_x	0.062	0.074	0.084	0.093	0.099	0.104	0.113	0.118	0.122	0.124
α_y	0.062	0.061	0.059	0.055	0.051	0.046	0.037	0.029	0.020	0.014

Chart 44 COMPRESSION WITH BENDING—Rectangular Section—Reinforcement Distributed Equally on Four Sides