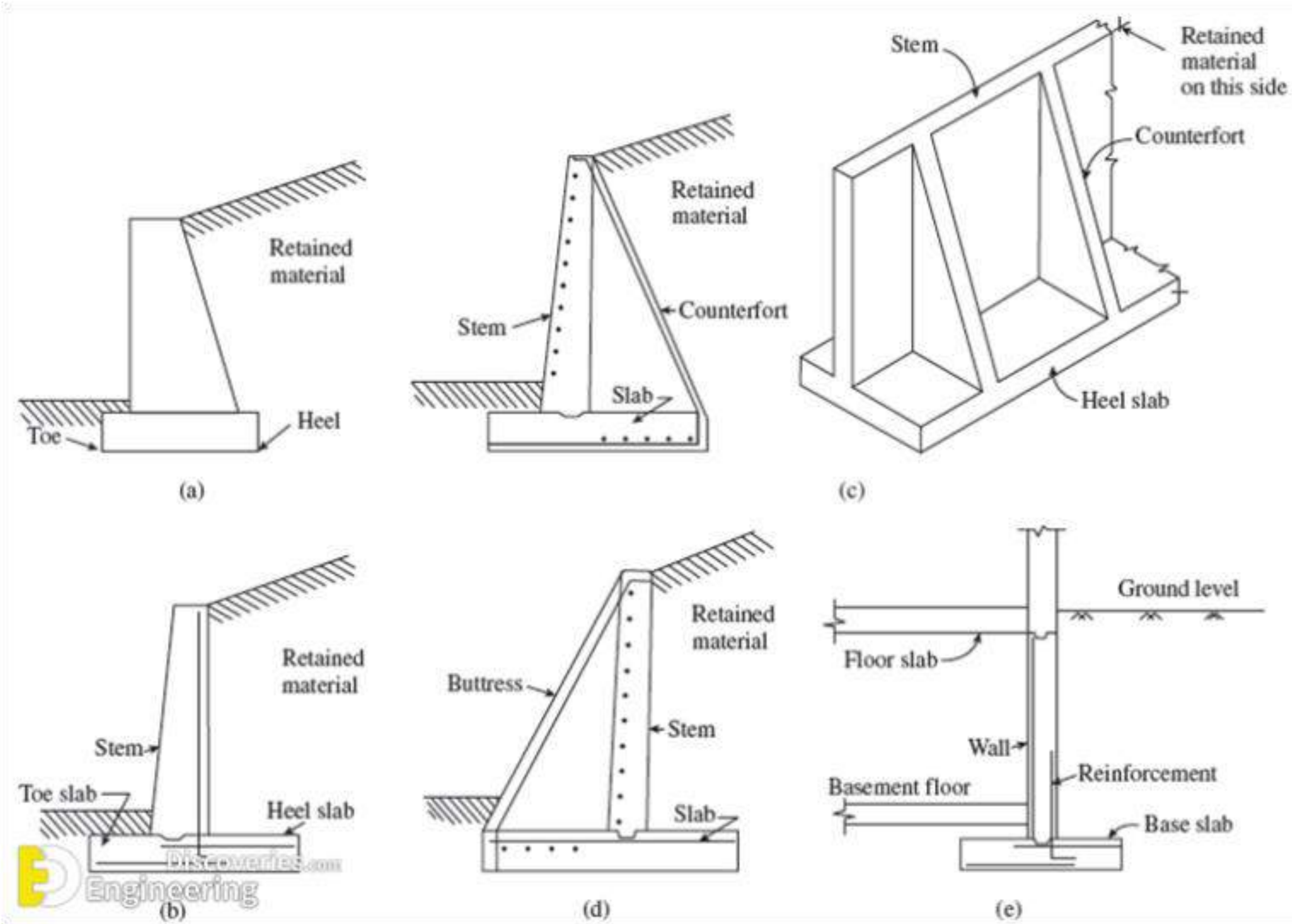
RETAINING WALL STRUCTURE

G.C.BEHERA



Retained material
Toe
Heel
(a)
Retained material
Stem
Counterfort
Slab
Stem
Retained material on this side
Counterfort
Heel slab
(c)
Retained material
Stem
Toe slab
Heel slab
(b)
Retained material
Buttress
Stem
Slab
(d)
Ground level
Floor slab
Wall
Reinforcement
Basement floor
Base slab
(e)
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TYPES OF RETAINING WALL STRUCTURES

- **USED FOR**

Construction of basement of buildings.

Abutment of bridge construction

Construction of Embankment

TYPES OF RETAINING WALL

1. Gravity retaining walls
2. Semigravity retaining walls
3. Cantilever retaining walls
4. Counterfort retaining walls

Gravity retaining walls

are constructed with plain concrete or stone masonry. The stability will be by their own weight and any soil resting on the masonry for stability. This type of construction is not economical for high walls.

semi gravity walls

When some reinforcement is added to gravity type to make it economical, it is known as semi gravity type.

Cantilever retaining walls;

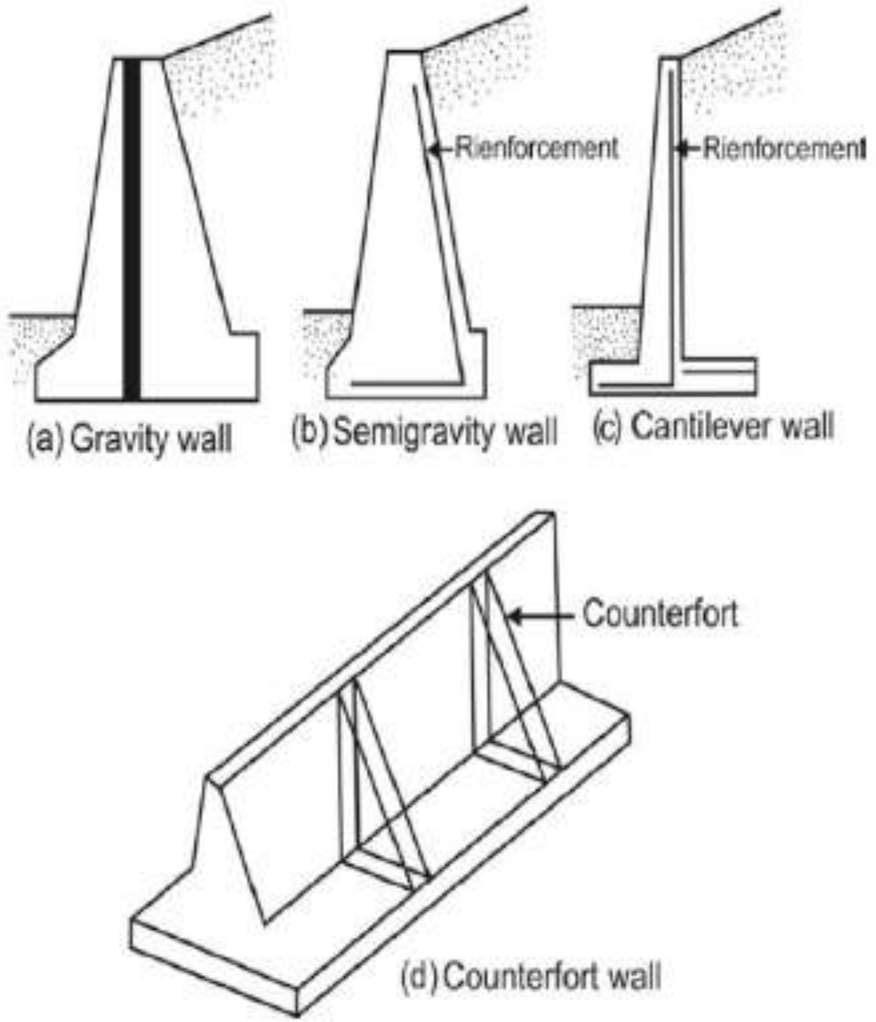
These are made by providing reinforcement which makes the stem thin. This is more economical for ht up to 8m.

Counterfort retaining walls:

Cantilever retaining walls provided with vertical concrete slabs at regular intervals(known as counterforts) are known as counterfort retaining walls. Counterforts are provided to minimize shear and the bending moments.

DESIGN PARAMETERS:

the unit weight, angle of friction, and cohesion-for the soil retained behind the wall , and the soil below the base slab.



DESIGN STEPS

These retaining wall structures should checked for stability. That includes checking for possible overturning, sliding, and bearing capacity failures.

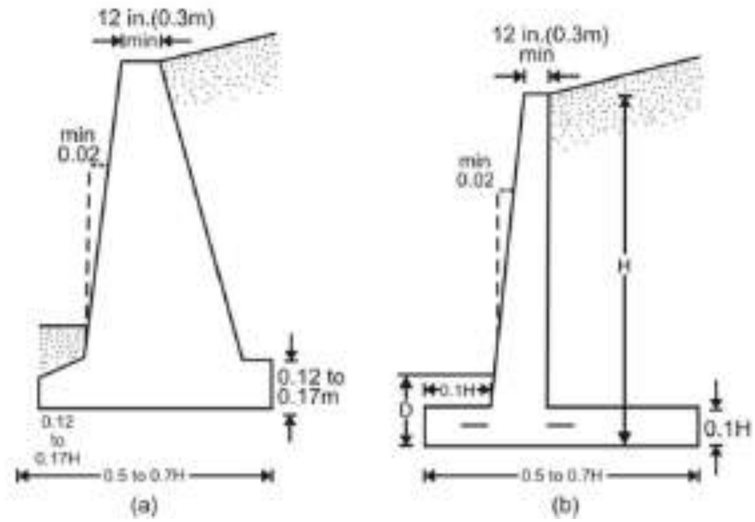
Second, each component of the structure is checked for *adequate strength, an*

When designing retaining walls, an engineer must assume some of the dimensions, called *proportioning, which allows the engineer to check trial sections for stability. If the stability checks yield undesirable results, the sections can be changed and rechecked.* ***Figure shows the general proportions of various retaining walls components that can be used for initial checks.*** *d the steel reinforcement of each component is determined.*

- Note that the top of the stem of any retaining wall should not be less than about 12 in.(≈0.3 m) for proper placement of concrete. The depth, *D, to the bottom of the base slab should be a minimum of 2 ft (≈0.6 m). However, the bottom of the base slab should be positioned below the seasonal frost line.*
- For counterfort retaining walls, the general proportion of the stem and the base slab is the same as for cantilever walls. However, the counterfort slabs may be about 12 in.(≈0.3 m) thick and spaced at center-to-center distances of 0.3HH to 0.7 HH.

- Active earth pressure tends to deflect the wall away from the backfill.
- active earth pressure for horizontal & cohesion less soil $K_a = \frac{1 - \sin\emptyset}{1 + \sin\emptyset}$
- active earth pressure for inclined back fill & cohesion less soil
- $$K_a = \left[\frac{\cos\theta - \sqrt{cos^2\theta - cos^2\varphi}}{\cos\theta + \sqrt{cos^2\theta - cos^2\varphi}}\right]\cos\theta$$
- Passive earth pressure for horizontal & cohesion less soil $K_p = \frac{1 + \sin\emptyset}{1 - \sin\emptyset}$
- Passive earth pressure for inclined back fill & cohesion less soil

$$K_p = \left[\frac{\cos\theta + \sqrt{cos^2\theta - cos^2\varphi}}{\cos\theta - \sqrt{cos^2\theta - cos^2\varphi}}\right]\cos\theta$$

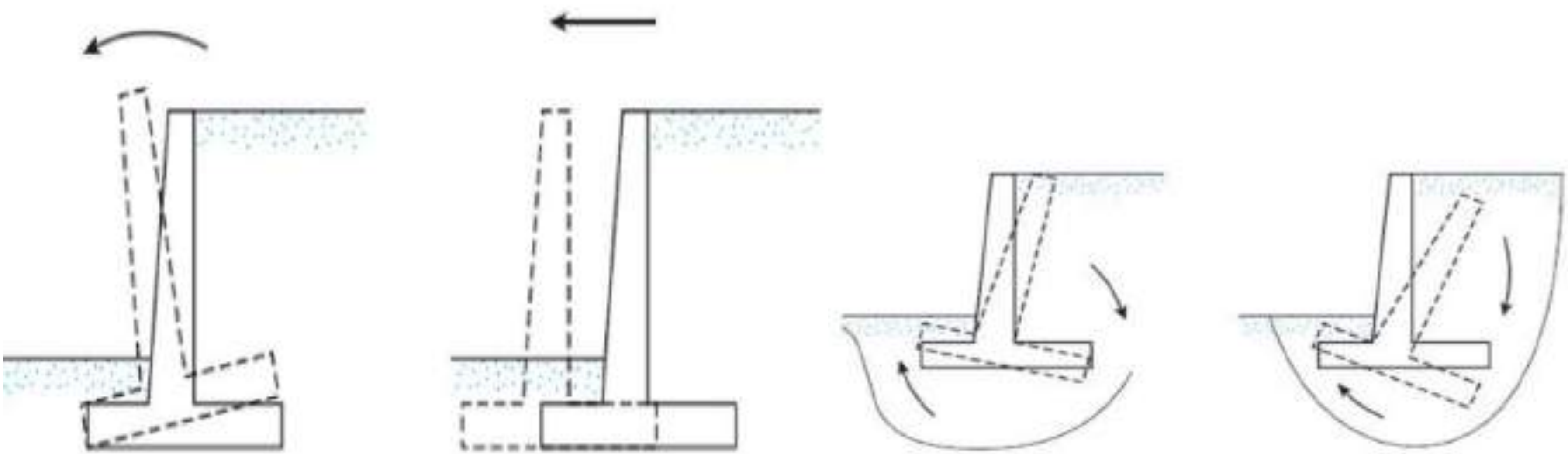


D min0.6m

φ is the angle of internal friction or angle of repose

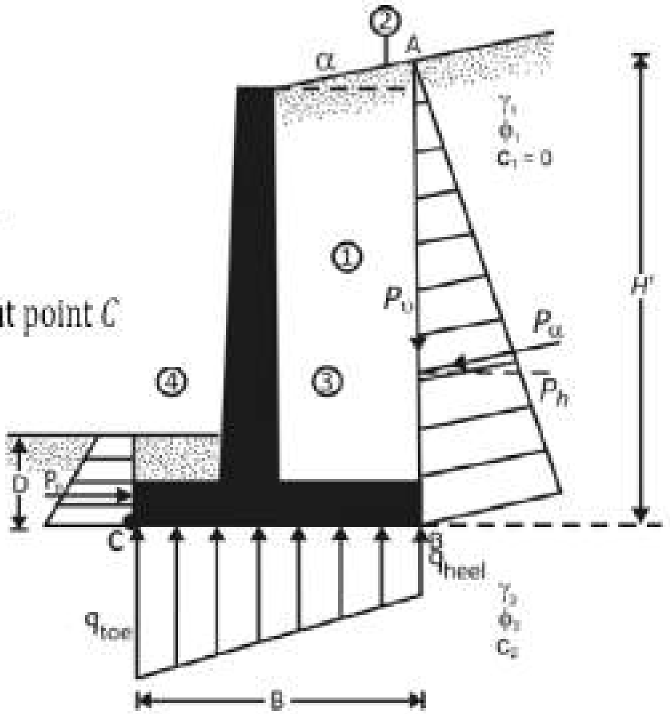
θ is the angle of backfill

- 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*



- The factor of safety against overturning about the toe-that is, about point *C*

$$FS_{(\text{overturning})} = \frac{\Sigma M_R}{\Sigma M_O}$$



Where

ΣM_O = sum of the moments of forces tending to overturn about point C

ΣM_R = sum of the moments of forces tending to resist overturning about point C

The overturning moment is

$$\Sigma M_O = P_h \left(\frac{H'}{3}\right)$$

Where $P_h = P_a \cos \alpha$

For calculation of the resisting moment, $\Sigma M R$

Note that the force Pv=also contributes to the resisting moment. Pv is the vertical component of the active force Pa

The moment of the force P_v about C is

$M_v = P_v B = P_a \sin \alpha\, B$

Where

B = width of the base slab

Section (1)	Area (2)	Weight/unit length of wall (3)	Moment arm measured from C (4)	Moment about C (5)
1	A_1	$W_1 = \gamma_1 \times A_1$	X_1	M_1
2	A_2	$W_2 = \gamma_2 \times A_2$	X_2	M_2
3	A_3	$W_3 = \gamma_c \times A_3$	X_3	M_3
4	A_4	$W_4 = \gamma_c \times A_4$	X_4	M_4
5	A_5	$W_5 = \gamma_c \times A_5$	X_5	M_5
6	A_6	$W_6 = \gamma_c \times A_6$	X_6	M_6
		P_v	B	M_v
		ΣV		ΣM_R

Note: γ_1 = unit weight of backfill

γ_2 = unit weight of concrete

Once ΣM_R is known, the factor of safety can be calculated as

$$FS_{(\text{overturning})} = \frac{M_1+M_2+M_3+M_4+M_5+M_6}{P_a \cos \alpha (H'/3) - M_v}$$

STABILITY REQUIREMENT

- (1) OVERTURNING

A retaining wall is subjected to over turning moment under the action of lateral force developed due to lateral earth pressure, which try to overturn the wall about the toe end. The overturning moment (M_0) is given by

$M_0=P_{ah}*H/3=(1/2)(K_a\gamma H)*H*H/3=(K_a*\gamma H^3/6)$

The resisting moment (MR) is provided by weight of the backfill , surcharge and self weight of the retaining wall. If ΣW is the resultant vertical load made up of self weight of the retaining wall and weight of the backfill on the base slab, then resisting moment is

$$M_R = \Sigma W * \bar{x}$$

$\bar{x}$ is the position of the resultant vertical load (ΣW) from the toe end.

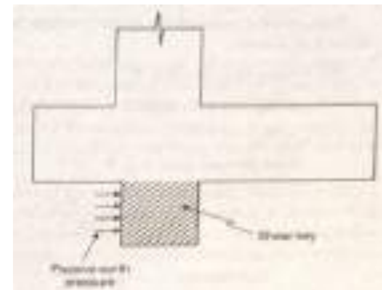
As per the Code- IS 456 : 2000 Clause 20.1, the stability of the retaining wall against overturning should be ensured that resisting moment should not be less than 1.4 times the maximum overturning moment. If the dead load provides restoring moment , then as per code 90% of the dead load should be taken into account.

$$f_{s1} = \frac{0.9M_r}{M_0} \qquad f_{s1} \geq 1.4 \qquad \frac{0.9(\sum W*\bar{x}}{\frac{K_a\gamma H^3}{6}} \geq 1.4$$

SLIDING

The lateral earth pressure tries to slide the retaining wall from the back fill. Frictional force between base slab and soil opposes this. If μ is the frictional coefficient betwwen concrete and soil , then frictional force of resistance

$$F_R = \mu \sum W$$



The lateral force causing sliding

$$F_s = P_{ah} = \frac{K_a \gamma H^2}{2}$$

Safety factor against sliding (fs2) is taken as 1.4 and only 0.9 times of Characteristic load should be taken.

$$f_{s2} = \frac{F_R}{F_s} = \frac{\mu \sum W}{P_{ah}}$$

$$\frac{0.9(\mu \sum W)}{P_{ah}} \geq 1.4$$

If the criteria is not satisfied , provide shear key.

PROPORTIONING OF CANTILEVER RETAINING WALL

Before the actual analysis, there is requirement for fixing of dimensions.

Depth of Dimension

1) Depth of Foundation $h_{min} = \frac{q_0}{\gamma}\left[\frac{1+sin\varphi}{1-sin\varphi}\right]^2$

Where H min is the depth below the ground level, q_0= Safe bearing capacity,

γ Is unit weight of soil and ϕ is the angle of repose.

2) Height of the retaining Wall (H)

$H=h+h_{min}$ where h is height of material to be retained.

3) Base width(b)

Width of the base slab can be determined by considering equlibrium of various forces at the base. Base width varies from 0.4 H to 0.6H.

4) Thickness of Base slab

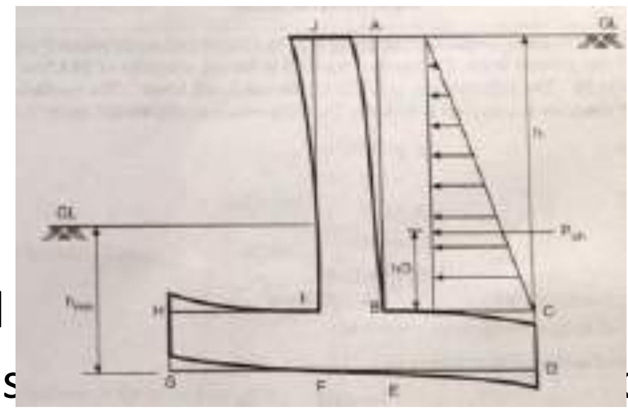
For the Preliminary analysis, thickness of the base slab can be taken H/10 to H/15. H is the total ht of the retaining wall. Min thickness not less than 300 mm. Thickness assumed should be checked from bending moment and shear force requirement.

- Thickness of Stem
- Thickness of the vertical stem is governed by moment criteria. It behaves like a cantilever. It is better to have trapezoidal section , 150 mm depth at topand at the base not less than 300 mm. Initially the stem may be assumed to be 8% to 10% of the total height of the retaining wall.
- Structural Behavior of Components of a Cantilever Retaining Wall:

1) STEM:

- It behaves like a cantilever.
- Maximum moment at the base B
- $M_0 = P_{ah} * h/3 = (1/2)(K_a \gamma h) * h * h/3 = (K_a * \gamma h^3/6)$

2) Heel Slab



The heel slab acts as a cantilever. It is subjected to upward soil pressure and downward press fill pressure generally more, so tension in top face BC.

3) Toe Slab:

This also behaves like a cantilever. Weight of the front fill is less, so bottom face is subjected tension. Reinforcement is to be provided along GF.

Problem: Design a cantilever retaining wall to retain the horizontal earthen embankment of a ht. 4 mt. above the ground level. Unit weight of backfill 18 kN/m^3, angle of repose 30^0, sbc 180 kN/m^3, coefficient of friction between concrete and soil 0.45. Use M20 concrete and Fe415 steel.

Solution: Coefficient of active earth pressure K_a

$$K_a = \frac{1 - \sin \emptyset}{1 + \sin \emptyset} = 1/3$$

Minimum Depth: $h_{min} = \frac{q_0}{\gamma}\left[\frac{1-sin\varphi}{1+sin\varphi}\right]^2 = \frac{180}{18}\lfloor 1/3 \rfloor^2 = 1.11\ m\ say\ 1.2\ m$

Total height of the wall= Depth of foundation + height of embankment

=1.2+4=5.2 m

Preliminary Dimension of the Retaining Wall:

1) Base width: b= 0.4H to 0.6H Take b=2.8 mt

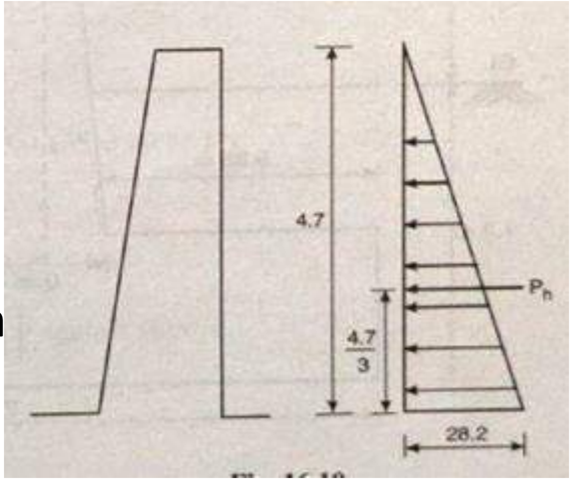
   Length of Toe= 0.3b to 0.4 b Take =850 mm

2) Thickness of base slab= Assuming H/10, Take 500 mm

3) Thickness of stem:

$$Moment\ at\ the\ base = \frac{K_a \gamma h^3}{6} = 103.83\ kNm$$

- Ultimate Moment M_u=1.5*103.83=155.74 kNm
- M_U=.138*fck*b*d^2
- d= SQRT[M_u/(0.138*fck*b)]=238 mm
- Assuming 60 mm cover =238+60=298 mm
- Take depth at base of stem 350 mm and 150 mm at the top.



Forces acting in Retaining Wall

TYPE OF FORCE	Magnitude of Force (kN)	Position of force from "O"	Bending Moment at Toe end "O" (kNm)
(1) P_{ah}=(½) K_a*γ*H^2	0.5*(1/3)*18*5.2*5.2=81.12	H/3=1.733	81.12*1.733=140.61 ΣM_0=140.61
(2) Restoring Force a) Wt of Back fill (W_1) b) Wt. of Stem i) Wt of rect. Portion(W_{21}) ii) Wt of Triangular portion(W_{22}) c) Wt of Base Slab (W3)	1.6*4.7*18=135.36 0.15*0.4.7*25=17.625 1.5*0.2*4.7*25=11.25 0.5*2.8*25=35	2.8-1.6/2=2.0 0.85+0.35-0.15/2=1.125 0.85+2/3*0.2=0.983 2.8/2=1.4	270.72 19.828 11.554 49
	ΣW=199.735		Σ351.1

Stability Check:

Overturning:$0.98M_R/M_0=0.9*351.10/140.61=2.2>1.4$ So OK.

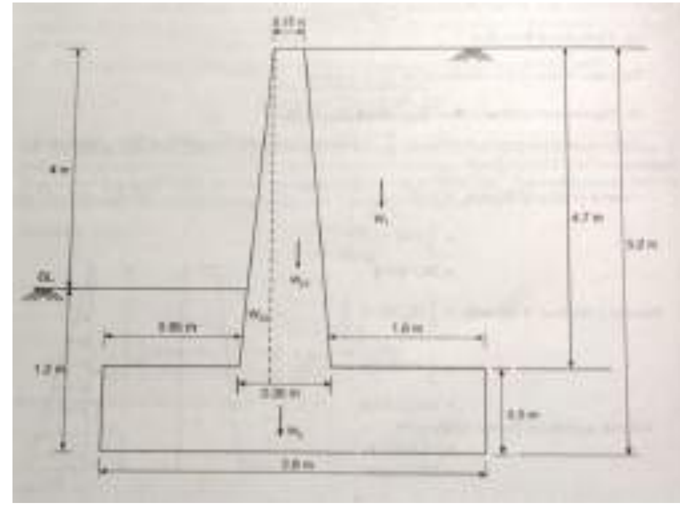
Stability Check: $0.9F_R/F_S \geq 1.4$

$F_R=\mu\Sigma W=0.45*199.735=89.88$ kN

$F_S=P_{ah}=81.12$ kN

$0.9*F_R/F_S=0.9*89.46/81.12=0.99<1.4$

Provide Shear key.



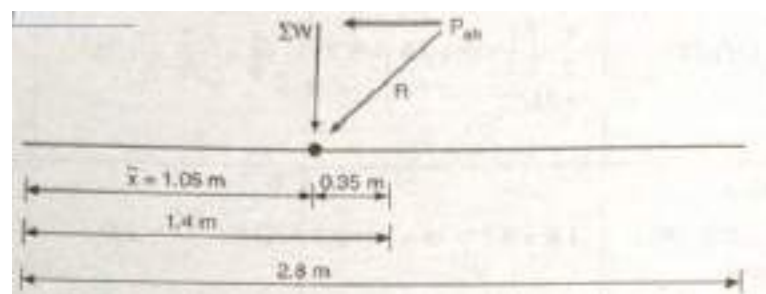
Base Pressure:

Resultant Moment at Toe end O=M_R-M_0 =351-140.61=210.49 kNm

The resultant of the vertical load=199.73 kN

This acts at a distance $\bar{x}$ from the toe end

$$\bar{x}=\frac{210.49}{199.73}=1.05\ m$$



Eccentricity=e=b/2- $\bar{x}$=1.4-1.05=0.35 m

Maximum eccentricity=b/6=2.8/6=0.466 m

Maximum Pressure at toe end=$p_{max} = \frac{\Sigma W}{b}\left[1+\frac{6e}{b}\right] = \frac{199.73}{2.8}\left[1+\frac{6*0.35e}{2.8}\right] = \frac{124.83kN}{m^2} < 180\,\frac{kN}{m^2}$

Minimum Pressure at heel end= $p_{min} = \frac{\Sigma W}{b}\left[1-\frac{6e}{b}\right] = \frac{199.73}{2.8}\left[1-\frac{6*0.35e}{2.8}\right] = \frac{17.83kN}{m^2} < 180\,\frac{kN}{m^2}$ and compressive

Design of Stem: D=350 mm, d=350-60=290 mm

Moment at the base=155.73 kNm

Area of Steel A_{st}

$M_u = 0.87 A_{st} f_y d[1 - \frac{f_y A_{sr}}{f_{ck} bd}]$

Putting the values, A_{st} required= 1693 mm2

Using 16 mm dia bars spacing 201*1000/1693=118 mm

Provide 16 mm dia bars @100 mm c/c.

Distribution steel @0.12%

A_{st}=0.12*1000(150+350)/(2*100)= 300 mm2

Using 8 mm dia bars spacing 167.5 mm, provide 150 mm c/c in inner face, in both the directions at outer face.

Check For Shear

The critical section for shear at a distance d effective from the base of the stem i.e, h=4.7-0.29=4.41

Shear force at this level=(1/2)*(1/3)*18*4.41*4.41=58.3 kN

V_u=58.3*1.5=87.52 kN

Normal shear stress= τ_{vu}=Vu/bd=87.52*1000/(1000*290)=0.3 N/mm^2

Pt=201*(1000/100)*1000/[1000*290]=0.69 %

τ_c=0.54 N/mm^2 >τ_{vu}

Curtailment of Reinforcement

As the beam behaves like a cantilever, reinforcement may be curtailed

For 16 mm dia bars ld= 0.87*f_y*ϕ/[4*τ_{bd}]=0.87*415*16/(4*1.2*1.6)=752 mm

Curtail the bar after 1000 mm from stem

Total depth at this portion=150+[200*3700/4700]=307 mm

Effective depth=307-60=247 mm

Moment at a depth 3.7 m from top=(1/6)*(1/3)18*3.7^3=50.7 kNm

M_u=1.5*50.7=76 kNm

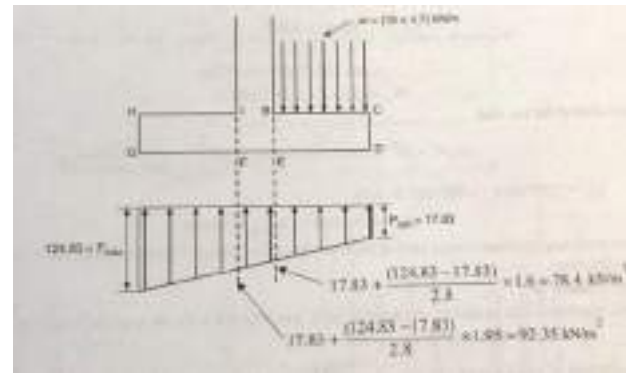
$$M_u = 0.87 A_{st} f_y d[1 - \frac{f_y A_{sr}}{f_{ck} bd}]$$

Steel required for this is 924 mm2.

Spacing of 16 mm dia bars=201*1000/924=217 mm c/c

Hence half of the bars can be curtailed after 12 * dia=12*16=192 mm or development length,Hence curtailment can be done 1.3 m from base or 3.4 from top.

Another curtailment can be done at 1.5 from top.
Factored Moment at this section=1.5*[18*1.5^3/(3*6)]=1.5*3.375=5.1 kNm
Overall depth at this section=150+200*3200/4700=286 mm
d= effective depth=286-60=226 mm
$A_{\text{st required}}$=$M_u = 0.87 A_{st} f_y d[1 - \frac{f_y A_{sr}}{f_{ck} b d}]$=65 mm^2 < A_{stmin}=300 mm2
Curtail another half of the bar at 1.5 mt from top, provide 16 mm dia bars 400 mm c/c.
Design of Heel Slab:
Weight of earth supported on heel=18*4.7=84.6 kN/m
Self weight of base slab=0.5*1.0*25=12.5 kN/m
Total load=97.1 kN/m



Maximum bending at B
=(97.1*1.6*1.6/2)-(17.83*1.6*1.6/2)-(1/2)*(78.4-17.83)*(1.6)*(1.6/3)=75.7 kNm
Mu=1.575.7=113.6 kNm
Dreq=SQRT[113.6*106/(2.76*1000)]=202 mm < 440 mm Hence OK.
Area of steel Ast= 741 mm^2
Spacing of 12 mm bars=113*1000/741=152 mm
Provide 12 mm dia bars 150 mm c/c.
Distribution steel=0.12*1000*500/100=600 mm^2
Provide 10 mm dia 100 mm c/c

Design of Toe Slab:

Neglecting the weight of the front fill above the toe slab,

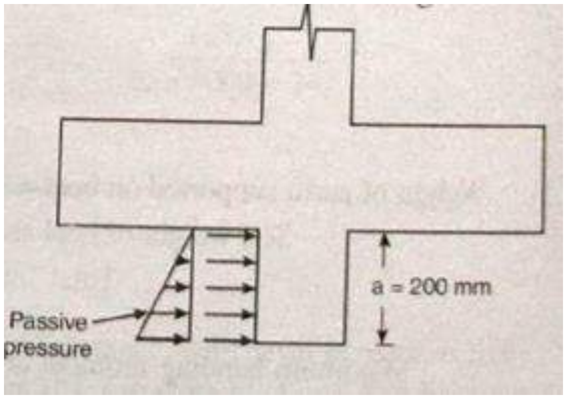
Maximum moment=[92.35*0.85*0.85/2]+(1/2)(124.83-93.35)*0.85*(2/3)*0.85=41.2 kNm

Mu=1.5*41.2=61.8 kNm

A_{st} required for this moment taking d=440 mm

A_{st}=396 mm^2 < [Astmin=600 mm^2]

Provide 8 mm dia 100 mmc/c for A_{stmin}.



Design of Shear key

Pressure at face of shear key=92.35 kN/m2

Coefficient of passive earth pressure=3

Let the depth of shear key=a

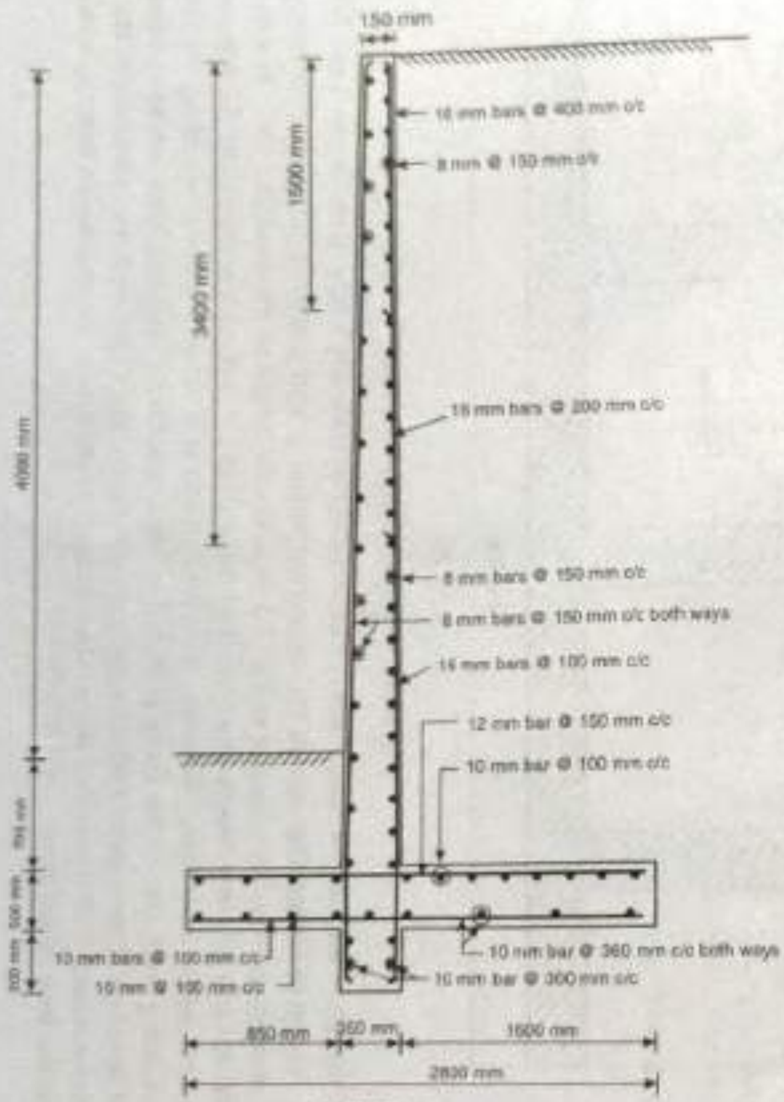
Resistance offered by shear key=3*92.35*a

Factor of safety against sliding along with shear key

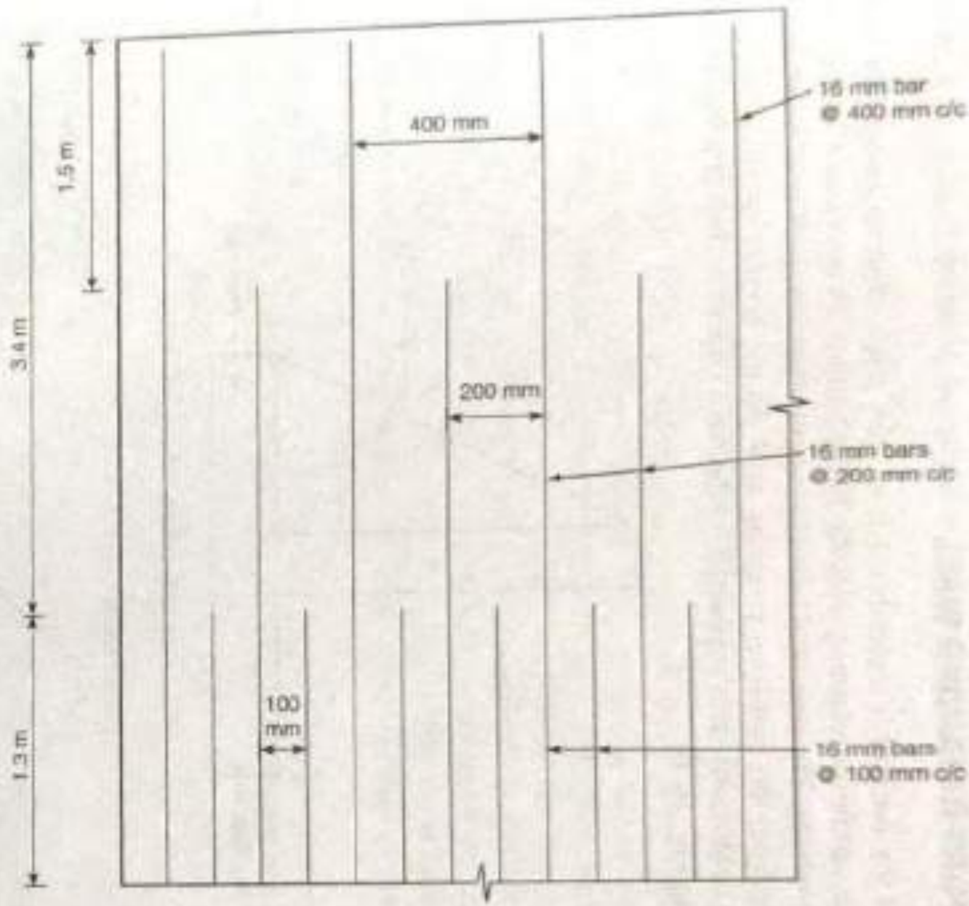
$$\frac{0.9*\mu\sum W+277.05a}{P_{ah}}=\frac{0.9*89.88+277.05a}{81.12}=1.14 \quad a=0.118\text{ m}$$

Provide shear key=200 mm X 200 mm.

Detailing of the reinforcement is given below.



(a) Section



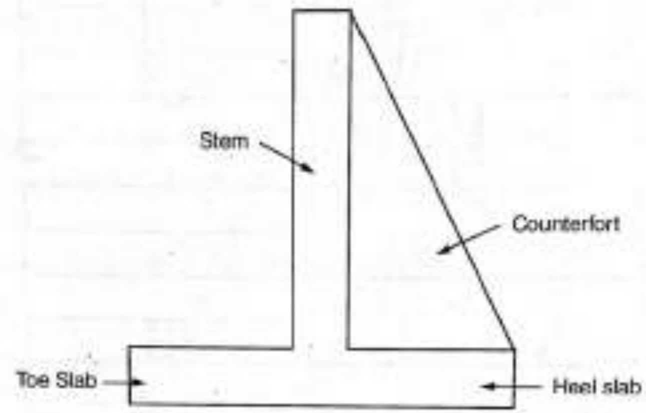
(b) L-Section of Stem

Fig. 16.15. Cantilever Retaining Wall

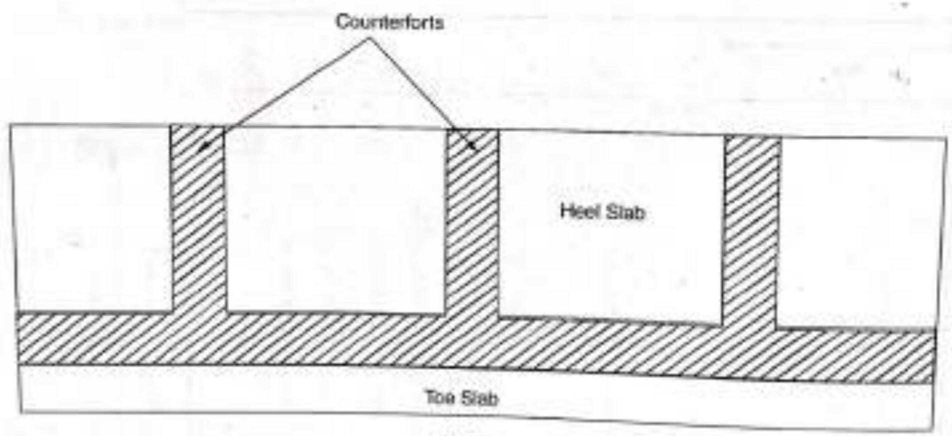
COUNTERFORT RETAINING WALL

Dr.G.C.Behera

When ht. Is more than 6 mt., Counterfort retaining walls are preferred. As ht. Increases, the BM also increases, so Counterfort retaining walls are preferred.
The structural behaviour of counterfort retaining wall is different from cantilever retaining wall. As the counter supports the stem and heel , they behave like continuous slabs.



(a) Elevation



(b) Plan

(1) Design of Stem: The stem of counterfort retaining wall acts as a continuous slab supported on counterforts which are spaced 3 to 3.5 m along the length of retaining wall. The stem is subjected to earth pressure which tries to deflect the wall away causing tension on the outer face and compression in inner face. Main reinforcement is put on the outer face along the retaining wall. Due to fixity provided by counterfort supports, some negative BM develops which cause tension in inner face near the counterforts. So, main reinforcement may be provided in inner face near counterforts. Maximum BM occurs at the base of the stem.

The load at the base per 1 m length w= $P_a*1*1=K_a*\gamma*H$ per mt. length.

Bending moment varies $wl^2/12$ at support and $wl^2/16$ at mid span.

(2) Design of Counterfort:

The counterforts are attached to stem and Hell slab.They act as T beams with varying cross section. Maximum depth near base . The earth pressure acting on stem tries to separate counterforts from stem. So, horizontal ties are required to hold stems with counterforts. The downward weight of backfill on heel try to separate heel from counterfort. Ties are also provided to connect heel and counterfort. Counterforts act like T beam supported on edges AB and BC, free on edge AC. Thickness of counterfort may be taken as thickness of the base slab.

Counterforts are designed for a maximum bending moment

$Mmax=Ka*\gamma*h^3*l/6$ where l is spacing of counterforts

(3)Design of Heel Slab

The heel slab behaves like stem. It is supported on three edges, counterforts, and stem and acted upon by downward backfill and upward soil pressure. If p is the net pressure, then p acts in downward direction, the maximum negative moment occurs at the counterforts $pl^2/12$ and positive moment $pl^2/16$ in the middle of heel slab.

(4) Design of Toe Slab:

Design of toe slab is same as cantilever retaining wall.

PROBLEM: Design a counterfort retaining wall for the following data with M20 concreteand Fe415 steel

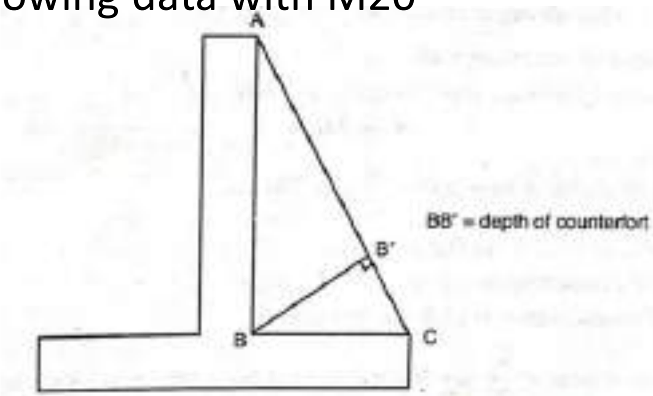
γ= 15 kN/m^3

φ=30^0

μ=0.6

h=4 mt

Q0=200 kN/m^3

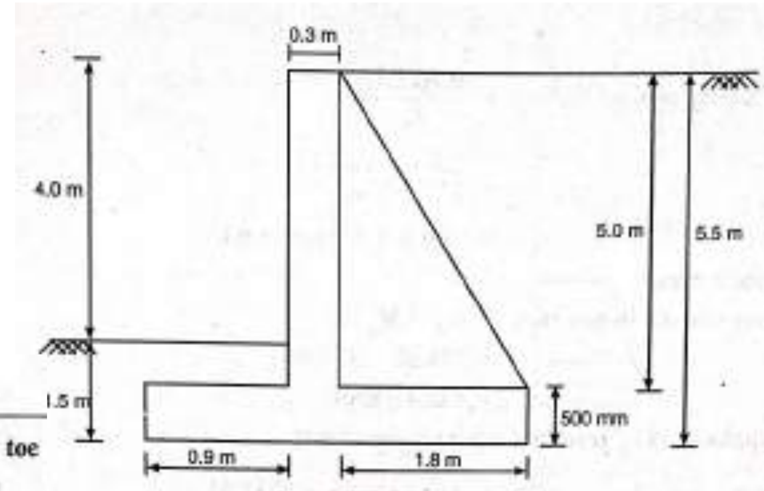


Minimum depth of foundation: $h_{min}= \frac{q_0}{\gamma}\left[\frac{1-sin\varphi}{1-sin\varphi}\right]^2$ =1.48 mt

$$h_{min}= \frac{q_0}{\gamma}\left[\frac{1-sin\varphi}{1+sin\varphi}\right]^2 = \frac{200}{15}[(1\text{-}sin30^0)/(1\text{+}sin30^0)]\text{\textasciicircum}2$$

Overall depth=4+1.5=5.5 m

Proportioning of Retaining wall: Width of base slab=0.6H; take b=3.0 m
Assuming thickness of base slab=H/12= say 500 mm
Toe projection=0.3b= say 0.9 m
Spacing of counterforts=3.0 m
Width of counterfort=0.05H say 300 mm
Thickness of stem = H/20 say 300 mm.



■ **Forces Acting on the Retaining Wall**

Force (Type)	Force (kN)	Distance from toe edge m	Moment about toe edge (kNm)
1. Overturning force $P_h = \frac{1}{2}(K_a \gamma H)H$	$\frac{1}{2}\times\left(\frac{1}{3}\times 15\times 5.5\right)5.5$ $= 75.625$	$\frac{H}{3}=\frac{5.5}{3}=1.833$	138.65
	$F_S = 75.625$ kN		$M_0 = 138.65$
2. Restoring forces			
(*i*) Weight of backfill (W_1)	$15 \times 5 \times 1.8 = 135$	$3.0 - \frac{1.8}{2} = 2.1$	283.5
(*ii*) Weight of stem (W_2)	$0.3 \times 5.0 \times 25 = 37.5$	$0.9 + \frac{0.3}{2} = 0.915$	34.31
(*iii*) Weight of base slab (W_3)	$0.5 \times 3 \times 25 = 37.5$	$\frac{3.0}{2} = 1.5$	56.25
	$\sum W = 210$ kN		$M_R = 374.06$ kNm

Stability Check:

(1) Overturning.

Factor of safety against overturning:=0.9MR/Mo=0.9*374.6/138.65=2.4> 1.4 so ok.

(2) Sliding: Safety factor against sliding:

0.9*$\mu\sum W/Fs$=[0.9*0.6*210]/[75.625]=1.49 > 1.4 so ok.

(3) Base Pressure Check: Net Moment at the edge =MR-MO=374.06-138.65=235.41 kNm

Resultant force distance $\bar{X} = Net\ Moment/\sum W$ =235.41/210=1.121 m

e= b/2-$\bar{X}$ =3.0/2 -1.121=0.379 m< (b/6=0.5 m)

Maximum Pressure at toe end=$p_{max} = \frac{\Sigma W}{b}\left[1+\frac{6e}{b}\right] = \frac{210}{3.0}\left[1+\frac{6*0.379}{3.0}\right] = \frac{123.06\ kN}{m^2} < 200\,\frac{kN}{m^2}$

Minimum Pressure at heel end= $p_{min} = \frac{\Sigma W}{b}\left[1-\frac{6e}{b}\right] = \frac{210}{3.0}\left[1-\frac{6*0.379}{3.0}\right] = \frac{16.94kN}{m^2} < 200\,\frac{kN}{m^2}$ and compressive

Design of Stem

Max. horizontal pressure at the base=p_h=(1/3)*15*5=25 kN/m^2

Stem acts as slab supported on counterforts w==25*1=25 kN/m

Maximum negative moment at counterforts=wl^2/12=25*3^2/12=18.75 kNm

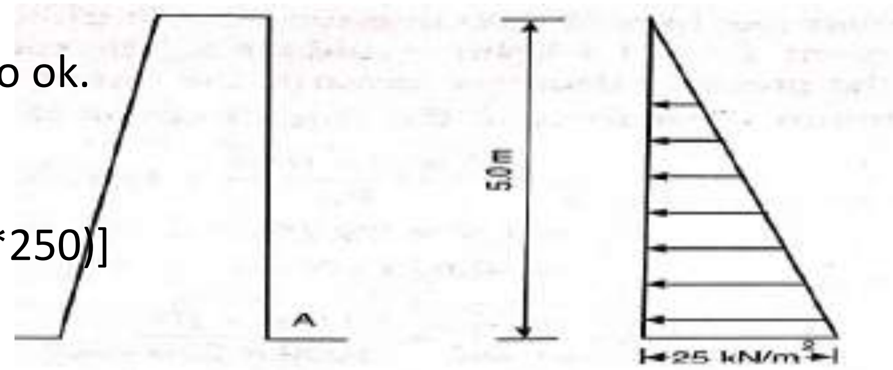
Mu=1.5*18.75=28.125 kNm

Max. positive factored moment at mid span M_u=1.5*wl^2/16=1.525*3*3/16=21.1 kNm

Depth Check= d= SQRT{Mu/Ru.b}=SQRT{28.12*10^6/(2.76*1000)}=101 mm

Assuming effective cover 50 mm

d provided=300-50=250 mm > 101 mm so ok.



Area of steel required

28.125*10^6=

0.87*415*Ast*250[1-415*Ast/(20*1000*250)]

=320 mm2

$A_{st\ min}$ =0.12*1000*300/100=360 mm^2

Using 10 mm dia bars spacing 78.5*1000/360=218 mm

Provide 10 mm dia bars 200 mm c/c in both directions all along the height of the stem.

Increase the spacing to 300 mm near the top as pressure decreases.

Shear Check: Maximum factored shear force at face of the counterfort=V_u=1.5*25*(3.0-0.3)/2=33.75*1.5=50.625 kN

$\tau_c=V_u/bd$=50.625*1000/(1000*250)=0.2 N/mm2

p_t=0.157%

τ_c=0.28 N/mm^2

$\tau_c > \tau_v$ so ok

■ **Design of toe slab:** The pressure distribution under the base slab is as shown below

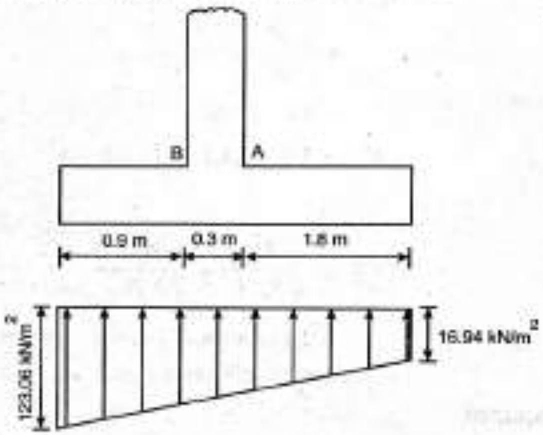


Fig. 16.20.

$$\text{Pressure below point } A = 16.94 + \left(\frac{123.06 - 16.94}{3.0}\right) \times 1.8$$

$$= 80.61 \text{ kN/m}^2$$

$$\text{Pressure below point } B = 16.94 + \frac{(123.06 - 16.94)}{3.0} \times 2.1$$

$$= 91.22 \text{ kN/m}^2$$

Neglecting the weight of earth retained on the toe slab, the cantilever moment at the section B is

$$= 91.22 \times \frac{0.9^2}{2} + \frac{1}{2}(123.06 - 91.22) \times 0.9 \times \frac{2}{3} \times 0.9$$

$$= 45.54 \text{ kNm}$$

$$M_u = 1.5 \times 45.54$$

$$= 68.31 \text{ kNm}$$

$$d_{\text{reqd}} = \sqrt{\frac{68.31 \times 10^6}{2.76 \times 1000}}$$

$$= 157 \text{ mm} < d \text{ provided. Hence o.k.}$$

$$\text{Total depth} = 500 \text{ mm}$$

$$\text{Effective cover} = 60 \text{ mm}$$

$$d_{\text{provided}} = 500 - 60 = 440 \text{ mm}$$

Area of steel required for toe slab:

$$68.31 \times 10^6 = 0.87 \times 415 \times A_{st} \times 440 \left[1 - \frac{415 A_{st}}{20 \times 1000 \times 440}\right]$$

$$A_{st \text{ reqd}} = 440 \text{ mm}^2$$

$$A_{st \min} = \frac{0.12}{100} \times 1000 \times 500 = 600 \text{ mm}^2 > 440 \text{ mm}^2$$

hence provide $A_{st} = 600 \text{ mm}^2$

Using 12 mm diameter bars, $A_\phi = 113 \text{ mm}^2$

$$\text{Spacing required} = \frac{113 \times 1000}{600} = 188 \text{ mm}$$

Hence provide 10 mm diameter bars @ 180 mm c/c in both directions in toe slab.

■ **Shear design:** The critical section for shear is at a distance '*d*' from face of the stem *i.e.*, 0.44 m from stem or

$$0.9 - 0.44 = 0.46 \text{ m from toe edge}$$

$$\text{Pressure at this section} = 91.22 + \frac{1}{2}\left(\frac{123.06 - 91.22}{0.9}\right) \times 0.44$$

$$= 99.0 \text{ kN/m}^2$$

$$\text{S.F. at this section} = 99 \times 0.46 + \frac{1}{2}(123.06 - 99) \times 0.46$$

$$= 51.1 \text{ kN per m run}$$

$$V_u = 1.5 \times 51.1$$

$$V_u = 76.65 \text{ kN}$$

$$\tau_V = \frac{76.65 \times 10^3}{1000 \times 440} = 0.17 \text{ N/mm}^2$$

$$p_t = \frac{600 \times 100}{1000 \times 440} = 0.14\%$$

$$\tau_c = 0.28 \text{ N/mm}^2$$

$$\tau_v < \tau_c \quad \text{Hence OK}$$

■ Design of heel slab

The heel slab also acts as a continuous slab supported on counterforts like stem.

$$\text{Weight of backfill} = 1.0 \times 5.0 \times 15$$

$$= 75 \text{ kN/m}^2 \text{ per m run}$$

$$\text{Self weight of slab} = 1.0 \times 0.5 \times 25 = 12.5 \text{ kN/m}^2$$

$$\text{Total downward weight} = 75 + 12.5 = 87.5 \text{ kN/m}^2$$

Maximum downward pressure at the edge of the heel slab

$$= 87.5 - 16.94 = 70.56 \text{ kN/m}^2$$

$$M = \frac{70.56 \times 3^2}{12} = 52.92 \text{ kNm}$$

$$M_u = 1.5 \times 52.92$$

$$M_u = 79.38 \text{ kNm}$$

Area of Steel required:

$$79.38 \times 10^6 = 0.87 \times 415 \times A_{st} \times 440\left[1 - \frac{415 A_{st}}{20 \times 1000 \times 440}\right]$$

$$A_{st} = 512 \text{ mm}^2 < 600 \text{ mm}^2 \ (A_{st\,min})$$

Hence provide 10 mm diameter @ 180 mm c/c in both directions.

■ **Design of counterforts:** Counterforts are designed as a triangular beam (beam of varying depth) supported on the stem and heel slab. It is also to be designed for the tension which tries to pull the counterfort away from stem and heel.

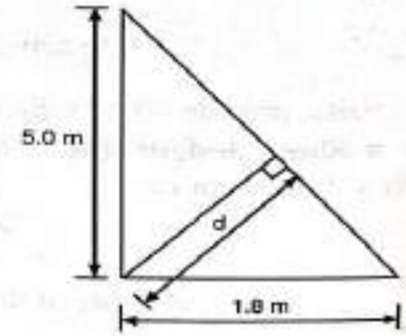


Fig. 16.21.

$$\tan\theta = \frac{5.0}{1.8} = 2.77$$

$$\theta = 43.71°$$

Depth of the triangular beam, (*d*)

$$d = 1.8 \sin\theta$$

$$= 1.8 \sin 43.71°$$

$$d = 1.243 \text{ m}$$

Maximum moment on the counterforts

$$= \left(\frac{1}{2} K_a \gamma h . h . \frac{h}{3}\right) \times L \quad \text{where } L \text{ is the spacing of counterforts}$$

$$= \left(\frac{1}{2} \times \frac{1}{3} \times 15 \times 5 \times 5 \times \frac{5}{3}\right) \times 3.0$$

$$M = 312.5 \text{ kNm}$$

$$M_u = 1.5 \times 312.5 = 468.75 \text{ kNm}$$

Area of steel required:

$$468.75 \times 10^6 = 0.87 \times 415 \times A_{st} \times 1243 \left[1 - \frac{415 A_{st}}{20 \times 300 \times 1243}\right]$$

$$A_{st} = 1114 \text{ mm}^2$$

$$A_{st\ min} = \frac{0.85 bd}{f_y} = \frac{0.85 \times 300 \times 1243}{415}$$

$$= 763 \text{ mm}^2 < 1114 \text{ mm}^2 \quad \text{Hence OK}$$

Providing 4 bars bars of 20 mm diameter

$$A_{st\ provided} = 4 \times \frac{\pi}{4} \times 20^2 = 1256 \text{ mm}^2 \quad \text{[curtailing 2 bars near the top]}$$

■ Design for Horizontal Tension in Counterforts

Horizontal ties are used for taking horizontal tension, caused due to the lateral earh pressure.

Considering the bottom 1 m height of the stem.

Maximum lateral pressure at the bottom

$$= K_a \gamma h$$

$$= \frac{1}{3} \times 15 \times 5$$

$$= 25 \text{ kN/m}^2$$

Total lateral pressure to be taken by counterforts

$$= 25(3 - 0.3) \text{ per m run}$$

$$= 67.5 \text{ kN}$$

$$\text{Factored tensile force } = 1.5 \times 67.5$$

$$= 101.25 \text{ kN}$$

Area of steel required: $T = 0.87 f_y A_{st}$

$$A_{st} = \frac{101.25 \times 1000}{0.87 \times 415} = 281 \text{ mm}^2$$

Providing 10 mm bars $A_\phi = 78.5 \text{ mm}^2$

$$\text{Spacing required } = \frac{78.5 \times 1000}{281} = 279 \text{ mm}$$

Provide 10 mm diameter ties @ 260 mm c/c in the horizontal direction.

- **Design for Vertical tension in counterforts**

The vertical tension in counterforts is caused due to the downward pressure which tries to separate out the counterfort and the heel.

Maximum downward pressure on the counterfort at the edge of heel = 70.56 kN/m^2

$$\text{Factored tensile force} = 1.5 \times 70.56$$

$$= 105.84 \text{ kN}$$

$$\text{Area of steel required } A_{st} = \frac{T}{0.87 f_y}$$

$$= \frac{105.84 \times 1000}{0.87 \times 415}$$

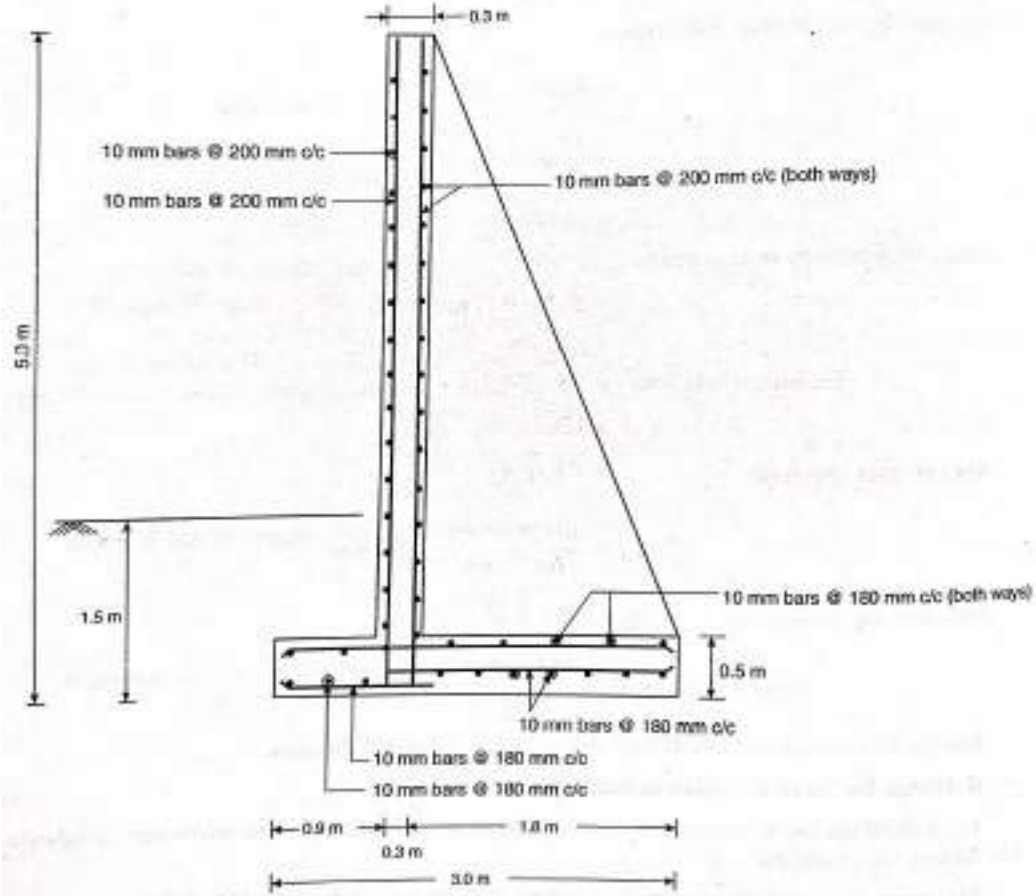
$$A_{st} = 293 \text{ mm}^2$$

Using 10 mm diameter bars,

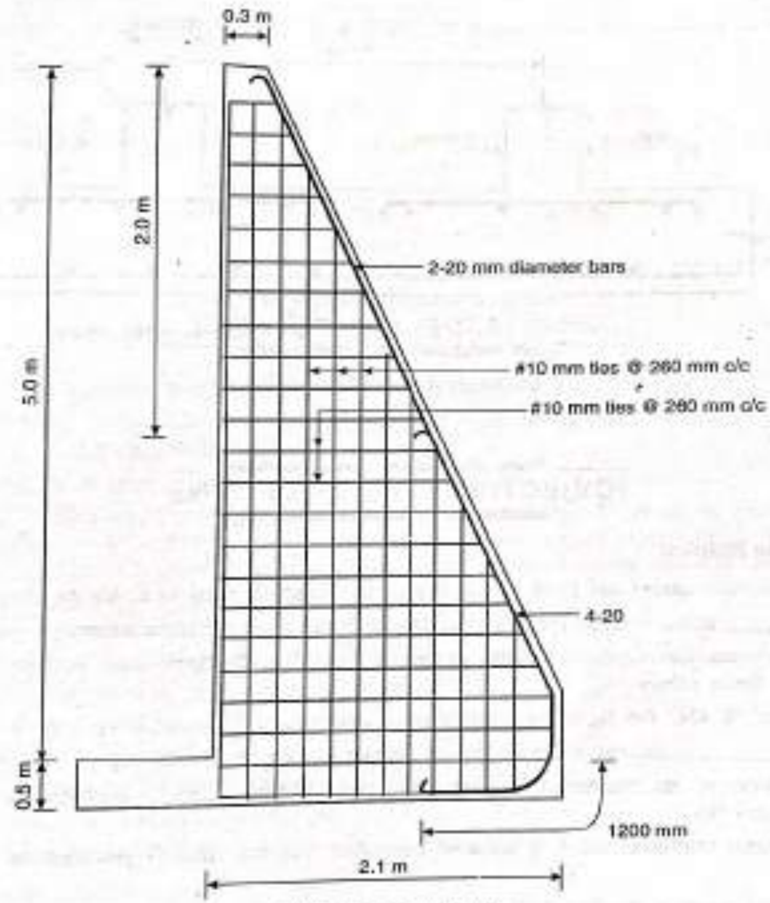
$$S_{v\,reqd} = 267 \text{ mm}$$

Hence provide 10 mm diameter ties @ 260 mm c/c.

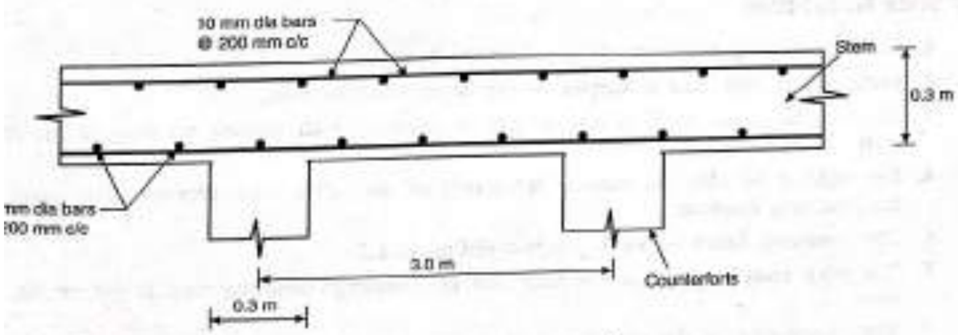
The detailing of reinforcement of the counterfort retaining wall is shown in Fig.



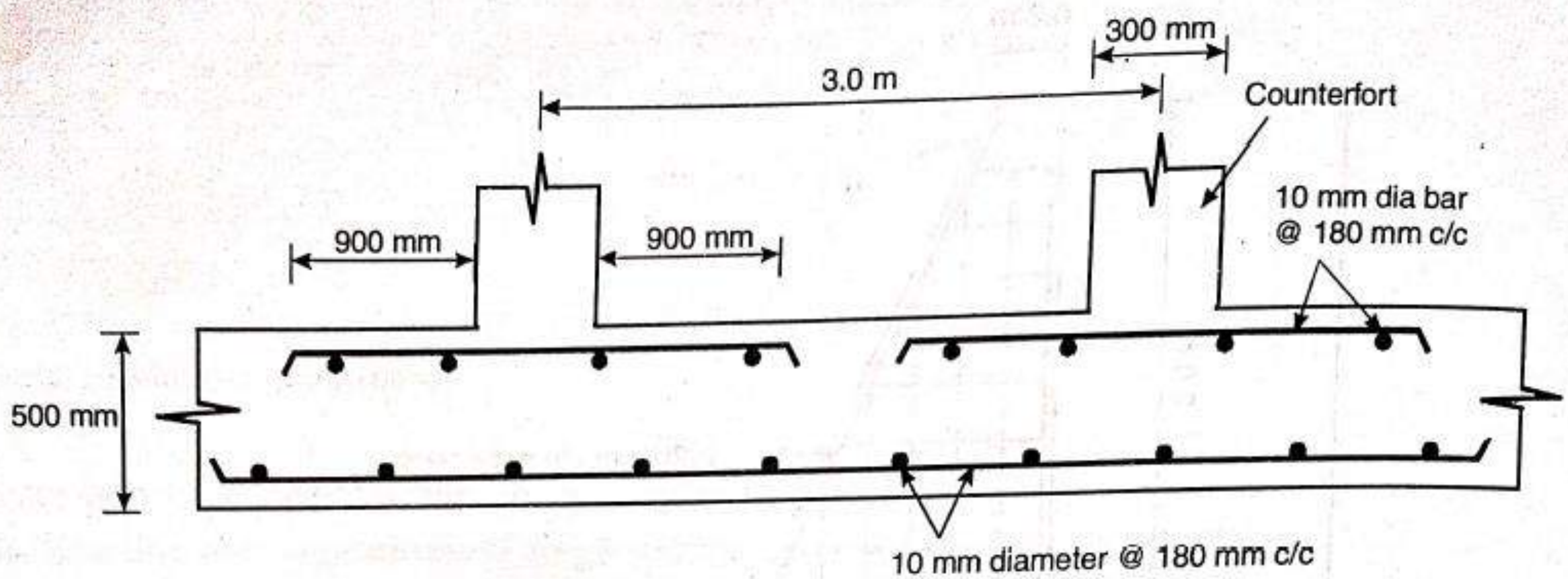
(a) Section at midspan of counterforts



(*b*) Section at counterfort



(*c*) Details of Stress reinforcement at base



(*d*) Details of heel slab (L-section)