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SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

UNIT – I – STAIRCASES – SCI1308

UNIT – I – STAIRCASES

Stair case, Components of stair case, Types of stair case - Limit state design of dog legged and open newel staircase.

Introduction

Staircase is an important component of a building providing access to different floors and roof of the building. It consists of a flight of steps (stairs) and one or more intermediate landing slabs between the floor levels. Different types of staircases can be made by arranging stairs and landing slabs. Staircase, thus, is a structure enclosing a stair.

Technical Terms

The definitions of some technical terms, which are used in connection with design of stairs, are given.

a) Tread or Going: horizontal upper portion of a step.

b) Riser: vertical portion of a step.

c) Rise: vertical distance between two consecutive treads.

d) Flight: a series of steps provided between two landings.

e) Landing: a horizontal slab provided between two flights.

f) Waist: the least thickness of a stair slab.

g) Winder: radiating or angular tapering steps.

h) Soffit: the bottom surface of a stair slab.

i) Nosing: the intersection of the tread and the riser.

j) Headroom: the vertical distance from a line connecting the nosing of all treads and the soffit above.

Types of Staircase

The various types of staircase adopted in different types of buildings can be grouped under geometrical and structural classifications depending upon their shape and plan pattern and their structural behavior under loads.

Geometrical Classification

a) Straight stairs

- All steps lead in one direction.
- This may be continuous with two flights with an intermediate landing
- Adopted when staircase is narrow.

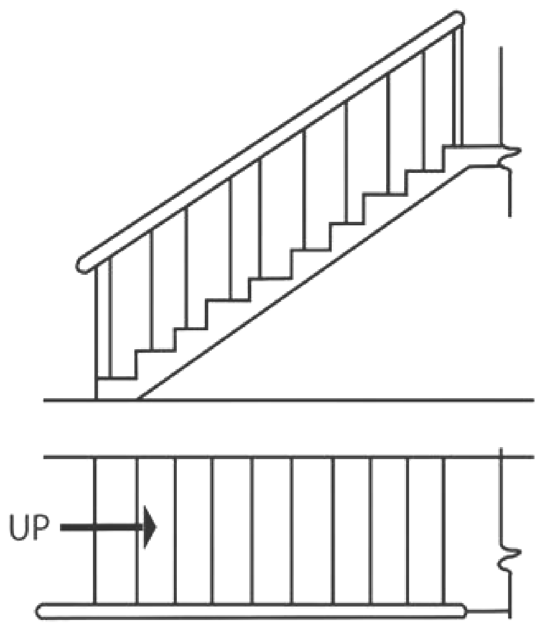


Fig.:1 Straight staircase

b) Dog-legged staircase

- Consist of two straight flights running in opposite direction

- There is no space between flights in plan
- Landing is provided at level which direction of flight changes.

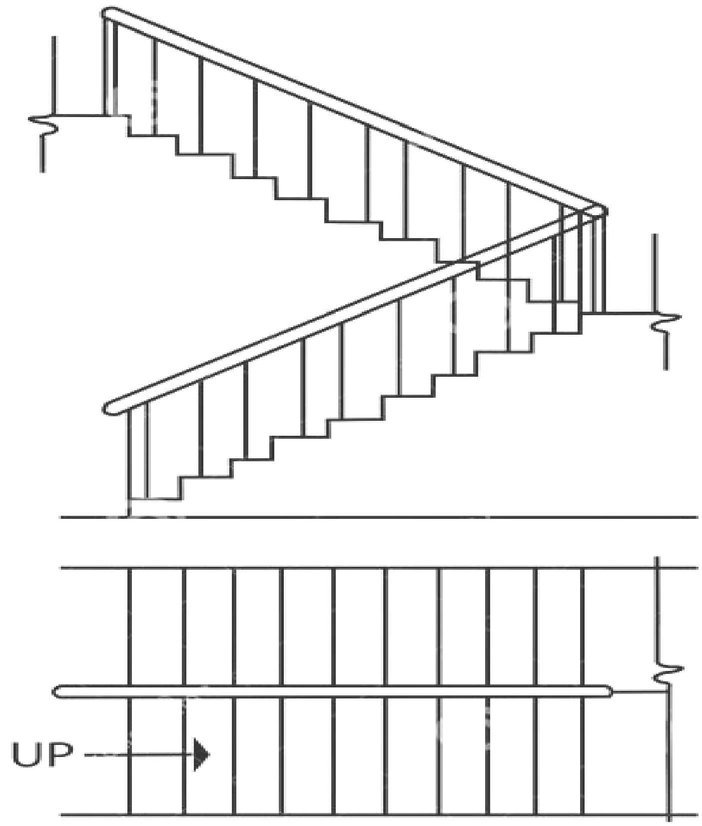


Fig.:2 Dog Legged staircase

c) Quarter turn Newel:

- A stair turning through 90° with the help of level landing
- Used in shops and public buildings

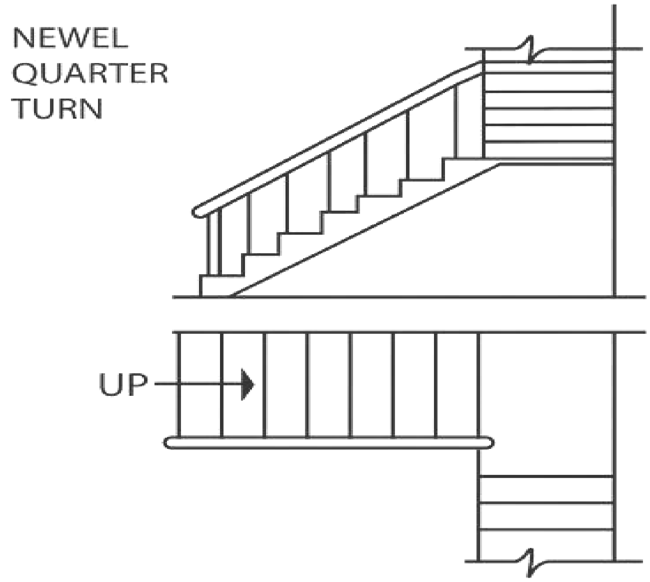


Fig.:3 Quarter turn Newel

d) Open Newel Stairs:

- Popularly known as open well stairs
- A well or opening is left between forward and backward flight

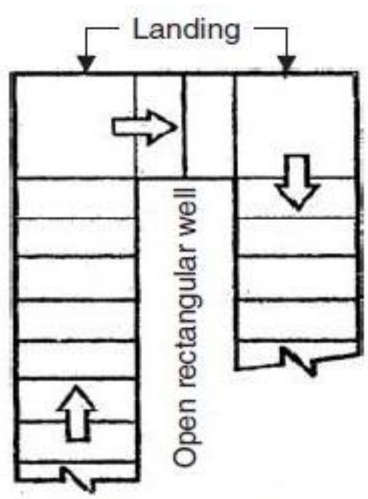


Fig.:4 Open Well or Newel

e) Circular stairs:

- All the steps radiate from new post or well hole

➢ Mostly located at rear of building

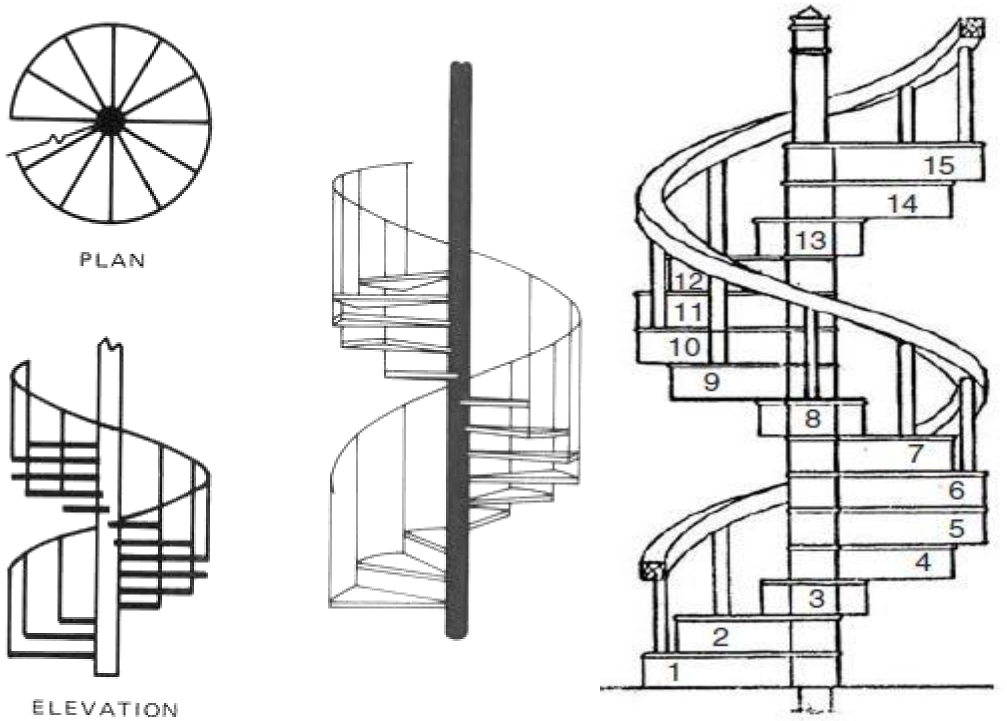


Fig.:5 Circular stairs

Structural behavior of staircases

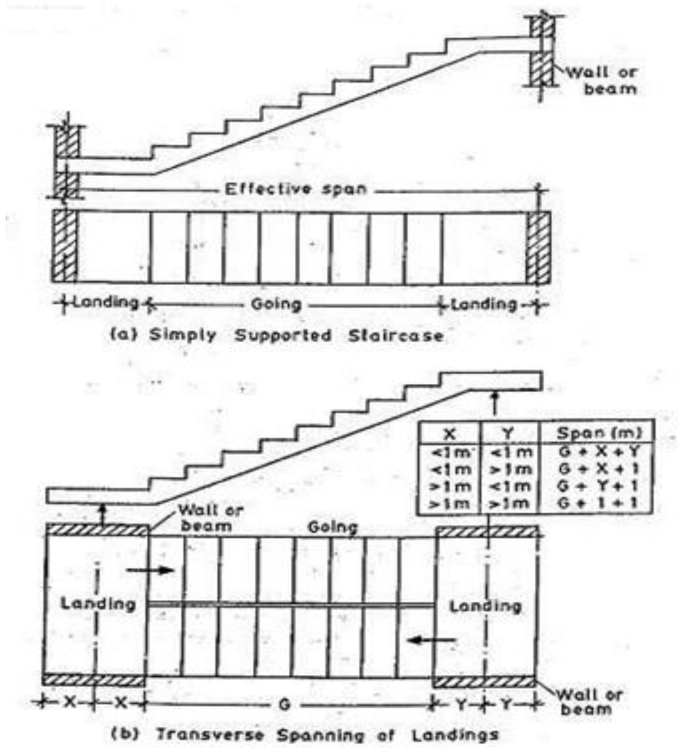
a) Stairs Spanning in Longitudinal direction:

➢ Inclined stair flight together with landing are supported on wall and beams

➢ The effective span is considered between the centre to centre of supports as in figure a

➢ As in figure b, the transverse spanning of landings span is taken

➢ In case of open well stairs where span partly cross at right angles the load on common area may distributed as one half in each direction in figure c.



(a) Simply Supported Staircase

(b) Transverse Spanning of Landings

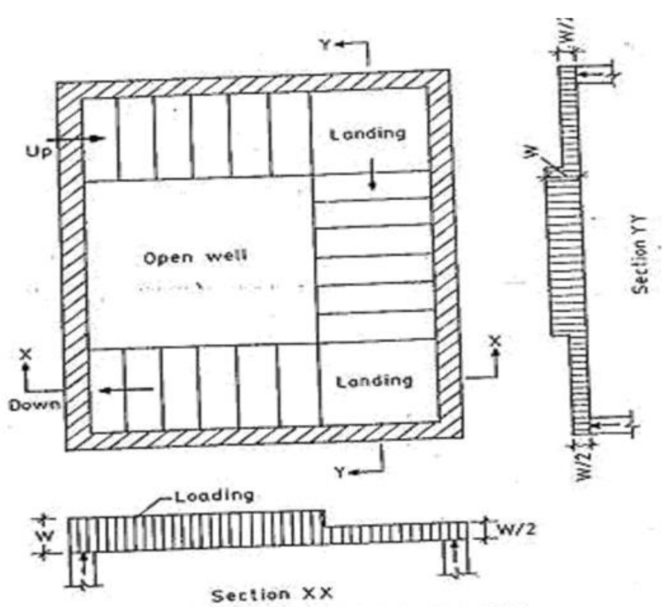
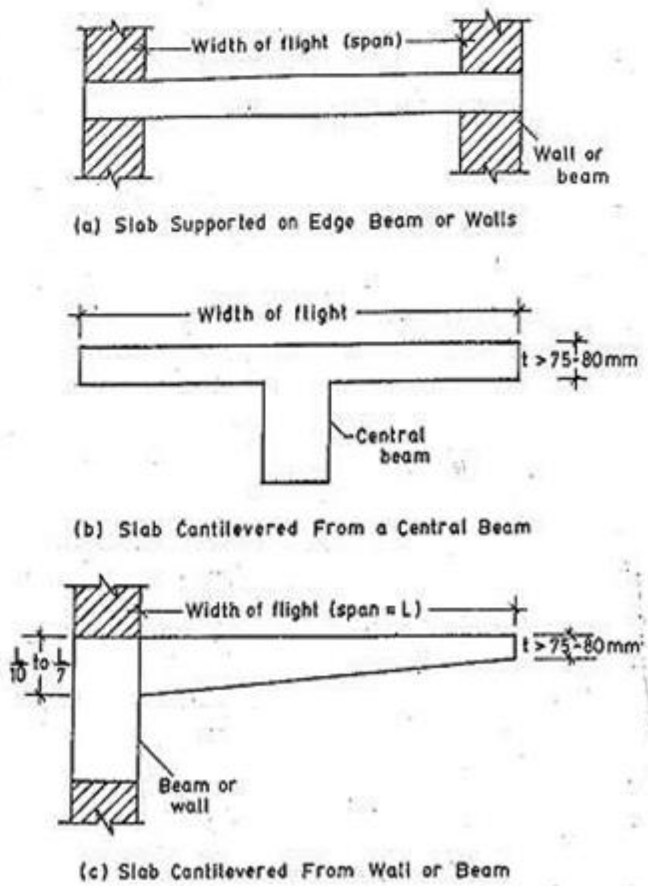


Fig. C Loading on Stairs With Open Wells

b) Stair slab spanning in the transverse direction:

- Following are the examples of slabs spanning in the transverse direction
- In these slabs width of flight being small (1-1.5m)

- Minimum thickness of 75 to 80mm should be provided.
- Minimum percentage reinforcement to resist maximum bending moment should be provided.



Staircase Slabs Spanning In the Transverse Direction

General Guidelines

The following are some of the general guidelines to be considered while planning a staircase:

Rise (R) : 150mm to 180mm

Tread (T) : 220 mm to 250 mm- for residential buildings.

Rise (R) : 120 to 150 mm

Tread (T) : 250 mm to 300 mm – for public buildings

[T + 2R] : Between 500 mm to 650 mm

The width of the stair

- 0.8 m to 1 m for residential building and
- 1.8 m to 2 m for public building.

DESIGN GUIDELINES

a. Geometrical Design

- ✓ Assume Suitable Tread and Riser
- ✓ No. of Riser = (F/F Height) ⁄ Rise
- ✓ No. of Risers in One Flight =0.5*(No. Of Risers)
- ✓ No. of Tread = (No. of Risers —1)
- ✓ Going distance = (No. of tread) × (tread width)

- ✓ Width of landing ≥ width of stair

b. Structural Design

- ✓ **Effective span calculation**

Effective span calculation = c/c distance between supports

If not given width of support can be taken in between 200 to 300 mm

- ✓ **Trial depth of waist slab**

According to is 456:2000 article 23.2.1 by calculating ratio of span to effective depth and after that ratio is multiplying by the modification factor. Modification factor can be calculated by assuming % of tension reinforcement

- ✓ **Load Calculation**

Calculations should be made by considering width of slab equal to 1 meter

Self weight of slab = $25 \times D \times \sqrt{(R^2+T^2)}/T$ (KN.m)

Wt of steps = $25 \times 0.5 \times R$ (KN.m)

Wt of floor finish = 1×1 (KN.m) (ASSUME)

Live load = 3 KN/m² (RESIDENTIAL BUILDING)

Live load = 4-5 KN/m² (PUBLIC BUILDING)

Net load (W) = $W1 + W2 + W3 + W4$

Factored load = $W' = 1.5 * W$

- ✓ **Calculation of design moments**

Find max bending either by considering it equals to $(0.125w' \times L^2)$

- ✓ **Check for effective depth**

- ✓ **Check for reinforcement**
    - ➢ *Calculate main steel (A_{st})*
        - ❖ Ast ≥ Ast minimum
    - ➢ *Provide suitable distribution steel = $A_{st\ (min)}$*
- ✓ **Check for shear**
    - ➢ Calculate max design shear force
        - ❖ Vud = 0.5 × W'L
    - ➢ Calculate shear resisted by concrete
        - ❖ Vuc = k × τc × b×d

<u>Vud < Vuc</u>

Values of k can be obtained by the following tables IS 456:200

Overall depth of slab (mm)	300 or more	275	250	225	200	175	150 or Less
k	1.00	1.05	1.10	1.15	1.20	1.25	1.30

- ✓ **Check for deflection**
    - ➢ Calculate actual % of reinforcement

<u>L/d provided < L/d max</u>

Example Problem

Design a dog legged stair case for a residential building hall measuring 2.2 m x 4.7 m. The width of the landing is 1m. The distance between floor to floor is 3.3 m. The rise and tread may be taken as 150mm and 270mm respectively. The weight of floor finish is 1 kN/m^2 . The materials used are M20 grade concrete and Fe415 grade steel. Sketch the details of steel. Here flight and the landing slabs spans in the same direction i.e, Flight spans longitudinally.

Data: f_{ck} = 20 MPa, fy = 415MPa, Landing = 1m, H = 3.3m, Size of stair case hall = 2.2m x 3.7 m. Assume the wall thickness as 200 mm.

Proportioning of stair:

R = 150 mm, T= 270 mm.

H = 3.3m. Height of each flight = H1= H/2 =3.3 /2 = 1.65m

Number of risers in each flight = H1/R = 1650 /150 = 11.

Thus Number of steps = 11-1 =10

Width of flight horizontally = 10 x 270 =

2700mm Width of hall = 4700mm.

Thus the each landing width = 1m

Width of step = 1m. Gap between the flights = 0.2 m

To fix the depth of waist slab:

l/d = 26, assuming partial fixity le = 4700 + 200 = 4900 mm.

d= 4900/26= 188mm. Assume effective cover = 20 mm (mild exposure) + 10/2 = 25 mm, D= d + de = 213 mm say 215mm. Thus, d= 190mm

Calculation of loads:

$\tan\theta = R/T = 0.56$. Hypotenuse, h = 309mm. $\cos\theta = T/h = 270/309 = 0.874$

Deal load of waist slab in plan = $(0.215 x1 x1 x25)\ 1/\cos\theta = 6.14\ kN/m^2$

Weight of all floor finish horizontally, assumed as = $1kN/m^2$

Weight of steps = R/2 x1 x1 x 24 = $1.80\ kN/m^2$

Imposed load = 3 kN/m2 Total load = w = $11.94\ kN/m^2$

Ultimate load= 1.5x11.94 = wu = $17.91\ kN/m^2$

Design of second flight:

Factored bending moment for second flight taking partial fixity

effect BM = $W_u l^2 /10 = 17.91 \times 4.9^2 /10 = 43.0$ KN.m

Mu, lim of the given waist slab as balanced section = Qbd^2 = 2.76 x 1000 x 1902

=99.63 kNm> 43kNm. Under reinforced section.

The depth provided is ok.

To find area of steel:

Main steel By calculation For Under Reinforced Section,

Area of steel: Mu = 0.87 fy Ast d(1 – fyAst/fck bd)

Solving Ast = 676 mm^2, p = 100Ast/bd = 0.356% < pt, lim = 0.96%,

Provide 10 mm @ s= 78.6/676 x 1000 =116.2, say 110mm c/c.

Provide #10@110. Here spacing is < k_s τ_c < τ_c, max. ok

Hence the slab is safe in shear.

Check for development length:

Provide enough development length at the junction of flight and landing and also necessary anchorage over the support.

L_d= 47φ =470 mm and l_d/3 =160mm.

Check for deflection:

As the effective depth provided is more than that required for controlling the deflection, the slab is safe in shear.

(Le/d) available < (L/d)basic * k_1 * k_2 * k_3.

4700/190 = 24.7 < 26x1x1.3x1.

ok.

Here k_2 is 1.3 from IS: 456-2000, Fig. 4, page 38, for p=0.35% and f_s = 240MPa of Fe415 steel.

Check for cracking:

As the detailing requirements with regard to diameter, spacing for main and dist. steel and cover for slab are satisfied as per the requirements of IS:456-2000, the cracking is prevented indirectly. Thus slab is safe in serviceability requirements.

SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

UNIT – II – YIELD LINE THEORY – SCI1308

YIELD LINE THEORY

Concept of yield line theory - Determination of collapse load and design of rectangular, circular and triangular slabs (formulas only)

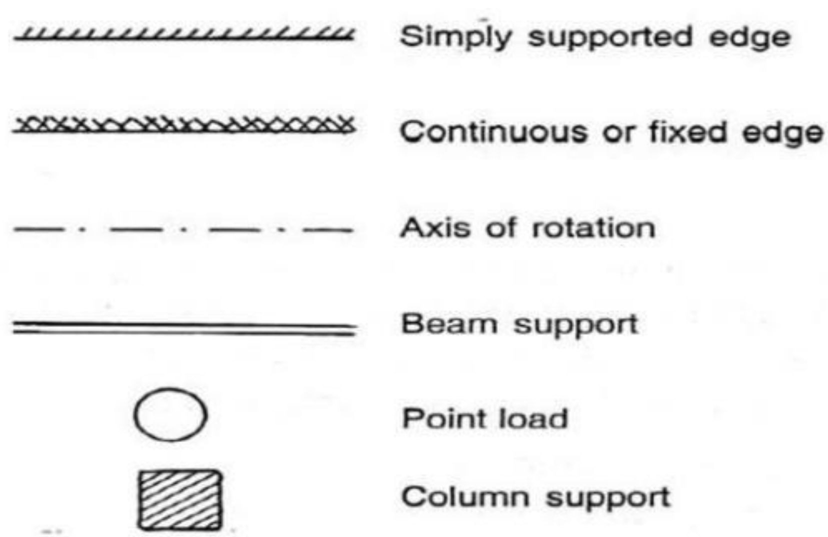
Yield lines

A Typical crack patterns is generated when ultimate moment is reached and that line is represented as Yield line.

The characteristics of yield lines are,

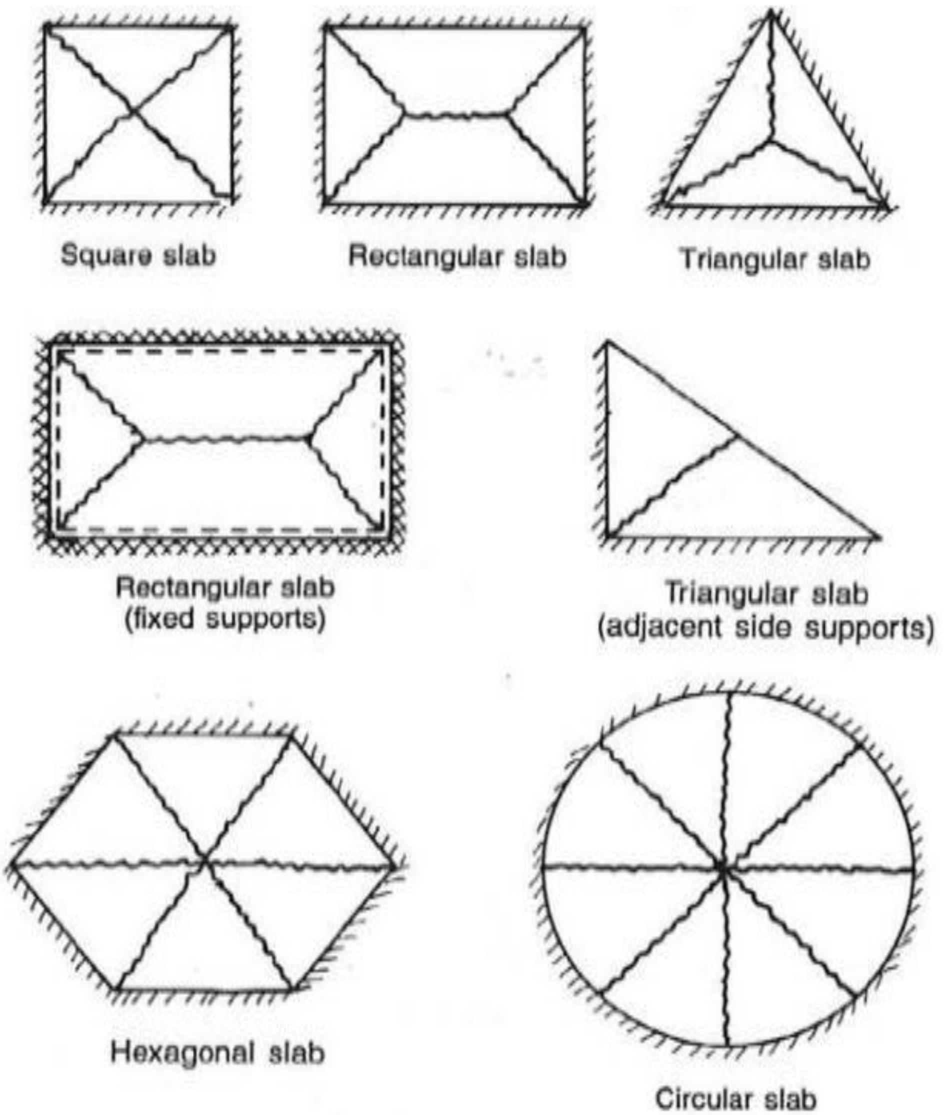
a) Yield lines are straight
b) Yield lines end at supporting edges of slab
c) Yield lines passes through intersection of axis of rotation of adjacent slab elements
d) Axis of rotation lies along lines of supports and passes over columns

Notations for Yield lines and Supports



. Notations for yield lines and supports.

Typical yield line pattern for different slabs



Typical yield line patterns in reinforced concrete slabs.

Assumptions in YLT

The following are the assumptions of the yield line analysis of reinforced concrete slabs.

1. The steel reinforcement is fully yielded along the yield lines at collapse. Rotation following yield is at constant moment.
2. The slab deforms plastically at collapse and is separated into segments by the yield lines. The individual segments of the slab behave elastically.
3. The elastic deformations are neglected and plastic deformations are only considered. The entire deformations, therefore, take place only along the yield lines. The individual segments of the slab remain plane even in the collapse condition.
4. The bending and twisting moments are uniformly distributed along the yield lines. The maximum values of the moments depend on the capacities of the section based on the amount of reinforcement provided in the section.
5. The yield lines are straight lines as they are the lines of intersection between two planes.

Rules of yield lines

The two terms, positive and negative yield lines, are used in the analysis to designate the yield lines for positive bending moments having tension at the bottom and negative bending moments having tension at the top of the slab, respectively.

The following are the guidelines for predicting the yield lines and axes of rotation

1. Yield lines between two intersecting planes are straight lines.
2. Positive yield line will be at the mid-span of one-way simply supported slabs.

3. Negative yield lines will occur at the supports in addition to the positive yield lines at the mid-span of one-way continuous slabs.

4. Yield lines will occur under point loads and they will be radiating outward from the point of application of the point loads.

5. Yield line between two slab segments should pass through the point of intersection of the axes of rotation of the adjacent slab segments.

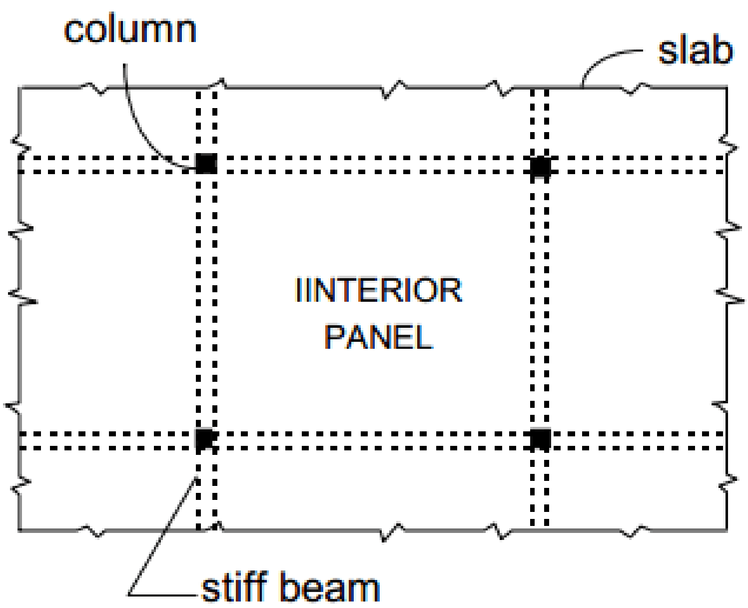
6. Yield lines should end at the boundary of the slab or at another yield line.

e) Yield lines represent the axes of rotation.

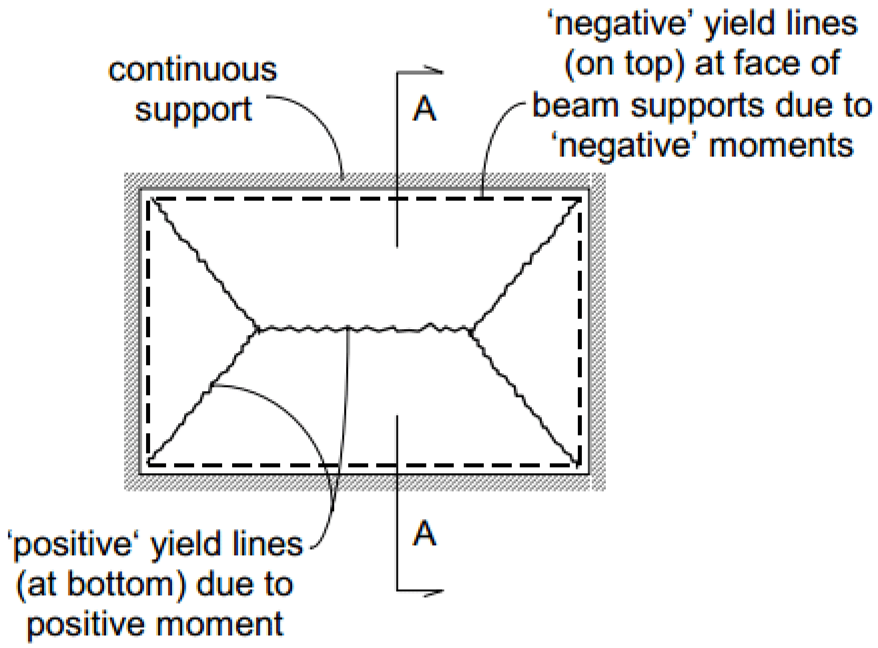
f) Supported edges of the slab will also act as axes of rotation. However, the fixed supports provide constant resistance to rotation having negative yield lines at the supported edges. On the other hand, axes of rotation at the simply supported edges will not provide any resistance to rotation of the segment.

Axis of rotation will pass over any column support, if provided, whose orientation will depend on other considerations.

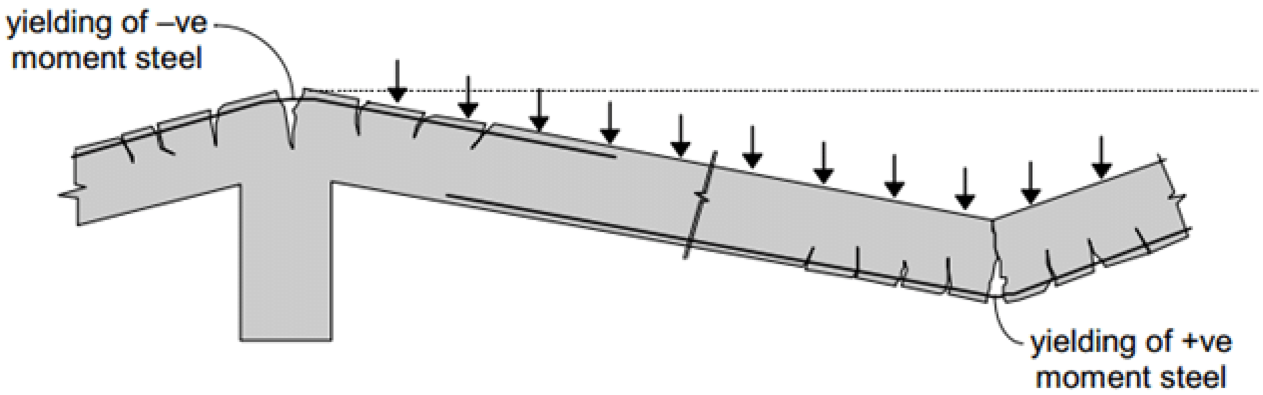
Typical interior panel in a two-way slab system



Yield line pattern under uniformly distributed collapse load



Deflected shape at collapse (section A-A)



Upper and Lower Bound Theorems:

According to the general theory of structural plasticity, the collapse load of a structure lies in between the upper bound and lower bound of the true collapse load. Therefore, the solution employing the theory of plasticity should ensure that lower and upper bounds converge to the unique and correct values of the collapse load.

The statements of the two theorems applied to slabs are given below:

(A) Lower bound theorem: The lower bound of the true collapse load is that external load for which a distribution of moments can be found satisfying the requirements of equilibrium and boundary conditions so that the moments at any location do not exceed the yield moment.

(B) Upper bound theorem: The upper bound of the true collapse load is that external load for which the internal work done by the slab for a small increment of displacement, assuming that moment at every plastic hinge is equal to the yield moment and satisfying the boundary conditions, is equal to the external work done by that external load for the same amount of small increment of displacement.

Thus, the collapse load satisfying the lower bound theorem is always lower than or equal to the true collapse load. On the other hand, the collapse load satisfying the upper bound theorem is always higher than or equal to the true collapse load.

The yield line analysis is an upper bound method in which the predicted failure load of a slab for given moment of resistance (capacity) may be higher than the true value. Thus, the solution of the upper bound method (yield line analysis) may result into unsafe design if the lowest mechanism could not be chosen. However, it has been observed that the prediction of the most probable true mechanism in slab is not difficult. Thus, the solution is safe and adequate in most of the cases.

However, it is always desirable to employ a lower bound method, which is totally safe from the design point of view.

Methods of Yield Line Analysis

1. Virtual work method
2. Equilibrium method (Equilibrium of individual elements of slab along yield line)

Formulae:

1. Square Slab

- ✓ Simply Supported at Edges: **Ultimate moment = mu = $W_u l^2/24$**
- ✓ Fixed Supported at Edges: **Ultimate moment = mu = $W_u l^2/48$**

2. **Rectangular Slab**
   - ✓ Simply Supported at Edges, **Ultimate moment, M_u = $[(W_u\ \alpha^2 l^2)/24]*\ [\sqrt{}\ (3+\mu\alpha^2) - \alpha\sqrt{}\mu^2]$**
3. **Equilateral Triangle**
   - ✓ Simply supported at edge, **Ultimate moment, $M_u = W_u l^2/72$**
4. **Generalized Triangular slab**
   - ✓ Simply supported along two edges, third edge being free, and isotropically reinforced **Ultimate moment, $M_u = W_u \alpha l^2/6$**
5. **Circular slab**
   - ✓ Simply supported at edge, **Ultimate moment, $M_u = W_u r^2/6$**
6. **Hexagonal**
   - ✓ Simply supported at edge, **Ultimate moment, $M_u = W_u r^2/8$**

Problem 1

Design a square slab fixed along all four edges, which is of side 5m. The slab has to support a service load of 4kN/m^2. Use M20 concrete and Fe415 steel.

As per IS 456-2000,

l/d = (0.8 x 35) = 28,

= 5000 / 28 = d

d = 178.6mm = 180mm

Provide D = 200mm

Loading on slab

Self weight = 0.2 x 25 = 5 kN/m^2

Live load = 4 kN/m^2

Floor finish = 1 kN/m^2

Total = 10 kN/m^2

Factored load (wu) = 1.5 x 10 = 15 kN/m^2

By yield line theory, $mu = wuL^2 / 48 = 7.8125 kNm$

Limiting moment, Mulim = $0.138.fck.b.d^2 = 0.138 \times 20 \times 1000 \times 180^2 = 89.424 \times 10^6$ Nmm

Mu < Mulim

K = Mu/bd^2 =

0.241 Ast =

122mm^2

Provide 8mm @ 300mm c/c

Problem 2

Design a rectangular slab of size 4m x 6m simply supported along all its edges, subjected to a live load of 4kN/m^2. The coefficient of orthotrophy is 0.7. Use M20 and Fe415.

For four edges simply supported condition,

Assume span/depth = 28 Eff.depth = 4000/28 = 142.86mm

Provide d = 150mm, D = 170mm Loading on slab:

Self weight = 0.17 x 25 = 4.25 kN/m^2

Live load = 4 kN/m^2

Floor finish = = 0.75 kN/m^2

Total = <u>9 kN/m^2</u>

Factored load (wu) = 1.5 x 9 = 13.5 kN/m^2

Ultimate moment, M_u = $[(W_u\ \alpha^2 l^2)/24]*\ [\sqrt{}\ (3+\mu\alpha^2) - \alpha\sqrt{}\mu^2]$ = 8.8209*1.602 = 14.13 KN.m

Limiting moment, Mulim = 0.138.fck.b.d^2 = 62.1 kNm

Mu < Mulim. The section is under-reinforced.

K = Mu/b.d^2 Ast = 277mm^2

Minimum Ast = 0.12% of c/s = 204mm^2

Ast along longer span = span0.7x237 =194$mm^{2} <$ 204 mm^2

Provide 8mm @ 240mm c/c

Problem 3

Design an equilateral triangular slab of side 5m, isotropically reinforced and is simply supported along its edges. The slab is subjected to a superimposed load of 3kN/m2. Use M20 concrete and Fe415 steel.

Assume span/depth = 28, Eff.depth = 5000/28 =

178.57mm Provide d = 180mm, D = 200mm

Loading on slab:

Self weight = 0.2 x 25 = 5 kN/m^2

Live load = 3 kN/m^2

Floor finish = 1 kN/m^2

Total = = 9 kN/m^2

Factored load (wu) = 1.5 x 9 = 13.5 kN/m^2

Limiting moment, Mulim = 0.138.fck.b.d^2 = 89.42 kNm Mu < Mulim.

The section is under-reinforced.

K = Mu/b.d^2 Ast = 72.754mm^2

Minimum Ast = 0.12% of c/s = 240mm^2

Ast < min Ast [240 mm^2]

Provide 8mm @ 200mm c/c

SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

UNIT – III – CIRCULAR SLABS AND FLAT SLABS – SCI1308

UNIT-III CIRCULAR SLABS AND FLAT SLABS

Limit state design of circular slabs - Simply supported and fixed with UDL. Design of flat slabs by direct design method according to IS code.

Introduction

The bending of a circular slab is essentially different from a rectangular slab where bending takes place in distinctly two perpendicular directions along the two spans.

When a circular slab simply supported at the edge is loaded with uniformly distributed load, it bends in the form of saucer, due to which stresses are develop both in the radial as well as in the circumferential directions. The tensile radial and circumferential stresses develop towards the converse side of the saucer and hence reinforcement need to be provided both in the radial as well as circumferential directions, but this arrangement would cause congestion and anchoring problem at the centre of the slab.

Hence an alternative method of providing reinforcement is adopted: reinforcement is provided on the form of a mesh of bars having equal area of cross section in both the directions, the area being equal to that required for the bigger of the radial and circumferential moments.

Moment Distribution of Circular Slab with different boundary conditions

Consider only two boundary conditions with uniformly distributed load (UDL).

a. Circular slab with simply supported ends
b. Circular slab with fixed supported ends

a. Circular Slab with simply supported ends having UDL

Let us consider a circular slab having

W → Uniformly distributed load (UDL)

a → Radius of slab

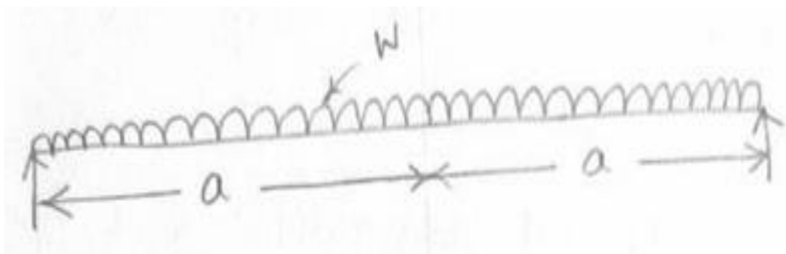
M_r → Radial bending moment at any point at 'r' distance from the centre of slab

$(M_r)_c$ & $(M_r)_e$ → Radial bending moments at centre and edge respectively

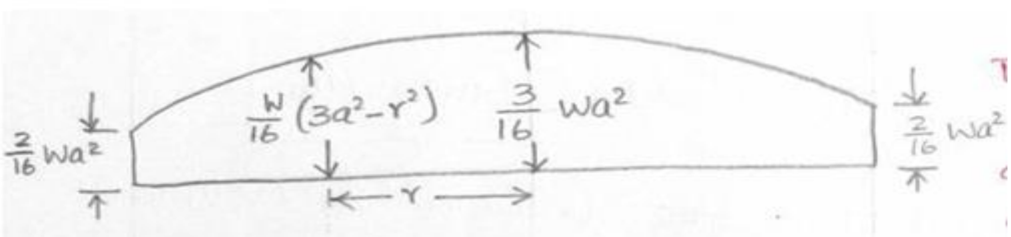
M_θ → Circumferential bending moment at any point at 'r' distance from the centre of slab

$(M_\theta)_c$ & $(M_\theta)_e$ → Circumferential moments at centre and edge respectively

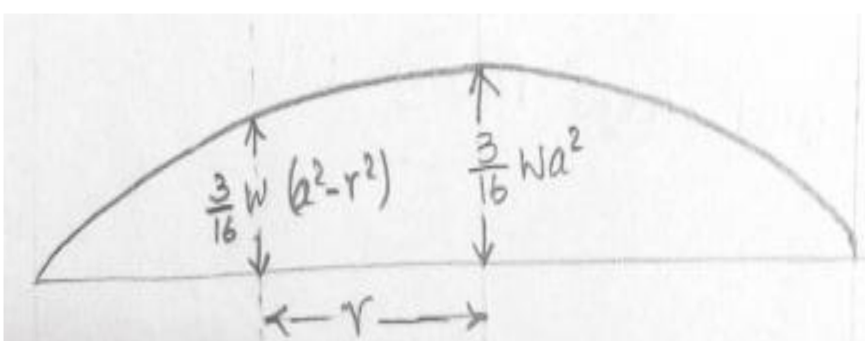
Loaded circular slab with simply supported ends having UDL



Circumferential moment M_θ distribution diagram along any diameter



Radial moment M_r distribution diagram along any diameter



The various values of moments and shear per unit width are as under:

The circumferential moment at centre of a slab $(M_\theta)_c$ = ± $(3/16)* Wa^2$

The circumferential moment at edge of a slab $(M_\theta)_e$ = ± $(2/16)* Wa^2$

The circumferential moment at any distance 'r' from centre (M_θ) = ± $(1/16)*[W* (3 a^2 – r^2)]$

The radial moment at centre of a slab $(Mr)_c$ = ± $(3/16)* Wa^2$

The radial moment at edge of a slab $(Mr)_e$ = 0

The radial moment at any distance 'r' from the centre (M_r) = ± $(3/16)*[W*(a^2 – r^2)]$

Note: The circumferential shear force is zero at everywhere.

The radial shear force V_u at any radius 'r' is given by V_u = 1/2 W*r per unit width

b. Circular Slab with fixed ends having UDL

Let us consider a circular slab having

W → Uniformly distributed load (UDL)

a → Radius of slab

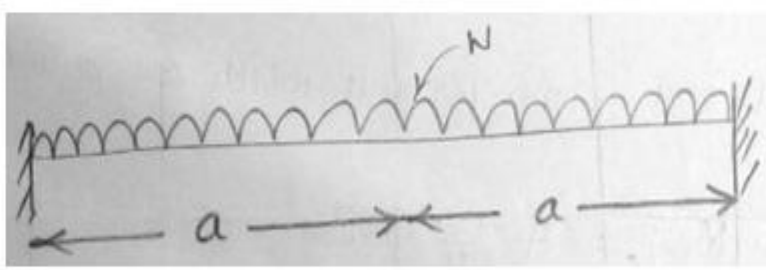
M_r → Radial bending moment at any point at 'r' distance from the centre of slab

$(M_{r)c}$ & $(M_r)_e$ → Radial bending moments at centre and edge respectively

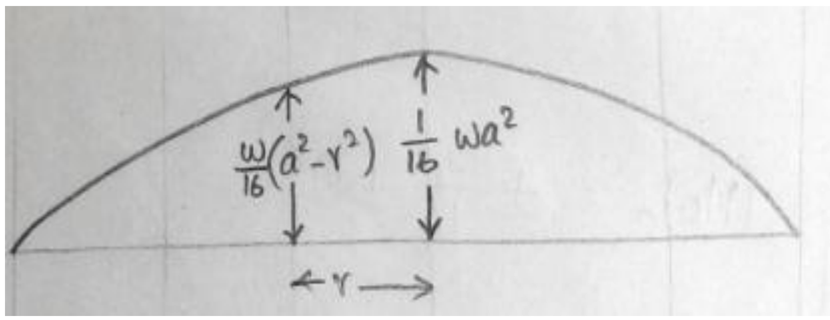
M_θ → Circumferential bending moment at any point at 'r' distance from the centre of slab

$(M_\theta)_c$ & $(M_\theta)_e$ → Circumferential moments at centre and edge respectively

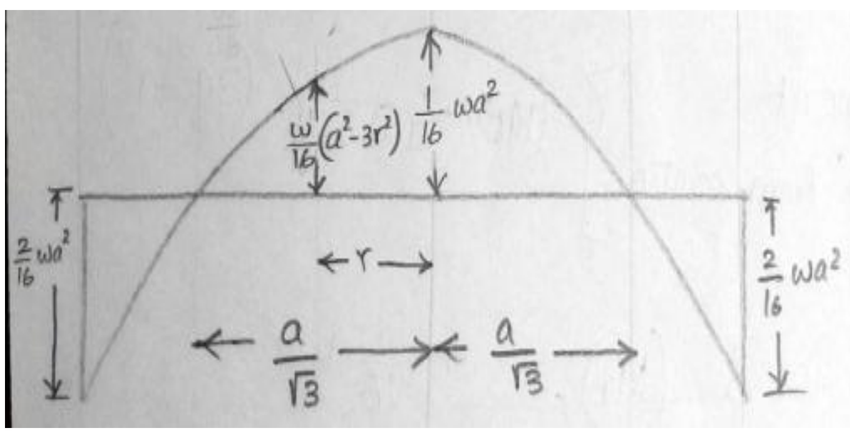
Loaded circular slab with fixed ends having UDL



Circumferential moment M_θ distribution diagram along any diameter



Radial moment M_r Distribution diagram along any diameter



Note: In radial moment, the point of contra flexure occurs at a distance of a/3 from the centre of the slab.

The various values of moments and shear per unit width are as under:

The circumferential moment at centre of a slab $(M_\theta)_c$ = ± (1/16)* Wa^2

The circumferential moment at edge of a slab $(M_\theta)_e$ = 0

The circumferential moment at any distance 'r' from centre (M_θ) = ± (1/16)*[W* ($a^2 - r^2$)]

The radial moment at centre of a slab $(Mr)_c$ = ± (1/16)* Wa^2

The radial moment at edge of a slab $(Mr)_e$ = ± (2/16)* Wa^2

The radial moment at any distance 'r' from the centre (Mr) = ± (1/16)*[W*($a^2 - 3r^2$)]

Slabs partially fixed at edges

It is an intermediate case between the circular slab with simply supported and fixed edges. The moment values are average between these two cases. In radial moment, the point of contra flexure is present at a 2/3 distance from the centre.

The various values of moments and shear per unit width are as under:

The circumferential moment at edge of a slab $(M_\theta)_e$ = + (1/16)* Wa^2

$(Mr)_c = (M_\theta)_c$ = ± (2/16)* Wa^2

The radial moment at edge of a slab $(Mr)_e$ = - (1/16)* Wa^2

Design Problem

A circular room has 5m diameter. Design a circular roof slab for room to carry a super imposed load of 3750 N/m^2. Assume that the slab is simply supported at the edges, use M20 concrete and Fe415 steel.

L/d = 20 (According to IS 456:2000 for simply supported span up to 10m).

<u>No special condition for circular slab in IS 456:2000</u>

Assume L/d = 1.333 x 20 = 26.66

<u>Note:</u> Deflection of circular slab is 1/3rd of deflection of rectangular slab.

<u>Modification of L/d ratio for effective depth (D) calculation:</u>

Assume 75% of $(P_t)_{lim}$

P(t) = 0.75 x 0.957 = 0.718%

Assume P(t) = 0.72%

<u>The modification factor for tensile reinforcement</u>

The stress in tensile reinforcement (f_s) = 0.58 fy

Area of c/s steel required/provided f($_s$) =0.58 fy (1) (

Assume Ast required = Ast provided)

f(s) = 0.58 x 415 = 240.7 N/mm^2

Modification factor (M.F.) = fs = 240.7 N/mm^2 and P(t) = 0.72% M.F.

= 1.1

Modified L/d ratio = 26.667 x 1.1 = 29.337

Assume L/d = 30

D = 5000/30 = 166.667 = 167 mm

Assume clear cover of 20mm and 12mm bar

Total depth (D) = 167 + 20 + 6 = 193mm

Assume 200mm = D (Total depth)

Effective depth = 200 – 20 – 6 = 174mm

<u>Load calculation:</u>

Dead load = 0.2 x 1 x 25 = 5 x 103 = N/m^2

Super imposed load = 3750 N/m^2

Total load = 8750 N/m^2

Factored load = 1.5 x 8750 = 13125 N/m^2

Factored load per m width w_u = 13125 x 1

W_u = 13125 N/m

<u>The radial and circumferential moment at a centre of a slab</u>

$(Mur)_c=(Mu_{\Theta})_c$ =5380 N-m

$(M_{u\Theta})_e$= 10253.9N.m

<u>Computation of effective depth(D):</u>

Mu= C x lever arm

$(M_u)_{max}$ =0.36fckbx_u x(d-0.42x_u)

D = 74.66mm 174mm.

Note:

Required effective depth 74.66mm is very less than provided depth 174mm. We can reduce this for an economical design. But we derived this effective depth 174mm from a deflection criteria (L/d). So to avoid an over deflection, we can maintain proceed a design with an effective ratio, obtain from L/d ratio d = 174mm.

Computation of Ast:

M_u = T x lever arm

$15380X10^3$=0.87fyAst (d-0.42x_u)

Substitute the values in the above equation

A_{st} = 8009.6mm^2,252.535mm^2

($A_{st})_{centre}$ = 252.535mm^2

Provide 12mm ϕ bar,

S = 447.848mm

Provide c/c distance of a bar as 300mm

Spacing $_{(S)}$ =376.99mm^2

Hence provide the reinforcement in the form of a mesh. Therefore provide 12mm bar in both direction at 300mm c/c

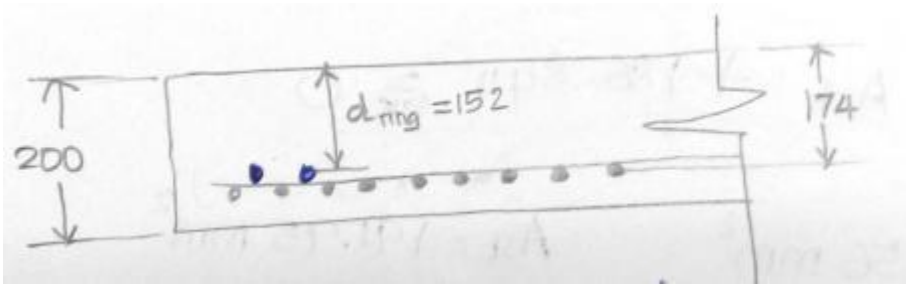
Near the edges, the bars are not have proper anchorage, since they are free and there will be slipping tendency. Hence the bars will not be capable of taking any tension.

At an edge, the radial tensile stress is zero, but a circumferential moment is present in it. So we need to arrest the slippage by providing circumferential reinforcements.

It is a circular slab, so we can provide the circumferential reinforcements in the form of a ring.

Available 'd' for a ring (d_{ring}) = 200 – 20 – 12 – 12 – 8/2

D_{ring} = 152mm



<u>Ast for circumferential reinforcement</u>

Circumferential reinforcement is provided for an end moment

$(M_{u\theta})_e$	= T x leverarm
$10253.906X10^3$	= 0.87fyAst (d-0.42x_u)
	= 0.00033Ast

Substitute the values in the above equation

Ast =7023.096mm^2 ,191.95mm^2

Ast for circumferential moment =191.95mm^2

S=250mm

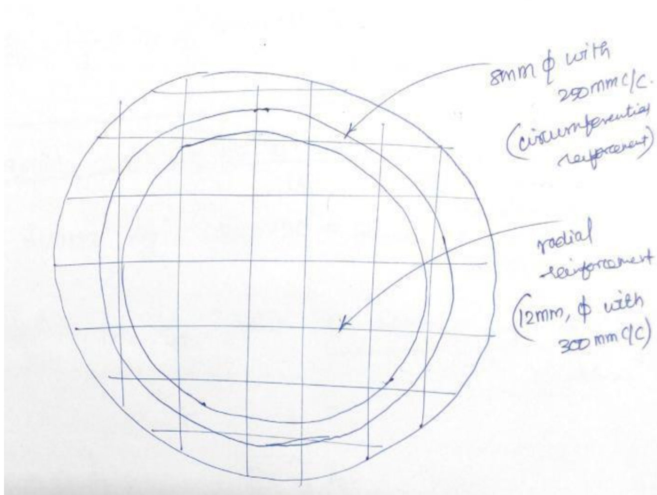
Circumferential reinforcement is anchorage reinforcement. The length of anchorage reinforcement is equal to its development length. (Ld)

L_d = 47 = 47 x 8

L_d = 376mm

No. of rings = Ld/s = 376/250 = 1.5 = 2 nos.

Provide 2 nos. of 8mm rings with 250mm c/c



DESIGN OF FLAT SLAB

Introduction

Slabs which directly rest on columns without any beams are flat slabs. However peripheral beams may be provided.

Advantages

- Aesthetics,
- Reduced form work cost,
- Free passage for AC ducts/wiring,
- Reduced floor height resulting more no. of storeys in a given height of building.

Disadvantages

- Lateral stiffness is less in the absence of beams,
- Lateral load path is not well defined and hence the behavior is suspect.

Definitions

Column drops

It is a portion of slab surrounding the column thickened to take care of shear.

Column capital

Enlarging the column at its junction with the slab/drop to take care of shear and also to reduce the effective span is Column capital

Two methods to analyze a flat slab

a. Direct design method
b. Equivalent design method

Design Problems

A Ware house of plan area 60 m x 40 m has to be provided with flat slab system. Use M_{20} concrete and Fe $_{415}$ steel impose load 5 KN/ Sqm . Adopt floor finish of thickness 50 mm.

Step 1:Spacing of columns:

Use 6 m along long direction

And 5m along short direction

Hence l_1	6	m
Hence l_2	5	m

Size of columns

Assume square columns as 500mm side

Step 2:Provide column capitals / heads

They are square in plan

Dimensions (l_1 + l_2)/8	1.375	m

Say Column drops	1.40	m
Minimum $l_1/3$ x $l_2/3$		
6/3 by 5/3	2 m x 1.67 m	
Say along long direction	2	m
Short direction	2	m

Step 3:Consider design along long direction

$l_n = l_1$ Column capital		
6-1.4	4.6	m
But not < $0.65l_1$	3.9	m
Hence l_n	4.6	m

Step 4:Thickness of slab

d_s =ln /26	0.176	m
Assume 20mm cover and	20	mm
12 mm dia bars	12	mm
Ds	202.92	mm
Say D_s	210	mm
Hence d_s = 190-20-12/2	184	mm

Step 5:Thickness of drop

$D_D = 1.25D_s$	262.5	mm

Say D_D	270	mm
d_D=240-20-12/2	244	mm

Step 6: Loading

Self weight of slab

Average thickness

Considering Drop and slab

$(D_D+Ds)/2$

Self weight 0.240 x 25	6	kN/m^2

Floor Finish = (50/1000) x 24	1.2	kN/m^2
Imposed load	5.0	kN/m^2
Total load (w)	12.2	kN/m^2
W_u = 1.5 x w	18.3	kN/m^2

Step 7:Panel BM

$M_o = Wl_n/8$

Where $W = w_u \times l_x l_n$

Mo	242.01	

(a)Division of M_o in to +ve and negative moments

Negative BM = 0.65 M_o	157.31	kN m
Positive BM = 0.35 M_o	84.70	kN m

(b)Column strip moments

Width of strip = $0.5l_2$	2.5	m
Negative BM = 0.75x157.31	117.98	kN m
Negative BM / per m width		
Negative BM / column strip width	47.19	kN m
Positive BM / Per m Width =50.82/2.5	20.32	kN m

(c)Middle strip moments

Width of middle strip = l_2-CS		
5-2.5	2.5	m
Negative BM = 0.25x157.31	39.32	kN m
Negative BM / per m width		
Negative BM / per middle strip width	15.73	kN m
Positive BM = 0.4 x 84.71	33.88	kN m
Positive BM / Per m Width =33.82/2.5	13.55	kN m
(d)Design BM		
Highest BM, Mu		
Negative moment in column strip	47.19	kN m
Governing depth = d_D	244	mm
Step(8):Check for Mu lim		

Mu lim=0.38 bd^2f_{ck}

b= 1000 mm

d = 244 mm

Fck = 20 N/mm^2

Mu < Mu,lim

R = M_u/bd^2	0.793	
Fy	415	N/mm^2
Ast	563	mm^2
Min A_{st} = 0.12% = 0.12 / 100 (b D_D)	324	mm^2
Using 12 dia bars spacing		
(area of 1 bar)/A_{st} x 1000	200.83	mm
Say	200	mm
Maximum spacing 3 d_d or 300		
3 d_D	732	mm
	300	mm
Provide spacing	200	mm

Step(9):Check for shear

There are 2 critical sections

One $d_D/2$ column capital

Another ds/2 from column drop

(a) at $d_D/2$ from column capital

Breadth of critical section B

Column capital + d_D = 1400 +244	1644	mm

Punching shear = w_u (l_1+l_2-B^2)	1048.53	kN
Perimeter of resistance b_0= 4 x B	6576	mm
T_v = Punching shear / ($b_0 d_D$)	0.65	N/mm^2
Permissible shear = t_c = 0.25√fck	1.18	0.k

(b) at d_s/2 from drop

Breadth of critical section B		
Column capital + d_D = 2000 +184	2184	mm
Punching shear = w_u (l_1+l_2-B^2)	461.71	kN
Perimeter of resistance b_0= 4 x B	8736	mm
T_v = Punching shear / ($b_0 d_s$)	0.28	N/mm^2
Permissible shear = t_c = 0.25√f_{ck}	1.18	0.k

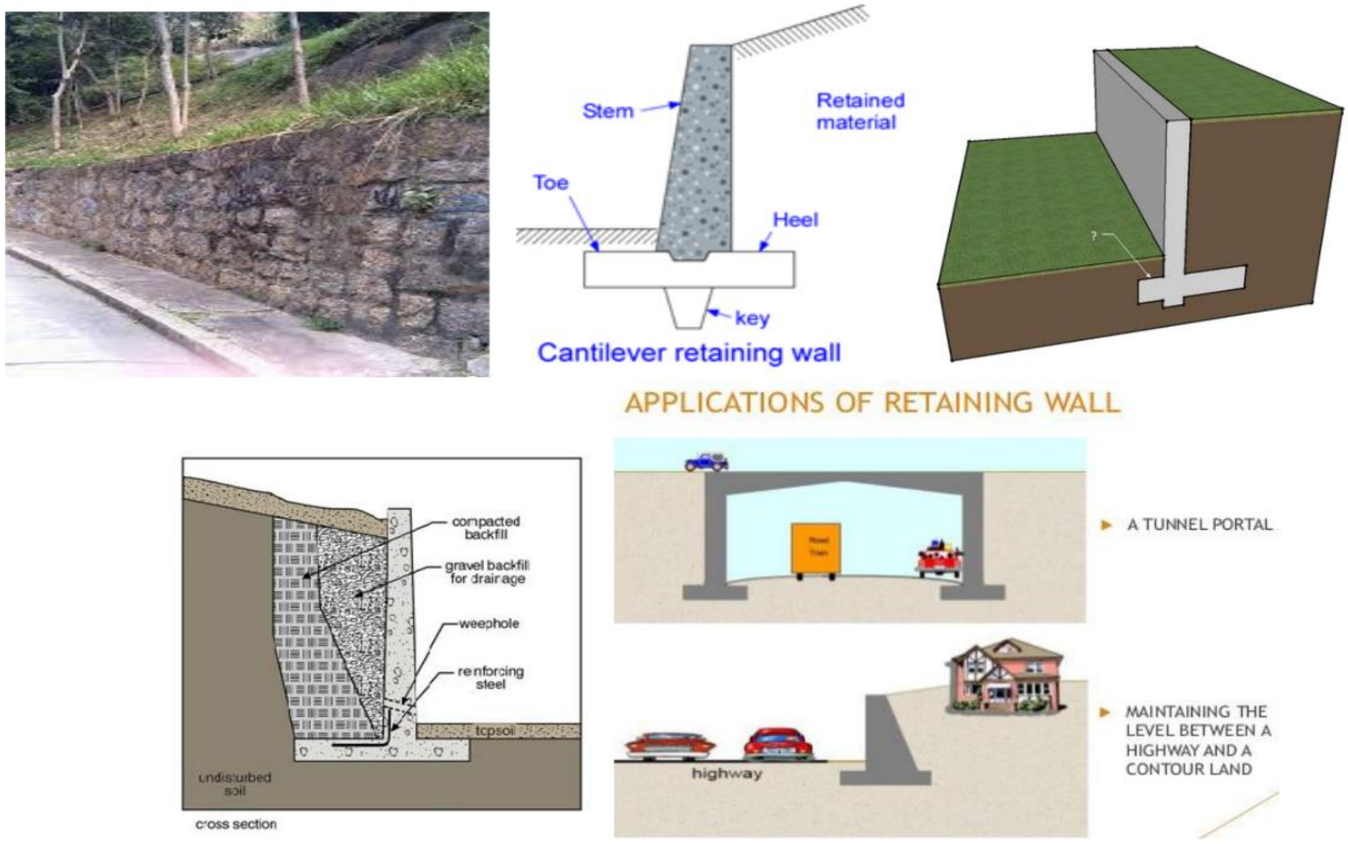
SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

UNIT – IV – RETAINING WALL – SCI1308

INTRODUCTION

Retaining walls are usually built to hold back soil mass. However, retaining walls can also be constructed for aesthetic landscaping purposes. These are the structures that are constructed to retail soil or any such materials which are unable to stand vertically by themselves. In other words, these are used to retain earth or other materials in a vertical or nearly vertical position at locations where a change in the ground level occurs. The wall prevents the retained from assuming its natural angle of repose. This causes the retained earth to exert a lateral pressure on the wall thereby tend to bend overturn and slide the retaining wall structure. The application of the RW is mentioned in the below figure.



They are also provided to maintain the grounds at two different levels. In general, retaining walls can be divided into two major categories:

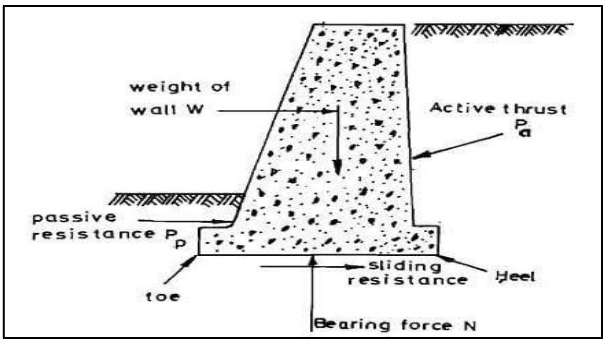
(a) Conventional retaining walls, and

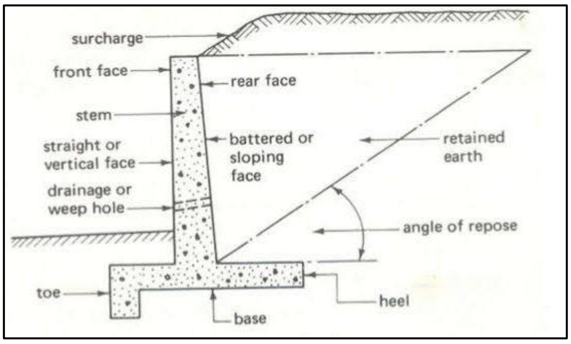
(b) Mechanically stabilized earth walls.

TYPES OF RETAINING WALL

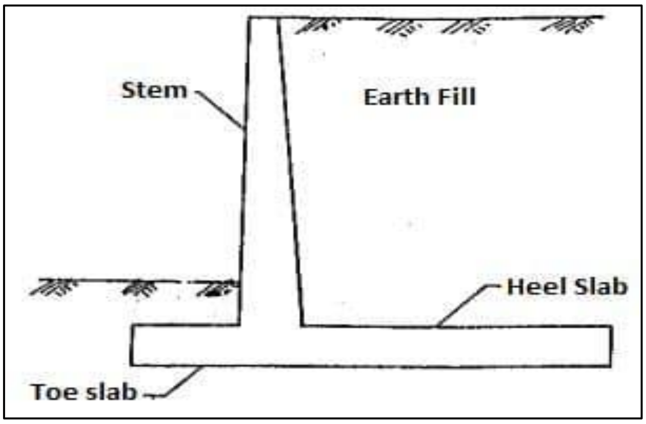
Conventional retaining walls can generally be classified as

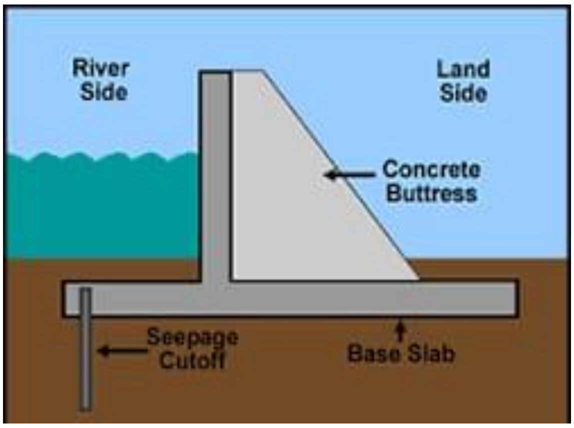
1. Gravity retaining walls - Masonry wall < 3 m high
2. Semi-gravity retaining walls
3. Cantilever retaining walls (Inverted T and L) - RCC 3 to 6 m high
4. Counterfort retaining walls - Height > 8 m
5. Buttress wall- Transverse stem support provided on front side



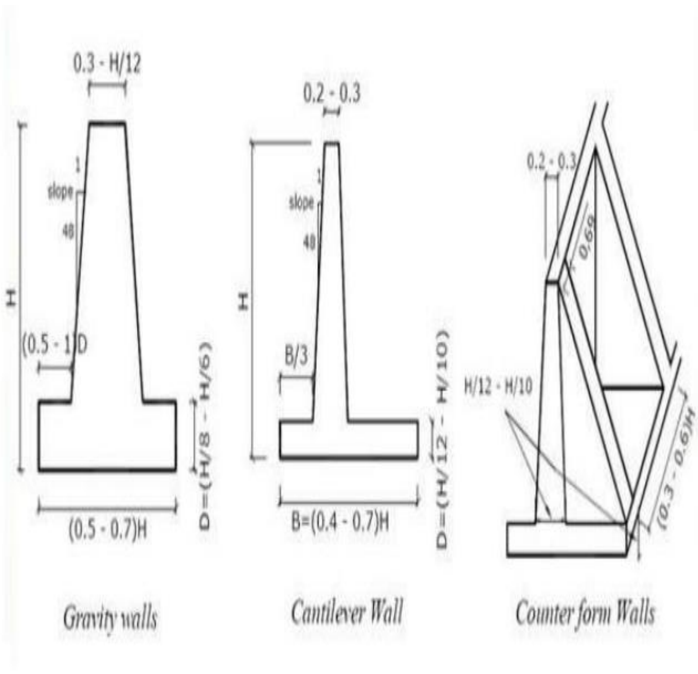


a) Gravity Retaining Wall b) Counterfort retaining Wall

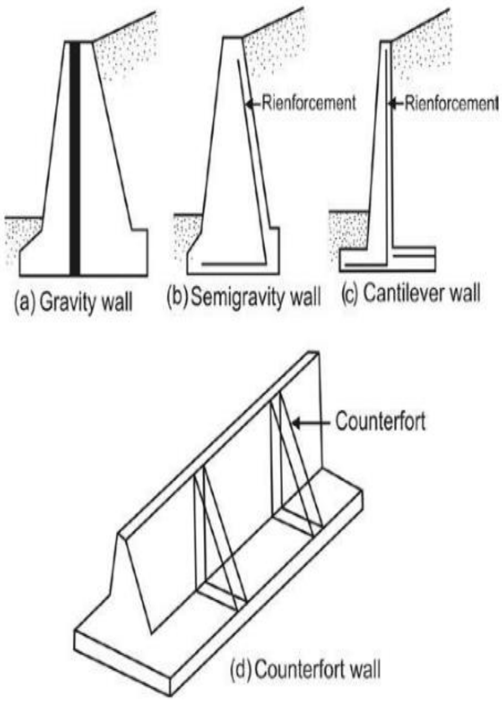




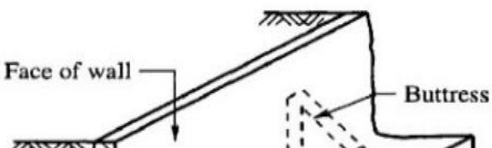
c) Cantilever retaining wall d) Buttress Retaining wall



Types of Retaining Wall



BUTTRESS WALL



Gravity retaining walls are constructed with plain concrete or stone masonry. They depend on their own weight and any soil resting on the masonry for stability. This type of construction is not economical for walls greater than 3m.

In many cases, a small amount of steel may be used for the construction of gravity walls, thereby minimizing the size of wall sections. Such walls are generally referred to ***as semi-gravity walls***

Cantilever retaining walls are made of reinforced concrete that consists of a thin stem and a base slab. They may be L or T type walls. This type of wall is economical to a height of about 3 to 6m.They Stability of the wall is provide by the weight of earth on heel slab and self weight of structure.

Counterfort retaining walls are similar to cantilever walls. When height of retaining wall exceeds about 8 m and when thickness of stem, heel toe becomes uneconomical counterforts may be provided at regular intervals, they have thin vertical concrete slabs known as counterforts that tie the wall and the base slab together. The purpose of the counterforts is to reduce the shear and the bending moments.

There are ***two phases in the design of conventional retaining walls***.

- First, with the lateral earth pressure known, the structure as a whole is checked for stability. That includes checking for possible overturning, sliding, and bearing capacity failures.
- Second, each component of the structure is checked for adequate strength, and the steel reinforcement of each component is determined.

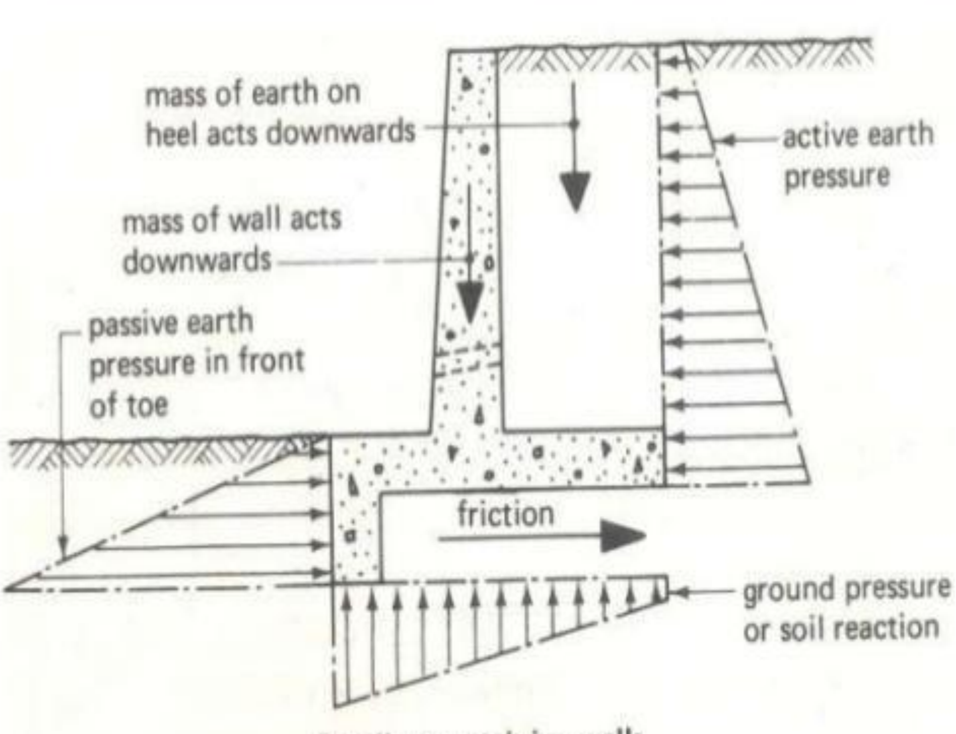
General Guidelines

To design retaining walls properly, an engineer must know the basic soil parameters-that is, the unit weight, angle of friction, and cohesion-for the soil retained behind the wall and the soil below the base slab. Knowing the properties of the soil behind the wall enables the engineer to determine the lateral pressure distribution that has to be designed for.

Earth Pressure (P)

Earth pressure is the pressure exerted by the retaining material on the retaining wall. This pressure tends to deflect the wall outward. There are two types of earth pressure and they are;

- Active earth pressure or earth pressure (Ka) and
- Passive earth pressure (Kp).



Cantilever retaining walls

Active earth pressure – It is the lateral pressure developed at the onset of shear failure by wall moving away from the soil in the direction of the acting earth pressure. The pressure exerted by the soil towards the structure. Active earth pressure tends to deflect the wall away from the backfill. Earth pressure depends on type of backfill, the height of wall and the soil conditions.

Passive earth pressure - It is lateral pressure developed at the onset of shear failure by wall moving (penetrating) in the direction opposite to the direction of acting earth pressure. The pressure exerted by the structure towards the soil.

<u>Proportioning Retaining Walls</u>

When designing retaining walls, the *proportioning* of retaining wall allows the engineer to check trial sections for stability. If the stability checks yield undesirable results, the sections can be changed and rechecked. The figure below shows the general proportions of various retaining walls components that can be used for initial checks.

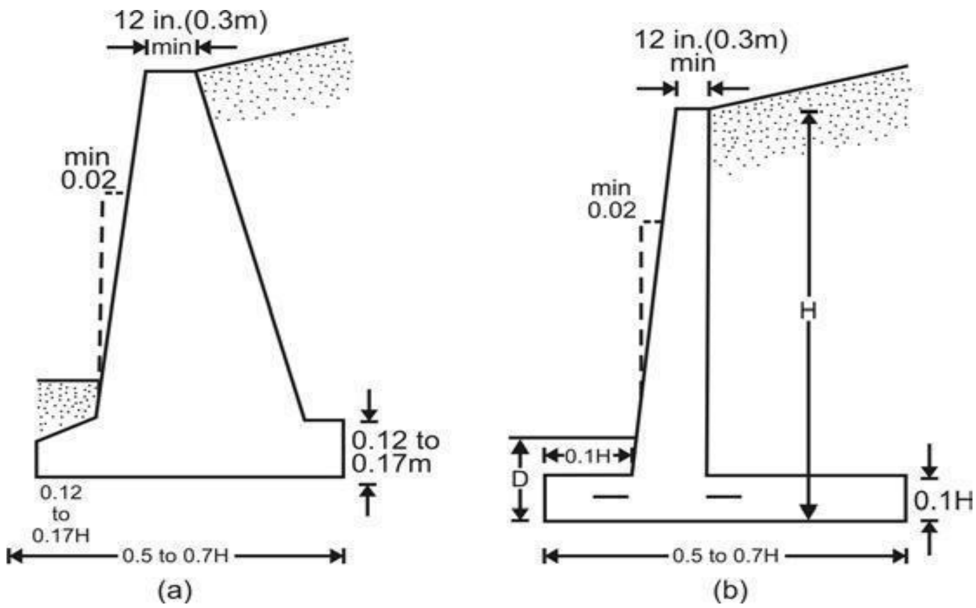
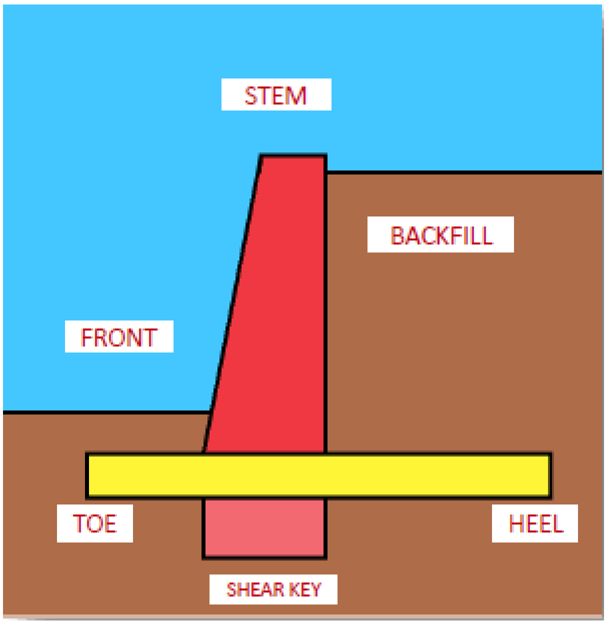


Figure: Approximate dimensions for various components of retaining wall for initial stability is a) gravity wall; (b) cantilever wall

<u>COMPONENTS OF RETAINING WALL</u>



1. STEM
   - ❖ Vertical stem in cantilever retaining wall resists earth pressure from backfill side and bends like a cantilever.
   - ❖ The thickness of cantilever slab is larger at the base of stem and it decreases gradually upwards due to reduction of soil pressure with decrease in depth.

2. BASE SLAB
   - ❖ The base slab forms the foundation of the retaining wall.
   - ❖ It consists of ***a heel slab and a toe slab***.
   - ❖ The ***heel slab*** acts as a horizontal cantilever under the combined action of the weight of the retaining earth from the top and the soil pressure acting from the soffit.
   - ❖ The ***toe slab*** also acts as a cantilever under the action of the soil pressure acting upward.
   - ❖ The stability of the wall is maintained by the weight of the earth fill and on the heel slab together with the self-weight of the structural elements of the retaining wall.
   - ❖ Cantilever type retaining walls are suitable up to 5m depth of backfill.

3. SHEAR KEY
   - ❖ The main purpose of installation of shear keys is to increase the extra passive resistance developed by the height of shear keys.
   - ❖ However, active pressure developed by shear keys also increases simultaneously.

❖ The success of shear keys lies in the fact that the increase of passive pressure exceeds the increase in active pressure, resulting in a net improvement of sliding resistance.

❖ On the other hand, friction between the wall base and the foundation soils is normally about a fraction of the angle of internal resistance (i.e. about 0.8 p) where p is the angle of internal friction of foundation soil.

❖ When a shear key is installed at the base of the retaining wall, the failure surface is changed from the wall base/soil horizontal plane to a plane within foundation soil.

❖ Therefore, the friction angle mobilized in this case is p instead of 0.8p in the previous case and the sliding resistance can be enhanced.

4. BACKFILL

❖ Backfill refers to the dirt behind the wall.

❖ In order to provide proper drainage, at least 12 inches of granular backfill (gravel or a similar aggregate) should be installed directly behind the wall.

❖ Compacted native soil can be used to backfill the rest of the space behind the wall. If you intend to do landscaping behind the wall, a 6+ inch layer of native soil should also be placed over the gravel fill.

To check the stability of a retaining wall, the following steps are necessary

1. Check for *overturning* about its toe
2. Check for *sliding* along its base
3. Check for *bearing capacity failure* of the base
4. Check for *settlement*
5. Check for *overall stability*

Overturning (As per IS: 456-2000 Clause 20.1 Page No. 33)

The stability of a structure as a whole against overturning shall be ensured so that the restoring moment shall be not less than the sum of 1.2 times the maximum overturning moment due to the characteristic dead load and 1.4 times the maximum overturning moment due to the characteristic imposed loads. In cases where dead load provides the restoring moment, only 0.9 times the characteristic dead load shall be considered. Restoring moment due to imposed loads shall be ignored.

Sliding (As per IS: 456-2000 Clause 20.2 Page No. 33)

The structure shall have a factor against sliding of not less than 1.4underthe most adverse combination of the applied characteristic forces. In this case only0.9 times the characteristic dead load shall be taken into account.

Formulae for design of RW

Coefficient of active earth pressure, (K_a) = 1-sin ϕ/1+ sin ϕ

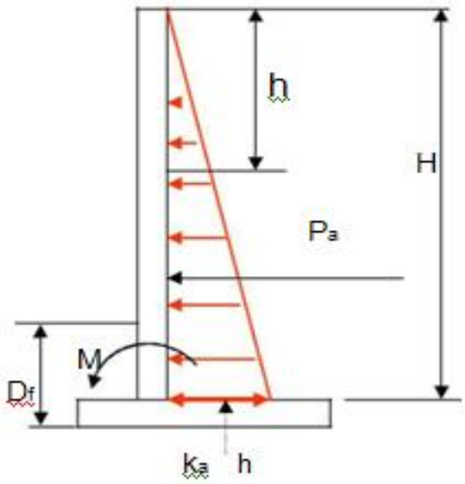
Coefficient of passive earth pressure, (K_p) = 1+ sin ϕ/1- sin ϕ

Minimum depth of foundation = [SBC/ϒ]*(1-sin ϕ/1+ sin ϕ)^2

Bending Moment, M = $K_a ϒ H^3/6$

Where,

M	*Maximum Bending Moment (KN.m)*
K_a	*Coefficient of active earth pressure*
K_p	*Coefficient of passive earth pressure*
ϕ	*Angle of Repose (in °)*
ϒ	*Unit weight of earth (KN/m^3)*
H	*Height of Wall (m)*
SBC	*Safe Bearing Capacity of soil (KN/m^2)*



DESIGN OF CANTILEVER RETAINING WALL

Problem (1) Design a cantilever retaining wall to retain earth embankment 4 m height above ground level. The unit weight of earth 18 kN/m^2. The embankment is horizontal at its top. The safe Bearing capacity of soil is 200 KN / m^{2}of the coefficient of friction between the soil and concrete is 0.5. Adopt M 20 grade concrete and Fe 415 steel. Take Factor of safety against overturning and sliding as 1.4.

***Step 1:* Given Data**

Ht of embankment above GL = 4 m

Unit weight of earth (ϒ) = 18 KN / m^3

Angle of repose (ϕ) = 30°

SBC = 200 KN / m^2

Coefficient of friction between soil and concrete = 0.5

f_{ck} = 20 N/mm^2

f_y = 415 N/mm^2

FOS = 1.4

Step 2: Dimension of Retaining Wall

Minimum depth of foundation = $[SBC/\Upsilon]*(1-\sin\phi/1+\sin\phi)^2$

$= [200/18]*(1-\sin 30^{\circ}/1+\sin 30^{\circ})^2$

= 1.23 m = 1.2 m

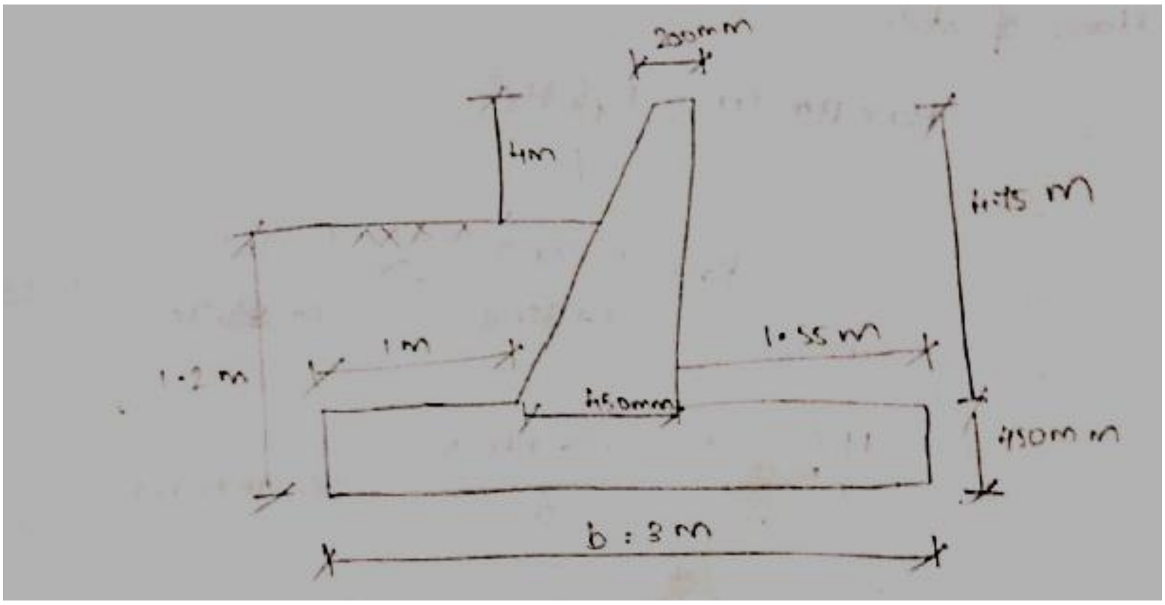
Overall height of wall, H = 4+1.2

= 5.2 m

Thickness of base slab = H/12

= 5.2/1.2 =0.43 m

= 430 mm ≈ 450 mm



Assume top width of stem as 200 mm

Bottom width of stem = Thickness of base slab

= 450 m

Width of base slab, b = 0.5 H to 0.6 H

= [0.5 * 5.2] to [0.6 * 5.2]

= 2.6 m to 3.12 m

Taking b as 3 m

Width of toe projection = 1/3 * b

=1/3*3

= 1

Width of heel projection = 3 – (1+0.45)

= 1.55 m

Step 3: Design of stem

Designed for maximum Bending moment at the bottom of stem

Max BM (M) = Ka $\Upsilon H_s^3/6$

To Find Ka =?

Ka = $1-\sin\phi / 1+\sin\phi$

= $(1-\sin 30^\circ) / (1+\sin 30^\circ)$

Ka = 0.33

BM = $(0.33*18*4.75^3) / 6$

= 106.10 KN.m

Factored BM, Mu = 1.5 * 106.10

= 159.12 KN.m

For Fe 415

Mu, Limit = $0.138\ fckbd^2$

B = 1000 mm

D = 450 mm (bottom width)

Assume an effective depth cover of 50 mm

D = D – d′

= 450 – 50

= 400 mm

Mu, limit = $0.138 * 20 * 1000 * 400^2$

= 441.6KN.m

Mu < Mu Limit

Therefore, Section is under reinforced

Mu = $0.87fy*Ast*d*(1 – \{(Ast*f_y) / (b*d*f_{ck})\})$

$59.15*10^6$ = 0.87*415*Ast *450 (1 – (Ast *415/1000*400*20))

Ast = 1173 mm^2

Ast min for Fe 415 = 0.12% of cross section

= 0.12/100 *1000*450

= 540 mm^2

Provide 16 mm diameter bar

Spacing of 16mmdia bar, S = 1000 *(π * 16^2/4) / 1173

= 171.45 mm

Provide 16 mm diameter bar @ 170 mm c/c

Distribution reinforcement

$Ast_{(Min)}$ = 540 mm^2

Provide 10 mm diameter bar

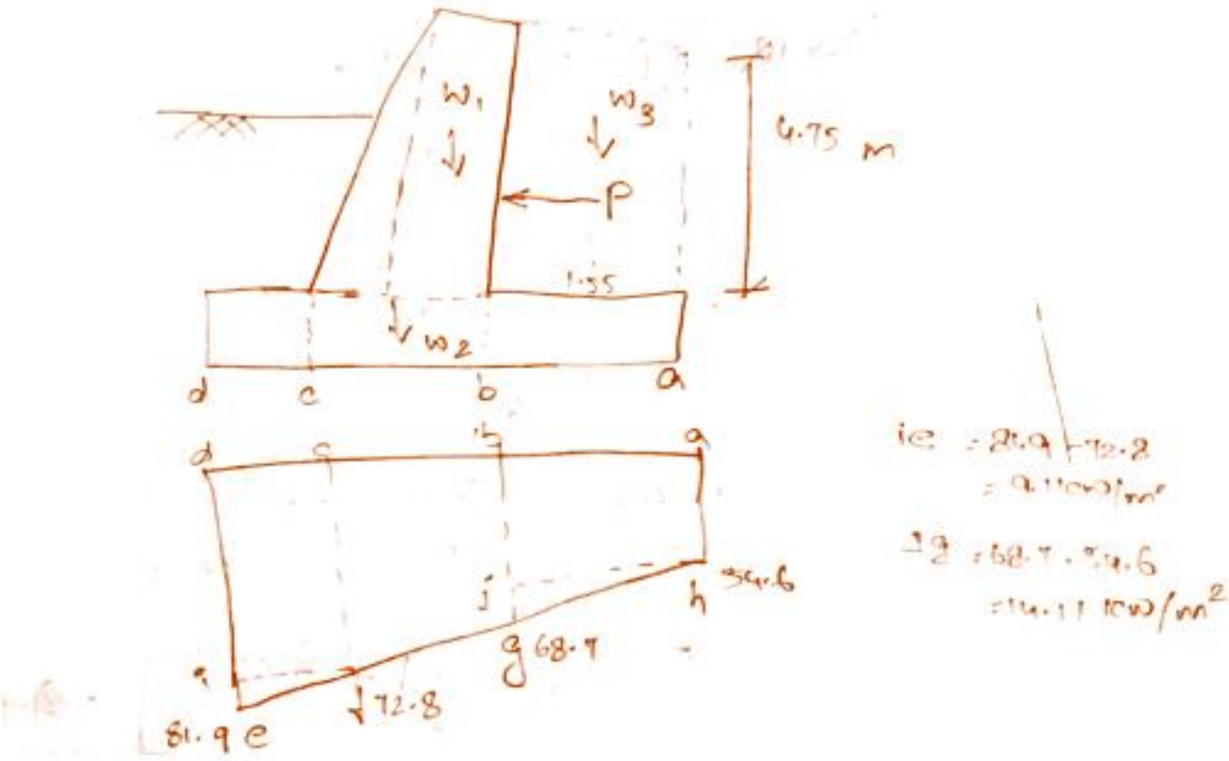
Spacing of 10 mm diameter bar, S = 1000 *(π * 10^2/4) / 540

= 145 mm

Provide 10 mm diameter bar @ 140 mm c/c

Step 4: Stability Calculation

Stability calculation for 1 m runs of wall



Loads	Magnitude of load (KN)	Distance from ‘a’ (m)	Moment (KN-m)
W1 – wt of stem Wt. of rectangular portion = 0.2 * 4.75 * 25	23.75	1.55+0.2/2 = 1.65	39.19
Wt of triangular portion = ½*0.25*4.75*25	14.8	1.55+0.2+1/3*0.25 = 1.83	27.13
W2 – wt of base slab = 3*0.45*25	33.75	3/2 = 1.5	50.63
W3 – wt. of retained earth	132.53	1.55/2 = 0.78	102.7

= 1.55*4.75*18			
Moment due to earth pressure from (step 3)	$\sum w = 204.83$		106.10

$\sum M = 325.15$ KN.m

Distance of point of application of resultant from ‘a’, Z $= \sum M / \sum W$

$= 325.75 / 204.83$

$= 1.6$ m

Eccentricity, e $= Z - b/2$

$= 1.6 - 3/2$

$= 0.1$

b/6 $= 3/6$

$= 0.5$

$e < b/6$

$P_{max} = \sum w / b\ (1 + 6e/b)$

$= 204.83/3\ (1 + 6*0.1/3)$

$= 81.9$ KN/m^2 at toe

$P_{min} = \sum w / b\ (1 - 6e/b)$

$= 204.83/3\ (1 - 6*0.1/3)$

$= 54.62$ KN/m^2 at heel

0 – 81.9

1 – ?

3 – 54.6

Interpolate

@1 = 81.9 – (81.9-54.6) (1.0) / 3

= 72.8 KN/m^2

0 – 81.9

1.45 – ?

3 – 54.6

Interpolate

@ 1.45 m = 81.9 – (81.9-54.6) (1.45-0) / 3

= 68.7 KN/m^2

Step 5: Design of Heel slab

Calculations for 1m run of wall

Loads	Magnitude of load (KN)	Distance from 'a' m	Moment (KN.m)
Downward Force Self wt of heel slab =1.55*0,45*25	17.44	1.55/2 = 0.78	13.49
Self wt of earth above heel slab = 1.55*4.75*18	132.53	1.55/2 = 0.78	103.37
Upward forces Upward pressure abjh =54.6*1.55	84.63	1.55/2 = 0.78	66.01
Upward pressure jgh =1/2*1.55*14.11	10.94	2/3*1.55 = 1.03	11.26

Downward moment = 13.49+103.37 = 116.8 KN-M

Upward Moment = 66.01+11.26 = 77.27KN- M

(-)

= 39.53 KN- M

Factored BM, Mu = 1 .5 * 39.53

= 59.29 KN- M

Mu, Limit = 441.6 KN-M *(from step 3)*

Mu < Mu limit

Therefore, Section is under reinforced

Mu = 0.87fyAst d (1 – Ast fy / b d fck)

$59.29 * 10^6$ = 0.87*415*Ast *450 (1 – (Ast *415/1000*400*20))

Ast = 420 mm^2

Fe 415, Ast_{min} = 0.12% of cross section

= 540 mm^2 (from step 3)

Provide 12 mm diameter bar

Spacing of 16mm diameter bar = 1000 *($\pi * 12^2/4$) /540

= 209 mm

Provide 12 mm diameter bar @ 200 mm c/c

Provide 16 mm diameter bar @ 140 mm c/c as distribution (step 3)

Step 6: Design of Toe slab

Loads	**Magnitude of load (KN)**	**Distance from 'a' m**	**Moment (KN-m)**
upward Force cdjf =72.82 *1	72.82	½ = 0.5	36.41
Fje = ½*1*9.1	4.5	2/3*1 = 0.67	3.071
Downward force Self wt of toe slab =1*0.45*25	11.25	½ = 0.5	5.62
Self wt of soil above toe slab = 1* 0.75*18	13.5	½ = 0.5	6.5

Upward moment = 36.41+3.071 = 39.48 KN-M

Downward Moment = 5.62+6.5 =12.12KN- M

(-)

= 27.36 KN- M

Factored BM,

Mu = 1 .5 * 27.36

= 41.04 KN- M

Mu, Limit = 441.6 KN-M (from step 3)

Mu < Mu limit

Therefore, Section is underreinforced

Mu =0.87fyAst d (1 – Astfy / b d fck)

$41.04 * 10^6$ =0.87*415*Ast *450 (1 – (Ast *415/1000*400*20))

Ast = $288 mm^2$

Fe 415, Ast_{min} = 0.12% of cross section

= $540 mm^2$(from step 3)

Provide 12 mm diameter bar

Spacing of 12 mm diameter bar = $1000 * (\pi * 12^2/4) / 540$

= 209 mm

Provide 12 mm diameter bar @ 200 mm c/c

Provide 16 mm diameter bar @ 140 mm c/c as distribution (step 3)

Step7 : Stability against Overturning

Factor of safety against Overturning = $0.9 M_r / M_o$

Resisting Moment (M_r) = $\sum W (b - z)$

= 204.8 (3-2.6) =2866.72 KN- M

Overturning Moment, M_0 = $Ra\Upsilon hs^3 / 6$

= $0.33*18*4.75^3 / 6$

= 106.1 KN- M

FOS = 0.9*2862/106.1

FOS = 24 > 1.4 (Safe)

Step : Stability against Sliding

FOS against Sliding = ∑Fr / ∑Fo

Resisting Force ∑Fr = μ * ∑W

= 0.5*204.8

=102.4 KN

Sliding Force ∑Fo = ½ * ka*ϒ*H^2

=1/2 * 0.33* 18* 5.22^2

= 80.30 KN

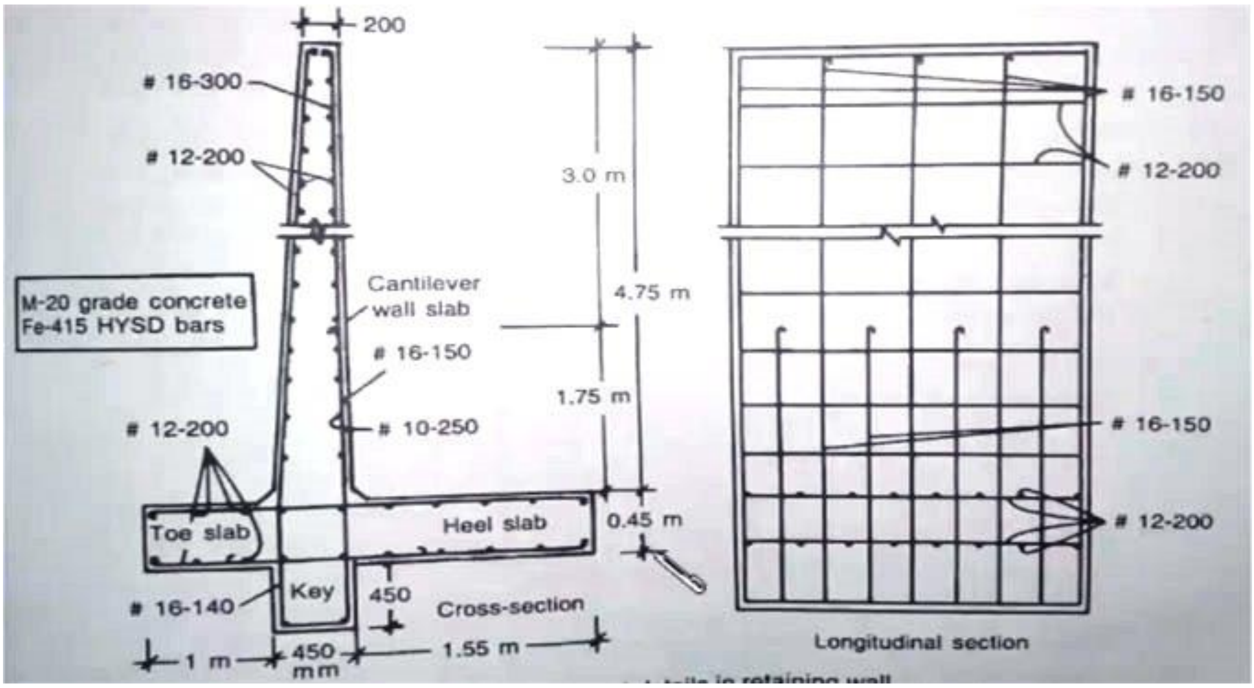
FOS = 102.4/ 80.30 = 1.27 < 1.4

Therefore Unsafe

Therefore, Shear Key has to be provided

Provide width of shear key = 450 mm

Depth of Shear Key = 450 mm



Problem (1) Design a counterfort retaining wall to suit the following data.

Height of wall above ground level 6m

The safe Bearing capacity of soil at site is 160KN / $m^{2.}$

Angle of internal friction 30°

Spacing of counterfort 3 m C/C

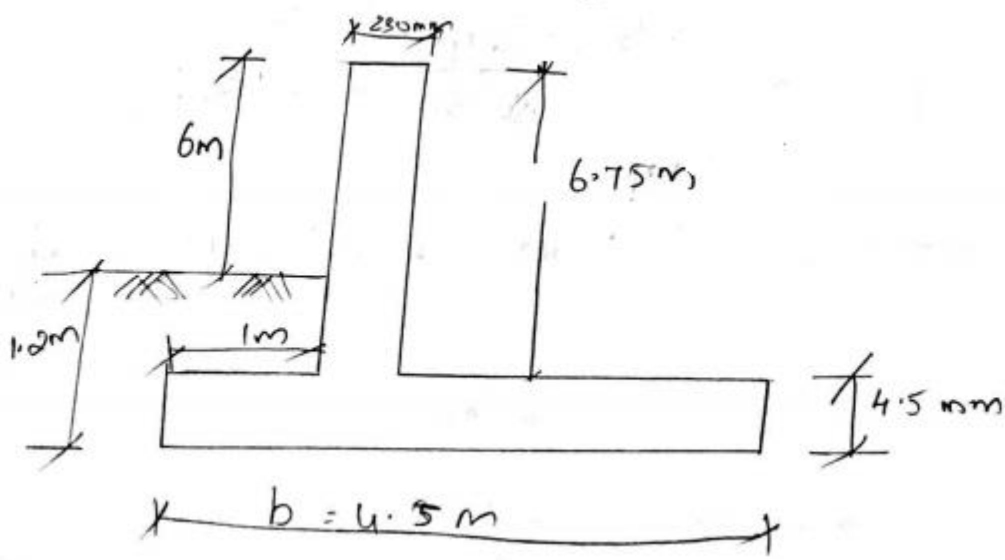
Density of the soil is 16 KN/M^2

Adopt M 20 grade concrete and Fe 415 steel.

Step 1 Given Data

Ht of wall above GL	= 6 m
Unit weight of earth (ϒ)	= 16KN / m^3
Angle of friction (ϕ)	= 30°
SBC	= 160 KN / m^2
Spacing of counterfort (l)	= 3 m
f_{ck}	= 20 N/mm^2
f_y	= 415 N/mm^2

Step 2: Dimension of Retaining Wall



Minimum depth of foundation = SBC/ϒ (1-sin ϕ/1+ sin ϕ)^2

= 160/16 (1-sin 30°/1+ sin 30°)^2

= 1.1 m = 1.2 m

Overall height of wall, H = 6+1.2 = 7.2 m

Thickness of base slab = 2LH cm

$= 2*3*7.2 = 43.2$ cm

$= 432$ mm ≈ 450 mm

Provide width of stem as 220 mm

Height of stem , Hs $= 7.2 - 0.45$

$= 6.75$ m

Width of base slab, b $= 0.4$ H to 0.7 H

$= 0.4 * 7.2$ to $0.7 * 7.2$

$= 4.3$ m to 5.04 m

Provide base slab width $= 4.5$ m

Width of toe projection $= 1/4* b = 1/4*4.5$

$= 1.1$ m ≈ 1 m

Width of heel projection $= 4.5 - (1+0.22)$

$= 3.28$ m

Step 3 Design of stem or vertical slab or upright slab

Maximum working moment $= wl^2 /12$

Provide intensity at base, w $= \Upsilon H_s (1\text{-sin } \phi/1+ \sin \phi)$

$= 16 * 16.75 (1\text{-sin } 30^\circ/1+ \sin 30^\circ)$

W $= 36$ KN / M^2

M $= 36*13^2 / 12 = 27$ KN– M

Factored BM , Mu $= 1.5 * 27$

$= 40.58$ KN-M

For Fe 415 Mu, Limit $= 0.138$ $fckbd^2$

b $= 1$ m $= 1000$ mm

D $= 220$ mm

Assume an effective depth cover of 50 mm

$d = D - d' = 220 - 50 = 170$ mm

Mu, limit = $0.138 * 20 * 1000 * 170^2$

= 79.76 KN-M.

Mu < Mu Limit

Therefore , Section is under reinforced

Mu = 0.87fyAst d (1 – Ast fy / b d fck)

$40.5 * 10^6$ = 0.87*415*Ast *170 (1 – (Ast *415/1000*170*20))

Ast = $723 mm^2$

Ast min for Fe 415 = 0.12% Cr.A

= 0.12/100 *1000*222

= $264\ mm^2$

Provide 12 mm dia bar

Spacing of 12 mm dia bar = $1000 * (\pi * 12^2/4) / 723$

Provide 12 mm dia bar @ 150 mm c/c

Distribution reinforcement

Ast Min = $264 mm^2$

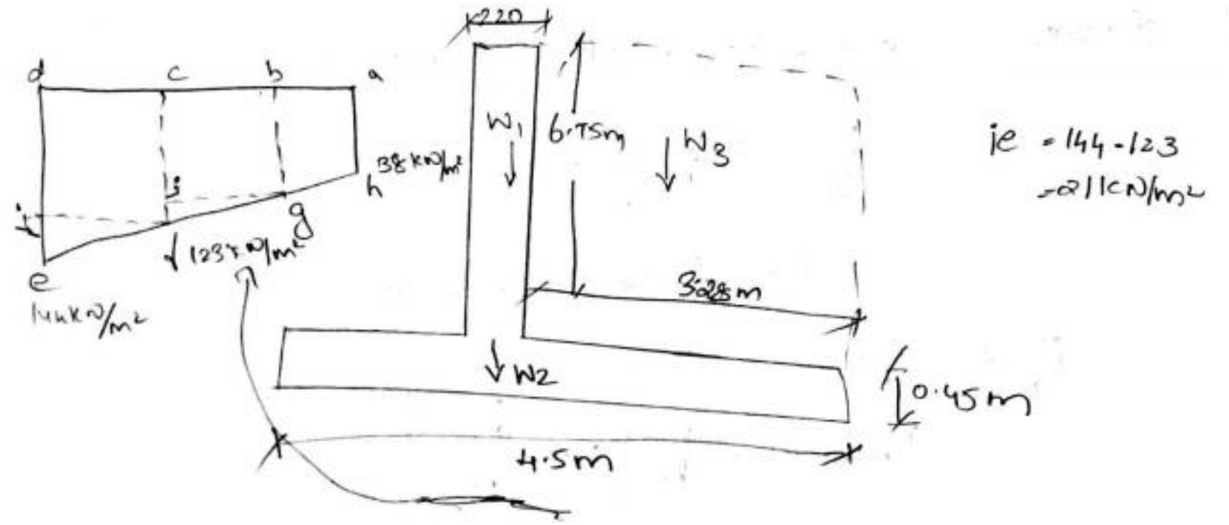
Provide 10mmdia bar

Spacing of 10 mm dia bar = $1000 * (\pi * 10^2/4) / 264$

Provide 10 mm dia bar @ 290 mm c/c

Step 4 : Stability Calculation

Stability calculation for 1 m run of wall



Loads	Magnitude of load (KN)	Distance from 'a' m	Moment (KN-m)
W1 – wt of stem = 0.2 * 6.75 * 25	37.13	3.28+0.22/2 = 3.39	25.87
Wt of Base slab W2 = 4.5*0.45*25	50.63	4.5/2 = 2.25	113.9
W3 – wt. of retained earth = 3.28*6.75*16	354.24	3.28/2 = 1.64	580.95
Moment due to earth pressure at base ($Ka*\Upsilon*H^3/6$) = $0.33*16*7.2^3/6$	$\sum$ w = 442		328.4

$\sum M = 114.912$

Distance of point of application of resultant from 'a', $Z = \sum M / \sum W$

$= 1149.12/ 442$

$=2.6$ m

Eccentricity, e = Z – b/2

= 2.6 – 4.5/2 = 0.35

b/6 = 4.5/6 = 0.75

e < b/6 (i.e.,) 0.35 < 0.75

P_{max}= ∑ w / b (1+ 6e/b)

= 442/4.5 (1+ 6*0.35/4.5)

= 144KN/m^2 at toe

P_{min}= ∑ w / b (1- 6e/b)

= 442/4.5 (1- 6*0.35/0.45)

= 52.38KN/m^2 at heel

0 – 144

1

3 – 52.38

@ 1 = 144 – (144 - 52.38(1-0) / (4.5-0)

= 123 KN/m^2

Step 5: Design of foot slab

Loads	**Magnitude of load (KN)**	**Distance from 'a' m**	**Moment (KN-m)**
upward Force edif = 1*123	123	½ = 0.5	61.5
fie = ½*1*21	10.5	2/3 * 1 = 0.67	7
Downward forces Self wt of toe slab = 1 *0.45*25	11.25	½ = 0.5	5.63
Self wt of earth			

above toe slab =1*0.7*16	12	½ = 0.5	6

upward moment = 61.5 + 7 = 68.5 KN-M

Downward Moment = 5.63 + 6 = 11.63KN- M

68.5 – 11.63 = 56.87KN- M

Factored BM, Mu = 1 .5 * 56.87

= 85.30KN- M

For Fe 415 Mu, Limit = 0.138 $f_{ck}bd^2$

b = 1 m = 1000 mm

D = 450 mm

Assume an effective depth cover of 50 mm

d = D – eff. Cover = 450 – 50 = 400 mm

Mu, limit = 0.138 * 20 * 1000 * 400^2

= 441.6 KN-M.

Mu < Mu limit

Therefore, Section is underreinforced

Mu = 0.87fyAst d (1 – Astfy / b d fck)

85.30 *10^6 = 0.87*415*Ast *450 (1 – (Ast *415/1000*400*20))

Ast = 610 mm^2

Ast min for Fe 415 = 0.12% Cr.A

= 540 mm^2(from step 3)

Provide 12 mm dia bar

Spacing of 12 mm dia bar = 1000 *(π * 12^2/4) /610

= 185 mm

Provide 12 mm dia bar @ 180 mm c/c

Distribution reinforcement = Ast min = 540 mm^2

Spacing of 10 mm dia bar = 1000 *(π * 10^2/4) / 540

Provide 10 mm dia bar @ 145 mm c/c

Step 6 : Design of Heel slab

Calculation for 1m wide strip of heel slab near heel end ‘a’

Downward Pressure:

Pressure due to self wt of heel slab = 1*0.45*25

$=11.23$ KN/M^2

Pressure due to earth above heel slab = 1*6.75*16

=108KN/M^2

Downward pressure = 11.23 + 108 = 119.25 KN- M^2

Upward Pressure = 1 * 52.38 = 52.38 KN- M^2

Net Pressure = 119.25 – 52.38 = 66.87KN- M^2

Max Working Moment (M) = wl^2/12

= 66.87 * 1.3^2/12 = 50.15 KN- M^2

Factored BM, Mu = 1 .5 * 5.615

= 72.23KN- M

For Fe 415 Mu, Limit = 0.138 fckbd^2

b = 1 m = 1000 mm

D = 450 mm

Assume an effective depth cover of 50 mm

d = D – eff. Cover = 450 – 50 = 400 mm

Mu, limit = 0.138 * 20 * 1000 * 400^2

= 441.6 KN-M.

Mu < Mu limit

Therefore, Section is underreinforced

Mu = 0.87fyAst d (1 – Astfy / b d fck)

75.23 *10^6 = 0.87*415*Ast *450 (1 – (Ast *415/1000*450*20))

Ast =536mm^2

Ast min for Fe 415 = 0.12% Cr.A

= 536 mm^2

Provide 12 mm dia bar

Spacing of 16mmdia bar = 1000 *(π * 12^2/4) /536

= 211 mm

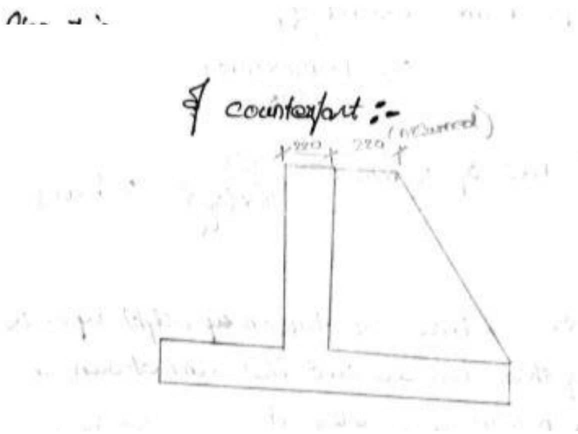
Provide 12 mm dia bar @ 180 mm c/c

Distribution reinforcement = Ast min = 540 mm^2

Spacing of 10 mm dia bar = 1000 *(π * 10^2/4) / 540

Provide 10 mm dia bar @ 145 mm c/c

Step 7: Design of Counterfort



Top width of counterfort = 220 + 220

(b) = 440 mm

Bottom width of counterfort (d) = 440*10 = 4400 mm

Max BM in Counterfort =[kaΥh^3/6]*l

Ka = $1-\sin\phi/1+\sin\phi$

= $1-\sin 30^\circ/1+\sin 30^\circ$

Ka = 0.33

= $[(0.33*16*6.75^{2)} / 6] * 3 = 812$ KN-M

Factored BM , Mu = 1.5 * 812

= 1218KNm

Mu = 0.87fyAst d (1 – Ast*fy / b d fck)

$1218 * 10^6$ = 0.87*415*Ast *4400 (1 – (Ast *415/440*4400*20))

Ast = $773 mm^2$

Ast Min for beam = 0.85 bd / fy

= 0.87*440*4400/415 = $3965 \ mm^2$

No. of bars of 32mmDia = 3965 / ($\pi * 32^2/4$)

= 5 bars

Out of 5 bars , 2 bars are taken right upto the top the remaining bars are curtailed in between at certain depth from bottom of counterfort.

Step 8: Connection between stem and counterfort

Provide 8 mm ϕ horizontal links to connect stem and counterfort at spacing of 300 mm c/c.

Step 9: Connection between Counterfort and Heel slab

Provide 10 mm ϕ vertical links to connect counterfort and heel slab at spacing of 300 mm c/c.

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SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

UNIT – V – WATER TANKS – SCI1308

UNIT – V – WATER TANKS

Types of water tank, design of water tanks using elastic method resting on ground, underground, rectangular, overhead circular and rectangular with staging.

(IS 3370 PART IV, IS 456:2000)

Introduction

A reinforced concrete is a very useful structure which is meant for storage of water for swimming and other purposes. Water tanks are used in uncracked theory where concrete is not allowed to crack on tension side which means tensile stress are kept within permissible tensile stress in concrete.

Types of tanks

WATER TANK

BASED ON PLACEMENT OF TANK

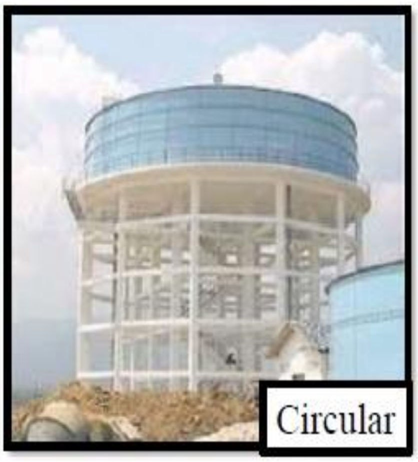
1. RESTING ON GROUND
2. UNDER GROUND
3. ELEVATED

BASED ON SHAPE OF TANK

1. CIRCULAR
2. RECTANGULAR
3. SPHERICAL
4. INTZ
5. CONICAL BOTTOM
6. DOOMED BOTTOM

Types of tanks based on shape

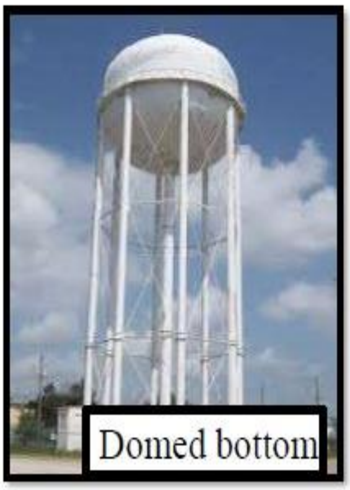
Rectangular



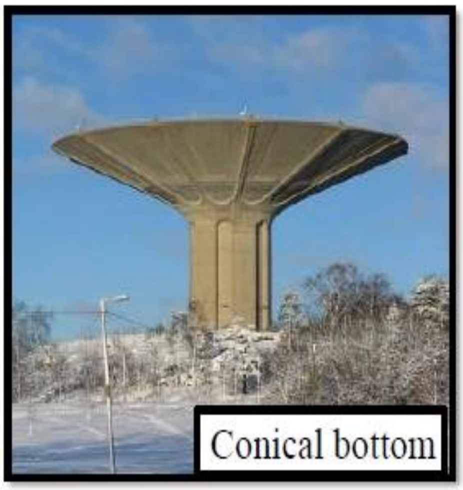
Circular

Intze

Spherical bottom



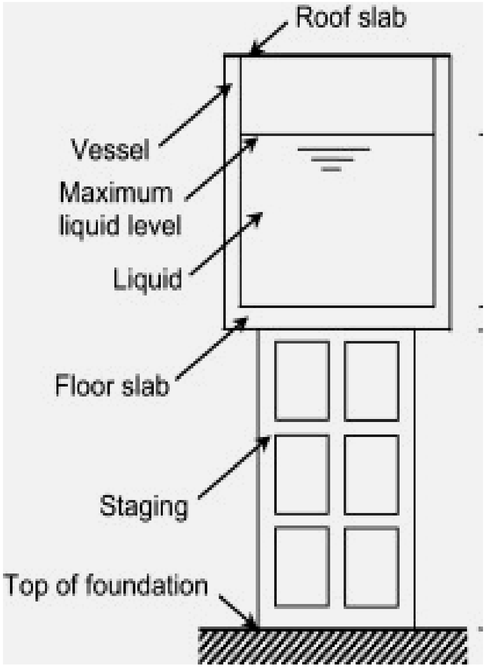
Domed bottom



Conical bottom

Rectangular Tank	Circular Tank	Intze Tank	Prestressed Tanks
For smaller capacities rectangular tanks are adopted	For bigger capacities circular tanks are recommended	Intze tank is constructed to reduce the project cost because lower dome in this construction resists horizontal thrust.	RCC tanks are cheaper only for smaller capacities up to 10-12 lakhs liters. For bigger tanks, prestressing is the superior choice resulting in a saving of up to 20%.

STAGING - It is required for tanks above ground level. It consists of column, braces and foundation. It is designed for gravity load due to water and self weight, wind load and earthquake load.



Stresses in concrete

Grade of Concrete	**Permissible Stress in N/mm²**		**Shear stress in N/mm²**
	Direct tension	**Tension due to bending**	
M_{15}	**1.1**	**1.5**	**1.5**
M_{20}	**1.2**	**1.7**	**1.7**
M_{25}	**1.3**	**1.8**	**1.9**
M_{30}	**1.5**	**2.0**	**2.2**
M_{35}	**1.6**	**2.2**	**2.5**
M_{40}	**1.7**	**2.4**	**2.7**

Permissible Stress

Sl. No.	TYPES OF STRESS	PERMISSIBLE STRESS in N/mm²	
		Fe250	Fe415
1	Tensile stress in member under direct tension	115	150
2	Tensile stress in member in bending a)on liquid retaining face of member b)on face away from liquid for member less than	115	150

	225mm thick c)on face away from liquid for member greater than 225mm thick	115 125	150 190

Minimum area of steel =0.3% gross area

Cover = 25mm

ELEMENTS IN WATER TANK DESIGN:

1. Roof slab
2. Wall
3. Base slab

NOTE: walls and base slab comes under JOINTS which are of two types – RIGID and FLEXIBLE

THICKNESS OF WALL - Should not be less than

- ❖ 150 mm
- ❖ 30mm per metre depth +50mm
- ❖ Thickness required limiting the tensile stress in concrete to 1.3N/mm^2 (M_{20})

Circular tanks - Circular tanks on ground may be designed either with flexible connection of wall with base or with the rigid connection of wall with base in flexible connection the walls are not monolithic with base whereas in rigid connection walls are monolithic with base.

Circular tank with flexible joint between wall and base

The wall of such a tank will be designed as vertical cylinder subjected to water pressure. The intensity of water pressure at any depth h, ***p = wh*** units per unit area. The corresponding hoop tension per meter height,

T= (Wh*D)/2

Where ,

W= specific wt of water

h=depth

D=diameter of tank

Problem (1) Design a circular tank resting on firm ground to the following particulars diameter of tank 3.5m, depth of water 3m, the wall and the base slab are not monolithic with each other. Specific weight of water is 9810 $N/m^{3.}$ Use M_{25} grade concrete and Fe 415 steel bars

Step1: Given data

Diameter of tank=3.5m

Depth of water=3m

Specific wt of water =9810N/m^3

M_{25}

Step2: Thickness of wall

Should not be less than 150mm

30mm per meter depth + 50mm = (30*3)+50 =140mm

Provide a thickness of 150mm

Step3: Reinforcements

Consider the bottom 1m height of wall

Pressure intensity corresponding to centre of bottom 1m height of wall

P = Wh

=9810*2.5

=24525 N/m^2

Hoop tension /m, T =PD/2

= (2425*3.5)/2

= 42919 N

Steel required for 1m height (A_{st}) = T/(safe stress in steel)

For Fe415 steel under direct tension, safe stress in steel = 150N/mm^2

A_{st} = 42919/150

= 237mm^2

Min steel required (A_{st} min) = 0.3% gross area (gross area=b*T

= (0.3/100) * 1000* 150

=450mm^2

Provide 10mm dia bars;

Spacing of 10mm dia bars = (1000*3.14*(10^2/4))/450

=174mm

Provide 10mm bars @ 170 mm c/c spacing

Vertical distribution steel = 0.3 % gross

=450mm^2

Step4: Base slab

Provide 150mm thick slab with top and bottom mesh with 10mm dia bars @250 mm c/c

Circular tanks with a wall retained at base (rigid) condition: in this case the wall will resist water pressure partly hoop action and partly by cantilever action at a certain height from the bottom there will be cantilever action and at a higher level hoop action

Methods to analyse

1. Dr. Reissners method
2. Carpenter's simplification of Dr. Reissners method
3. IS code

IS CODE

$H^2/(D*t)$

Hoop tension per metre height = coefficient * WHD/2 N

BM per metre run = coefficient * WH^3 N.m

Shear force at base of wall = coefficient * WH^2 N

Problem (2) Design a circular tank 12 m diameter of 4m height. The tank rest on firm ground and the walls of the tanks are retained (rigid) use M_{20} grade concrete and Fe 415 steel?

Solution:

Step1: given data

Diameter of tank =12m

Height of tank = 4m

Walls are retained at base

Step2: Thickness of wall

Should not be less than 150mm

30mm per metre depth +50mm = (30*4)+50 =170mm provide

Step3: reinforcement

Hoop tension (refer under 0.6 H)

$H^2/(D*t)$ $=4^2/(2*0.17)$

= 7.81

Coefficient under 0.6H

6	0.514
7.8	?
8	0.575

Interpolate:

(0.514-(0.514-0.575)*(7.8-6)) / (8-6)

Coefficient for 7.8 = 0.57

Max hoop tension = coefficient*(W*H*D)/2

= (0.57*9810*4*12)/2

= $134.2*10^3$ N

A_{st} = T/ safe stress in steel

$=134.2* 10^3/150$

= $394.6mm^2$

Ast min =0.3% gross area

= (0.3/100)*1000*170

= $510mm^2$

Provide 10mm dia bars

Spacing of 10mm dia bars = $(1000* 3.14 * (10^2/4))$ /894.6

=87.99mm

Provide 10mm dia bars @100 mm c/c

Cantilever action (BM) at 1H:

$H^2/(D*t)$	1H
6	-0.0187
7.8	?
8	-0.0146

Interpolate:

(- 0.0187-(-0.0187-(-0.0146))*(7.8-6) / (8-6) = -0.015

Maximum cantilever moment = Coefficient $*WH^3$

= $-0.015* 98109*4^3$

=9417.6 N.m

A_{st} = BM/ (safe stress * 0.85d) (d=eff. Cover of 30mm provide)

= $9417.6 *10^3/ (150*0.85*140)$

= $527.6mm^2$

A_{st} min = 0.3/100 * 1000* 170

=$510mm^2$

Provide 10mm dia bars

Spacing = $(1000*3.14*(10^2/4))/527.6$

=148mm c/c

Vertical steel distribution = 0.3% gross are

= $510mm^2$

Step 4: Base slab

Provide 150mm thick slab with top and bottom mesh with 10mm dia bars @ 250mm c/c.

RECTANGULAR WATER TANK RESTING ON GROUND

Rectangular tanks are useful for smaller capacities and for large capacity circular water tank are preferred .let centre line dimension of the tank be (L*l) where,

L= long plan dimension

L=short plan dimension

i. If (L/l <2) the wall is designed as continuous horizontal slab subjected to a water pressure ‘Wh ‘per unit area.
ii. If (L/l >=2) the long walls are considered as vertical continuous cantilevering for the whole pipe from base. The short walls are considered as spanning between long wall at end taken as fixed.

PROBLEM 1 A reinforced concrete tank is 6m*3m with a max depth of 2.5m of water. The tank rest on ground 150m *150mm. splays are provided at the junction of walls and base slab. Design a tank using M_{20} grade concrete and Fe 250 steel.

Solution:

Step1: given data

L=6m

L=3m

Depth of water =2.5m

150mm*150mm splays are provided

M_{20} fe415, L/l =6/3=2

Since the ratio is equal to 2, the long walls will be designed as vertical cantilever and short wall as spanning between the long walls

Step2: design of long wall

They are designed as vertical continuous since splays are provided effective height

Effective ht =2.5-0.15=2.35m

Max BM per metre width of long wall **= $(W*H^3)/6$**

Where,

W= specific wt

$WH^3/6$ = $(9810*2.35^3)/6$

= 21219 N.m

This BM produces tension near water face

Moment of resistance (M.R)= Qbd^2

In code book

M_{20}**:** Fe 250; Q=1.33 &

M_{20}**:** FE415; Q=1.16

Equate BM to M.R

$21219*10^3$ = $1.33 *1000d^2$

D = 126mm (130mm)

Provide an effective cover of 30mm

Overall thickness of wall = 130+30 =160mm

A_{st} =BM/ (safe stress *0.85*d)

= $(21218* 10^3)/ (115*0.35*130)$

=$1670mm^2$

A_{st} min = 0.3% GA

= (0.3/100)*1000*160

=480mm^2

Provide 12mm dia bars

Spacing of 12mm dia bars = (1000*3.14*(12^2/4))/1670

=68mm

Provide 12mm dia bars @ 100 mm c/c

Pull in long wall for bottom 1m ht = (WHD)/2

= (9810*(2.35-1)*3)/2

=19865N

A_{st} = T/(safe stress)

= 19865/115

=172mm^2 (safe stress =115N/mm^2)

A_{st} min =480 mm^2

Spacing of 12mm dia bar = (1000*3.14*(12^2/4))/480

=235mm

Provide 12mm dia bars @ 100mm c/c.