DESIGN AND DETAILING OF COUNTERFORT RETAINING WALL

When the height of the retaining wall exceeds about 6 m, the thickness of the stem and heel slab works out to be sufficiently large and the design becomes uneconomical. In such a case counterforts having trapezoidal section fixed at the base slab are provided at intervals of 1.5 m to 3 m. The counterforts support the heel slab and the vertical stem. The design principles for different components of the wall are discussed as under.

Design of Stem
The stem acts as a continuous slab spanning longitudinally over the counterforts. The horizontal active soil pressure acts as the load on the slab. Since the earth pressure varies linearly over the height of the stem, the slab deflects away from the earth face between the counterforts and hence the main steel is provided at the outer face of the stem and at the inner face near the supporting counterforts. The bending moment in the stem is maximum at the base and reduces towards top. But the thickness of the wall is kept constant and only the area of steel is reduced.

If \( l \) is the clear distance between the counterforts and \( p \) is the intensity of soil pressure, the slab is designed for bending moment as under:

- Maximum +ve B.M = \( p l^2 / 16 \) (occurring mid-way between counterforts) and
- Maximum -ve B.M = \( p l^2 / 12 \) (occurring at inner face of counterforts)

The main reinforcement is provided horizontally along the length of the wall. The ties are provided horizontally for the full value of reaction to prevent slab separating from counterforts.

Design of Toe Slab
The base width is approximately taken equal to 0.6 \( H \) to 0.7 \( H \), where \( H \) is the overall height of the wall. The projection of toe slab is approximately taken between 1/3 to 1/4 of base width. The toe slab is subjected to an upward soil reaction and is designed as a cantilever slab fixed at the front face of the stem. Due to upward soil pressure, the tension develops on the earth face and the reinforcement is provided on earth face along the length of the toe slab. In case the toe slab projection is large i.e. \( b/3 \), front counterforts are provided above the toe slab (normally up to the ground level) and the slab is designed as a continuous horizontal slab spanning between the front counterforts.

Design of Heel Slab
The heel slab is designed as a continuous slab spanning over the counterforts, as in the case of stem. The heel slab is subjected to downward forces due to weight of soil plus self weight of slab and an upward force due to soil reaction. The net force acts downward producing tension towards the earth face between the counterforts and negative moment develops at the support provided by counterforts.

If \( p \) is the net downward force and \( l \) is the clear span between the counterforts the
B.M. is given by:
Maximum +ve B.M = \( \frac{pl^2}{16} \) (mid-way between counterforts towards earth face)
Maximum -ve B.M. = \( \frac{pl^2}{12} \) (occurring at counterforts)

**Design of Counterforts**

Since the active earth pressure on stem acts outward and stem is considered to be fixed at counterforts, the counterforts are subjected to outward reaction from the stem. This produces tension along the outer sloping face of the counterforts. The inner face supporting the stem is in compression. Thus, the stem lies in the compression zone with respect to the bending of the counterforts and hence the counterforts are designed as a T-beam of varying depth. The main steel provided along the sloping face shall be anchored properly at both ends. The depth of the counterfort is measured perpendicular to the sloping side.

In order that the counterfort and stem should act as one unit, it is joined firmly to the stem by providing ties in the horizontal plane. The base is tied with vertical ties to prevent its tendency to separate out under the action of net downward force. The provision of ties ensures transfer of forces to the counterforts. The net forces acting on the different components of the counterforts, position of main steel and horizontal and vertical ties are schematically shown.

**PROBLEM:** A R.C.C. retaining wall with counterforts is required to support earth to a height of 7 m above the ground level. The top surface of the backfill is horizontal. The trial pit taken at the site indicates that soil of bearing capacity 220 kN/m\(^2\) is available at a depth of 1.25 m below the ground level. The weight of earth is 18 kN/m\(^3\) and angle of repose is 30°. The coefficient of friction between concrete and soil is 0.58. Use concrete M20 and steel grade Fe 415. Design the retaining wall.

Given: \( f_{ck} = 20\text{ N/mm}^2, f_y = 415\text{N/mm}^2, H = 7\text{ m above G.L}, \) Depth of footing below G.L. = 1.25 m, \( \gamma = 18\text{ kN/m}^3, \mu = 0.58, f_b = 220\text{ kN/m}^2 \)

Required : Design the counterfort retaining wall.

<table>
<thead>
<tr>
<th>Design constants</th>
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<tbody>
<tr>
<td>( Q = 2.76\text{ N/mm}^2 )</td>
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<tr>
<td>( L_d = (0.87 f_y / 4 \tau_{bd}) \varphi = 0.87 \times 415/4 \times (1.2 \times 1.6) \varphi = 47 \varphi_{bar} )</td>
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<tr>
<td>For ( \varphi = 30^\circ )</td>
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<tr>
<td>Coefficient of active pressure ( k_a = (1 - \sin \varphi)/(l + \sin \varphi) = 1/3 )</td>
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<tr>
<td>Coefficient of passive pressure ( k_p = (1 + \sin \varphi)/(l - \sin \varphi) = 3 )</td>
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**a. Proportioning of Wall Components**

The height of the wall above the base = \( H = 7 + 1.25 = 8.25 \) m.
Base width = 0.6 H to 0.7 H i.e. between 4.95 m to 5.78 m.
Assume base width \( b = 5.5 \) m
Toe projection = \( b/4 = 5.5/4 = \) say 1.2 m
Assume thickness of vertical wall = 250 mm
Thickness of base slab = 450 mm

Clear spacing between counterforts is given by:

\[ L = 3.5 \left( \frac{H}{\gamma} \right)^{0.25} = 3.5 \left( \frac{8.25}{18} \right)^{0.25} = 2.88 \text{ m} \]

\[ \therefore \text{ c/c spacing} = 2.88 + 0.25 = 3.13 \text{ m say 3 m} \]

\[ \therefore \text{Provide counterforts at 3 m c/c.} \]

Assume width of counterfort = 400 mm

\[ \therefore \text{clear spacing provided} = L = 3 - 0.4 = 2.6 \text{ m} \]

The preliminary dimensions of the components of the wall are shown in Figure.

\[ \text{Vertical earth pressure on full height of wall} = P_h = \gamma h^2 k_a / 2 = 18 x 8.25^2 / (3 x 2) = 204.19 \text{ kN} \]

\[ \text{Horizontal earth pressure on full height of wall} = \]
Overturning moment = \( M_0 = P_h \times H/3 = 204.19 \times 8.25/3 \)
= 561.52 kN.m.
Factor of safety against overturning
= \( \sum M / M_0 = 2210.71/561.52 = 3.937 > 1.55 \) \( \therefore \) safe.

Check for sliding
Total horizontal force tending to slide the wall = \( P_h = 204.19 \) kN
Resisting force = \( \sum \mu.W = 0.58 \times 679.25 = 393.97 \) kN
\( \therefore \) Factor of safety against sliding = \( \sum \mu.W / P_h = 393.97/204.19 \)
= 1.93 > 1.55 \( \therefore \) safe.

Check for pressure distribution at base
Net moment = \( M = 2210.71 - 561.52 = 1649.19 \) kN.m.
Let \( x \) be the distance from the toe where the resultant \( R \) acts,
\( \therefore x = M / \sum W = 1649.19/679.25 = 2.43 \) m
Eccentricity = \( e = b/2 - x = 5.5/2 - 2.43 = 0.32 < b/6 \) (= 0.91 m)
\( \therefore \) Whole base is under compression.
Maximum pressure at toe
\( = p_A = \sum W / b (1 + 6e/b) = 679.25/5.5 (1 + 6*0.32/5.5) \)
= 166.61 kN/m\(^2\) < \( f_b \) (= 220 kN/m\(^2\))
Minimum pressure at heel
\[ p_D = \frac{\sum W}{b (1 - 6e/b)} = 679.25/5.5 \times (1 - 6 \times 0.32/5.5) \]
\[ = 80.39 \text{ kN/m}^2 \text{ compression.} \]
The distribution of stresses under the base is shown in Fig

Intensity of pressure at junction of stem with toe i.e. under B
\[ p_B = 80.39 + (166.61 - 80.39) \times 4.3/5.5 = 147.8 \text{ kN/m}^2 \]
Intensity of pressure at junction of stem with heel i.e. under C
\[ = P_c = 80.39 + (166.61 - 80.39) \times 4.05/5.5 = 143.9 \text{ kN/m}^2 \]

(b) Design of Toe slab
Since the projection of the toe is small, it is designed as a cantilever fixed at the stem.
Intensity of pressure at B = 147.8 kN/m²
Neglecting the weight of soil above the toe slab, the forces acting on the toe slab are:
(i) downward force due to weight of toe slab TB
(ii) upward soil pressure on length AB.
Ultimate moment at B,
\[ M_B = L.F \text{ (moment due to soil pressure - moment due to wt. of slab TB) } \]
\[ = 1.5 \times (147.8 \times 1.2^2/2 + (166.61 - 147.8) \times 1.2 \times 2/3 \times 1.2) \]
\[ - (25 \times 1.2 \times 0.45 \times 1.2/2) = 174.57 \text{ kN.m.} \]
\[ \therefore d = \frac{174.57 \times 10^6}{(2.76 \times 1000)} = 251.49 \text{ mm} < d (=390 \text{ mm } \therefore \text{o.k.}) \]
\[ M_u/bd^2 = 1.15, \rho_t = 0.343, A_{slab} = 1336 \text{ mm}^2 \]
Using # 16 mm bars, spacing = 1000 x 201/1335 = 150 mm
However, the spacing is limited to 110 mm c/c from shear considerations.
\[ \therefore \text{Provide #16 mm @ 110 mm c/c, Area provided = 1827 mm}^2, \rho_t = 0.47\% \]

The bars shall be extended beyond the front face of the wall for a distance equal to development length of 750 mm (= 47 x 16) Distribution steel = 0.12 x 1000 x 450/100 = 540 mm²
Provide #12 mm at 200 mm c/c. Area provided = 565 mm²

Check for Shear
Since the soil pressure induces compression in the wall the critical section for shear is taken at a distance d from the face of the stem. Intensity of pressure at distance d (= 390 mm) from the face of the toe.
\[ p_E = 80.39 + (166.61 - 80.39) (4.3 + 0.39)/5.5 = 153.9 \text{ kN/m}^2 \]

Net vertical shear = Shear due to pressure varying from 166.61 kN/m² to 153.9 kN/m²
- Shear due to downward force of slab in length of 0.81 m (= 1.2 - 0.39) = (166.61 + 153.9) x 0.81/2 - (25 x 0.45 x 0.81) =120.7 kN.
\[ \therefore \text{Net ultimate shear } = V_{u,\text{max}} = 1.5 \times 120.7 =181.05 \text{ kN.} \]
\[ \zeta_p = 181.05 \times 10^6 / 1000 \times 390 = 0.46 \text{ MPa} \]
\[ \zeta_c = 0.47 \text{ MPa for } \rho_t = 0.47\%, \therefore \text{safe.} \]

(c) Design of Heel Slab
The heel slab is designed as a continuous slab supported on counterforts. The downward force will be maximum at the edge of the slab where intensity of soil pressure is minimum.

\[ \text{\therefore Consider 1 m wide strip near the outer edge D} \]

The forces acting near the edge are:

(a) Downward wt. of soil of height 7.8 m = 18 \times 7.8 \times 1 = 140.4 \text{ kN/m}

(b) Downward wt. of heel slab = 25 \times 0.45 \times 1 = 11.25 \text{ kN/m}

(c) Upward soil pressure of intensity 80.39 kN/m\(^2\) = 80.39 \times 1 = 80.39 \text{ kN/m}

\[ \text{\therefore Net downward force at D} = 140.4 + 11.25 - 80.39 = 71.26 \text{ kN/m} \]

Also net downward force at C = 140.4 + 11.25 - 143.9 = 7.75 \text{ kN/m}

Let the width of the counterfort = 400 mm

Clear spacing between counterforts = \( \ell = 2.6 \text{ m} \)

Maximum -ve ultimate moment in heel slab at counterfort

\[ M_u = \frac{(L.F.) \cdot p\ell^2}{12} = 1.5 \times 71.26 \times 2.6^2/12 = 60.2 \text{ kN.m.} \]

\[ M_u / b d^2 = 60.2 \times 10^6 / (1000 \times 390^2) = 0.4, \quad p_t = 0.114, \text{ provide 0.12\%GA} \]

\[ (A_{st})_{\min} = 0.12 \times 1000 \times 450/100 = 540 \text{ mm}^2 \]

\[ \text{\therefore Provide \#12 mm @ 200 mm c/c, Area provided = 565 mm}^2 \]

\[ P_{t_r} = 100 \times 565 \div (1000 \times 390) = 0.14 \% \]

**Check for shear**

Maximum shear = \( V_{\text{umax}} = 1.5 \times 71.26 \times 2.6/2 = 139 \text{ kN} \)

\[ \zeta_v = 139 \times 10^3 / 1000 \times 390 = 0.36 \text{ MPa} \]

\[ \zeta_c = 0.28 \text{ MPa}. \text{ Unsafe and hence shear steel is needed.} \]

Using \#8 mm 2-legged stirrups,

\[ \text{Spacing} = 0.87 \times 415 \times 100 \div [(0.36 - 0.28) \times 1000] \]

\[ = 452 \text{ mm} \leq (0.75 \times 390 = 290 \text{ mm or } 300 \text{ mm}) \]

\[ \text{\therefore Spacing} = 290 \text{ mm} \]

\[ \text{\therefore Provide \#8 mm 2-legged stirrups at 290 mm c/c.} \]

Since shear force varies linearly along the span of 2.6 m, the zone of design shear reinforcement can be determined.

Let \( x_1 \) be the distance from the counterfort where S.F. = 109.2 kN

then \( x_1 = 1.30 \times (139-109.2) / 139 = 0.28 \text{ m} \)

Further in the transverse direction the S.F. decreases due to increase in the soil pressure.

Let the net downward ultimate force/m at a distance \( y_1 \) from C be equal to \( w_1 \)

Then ultimate S.F. at \( y_1 = w_1 \times 2.60/2 = 1.30 w_1 \)

and this must be equal to \( V_{uc} \) i.e. \( 1.30 w_1 = V_{uc} = 109.2 \therefore w_1 = 84 \text{ kN} \)
net downward working load = \( w' = 84/1.5 = 56 \, \text{kN/m} \).

Now, variation of net downward force is linear having value of 71.26 kN/m at D and 7.75 kN/m at C.

Let \( y_2 \) be the distance from C where net downward force is 56 kN/m.

\[ y_2 / (56 - 7.75 ) = 4.05 / (17.26 - 7.75) \quad \therefore \quad y_2 = 3.08 \, \text{m from C}. \]

\[ y_1 = 4.05 - 3.08 = \text{say 1 m from end D} \]

\[ \therefore \quad \text{Provide # 8 mm 2-legged stirrups in heel slab at 290 mm c/c for a distance of 0.340 m on either side of the counterfort and for a length of 1 m along the length of the counterfort in the triangular portion. But from practical considerations provide the stirrups in the rectangular portion of (0.34 m x 1 m)}. \]

**Check for development length**

For a continuous slab the check for development length satisfying the curtailments rules as per SP34.

**Area of steel for +ve moment**

Maximum +ve ultimate moment = L.F. \( \times \frac{p^2}{16} \)

\[ = \frac{3}{4} M_u = 0.75 \times 60.2 \]

\[ = 45.15 \, \text{kN.m}. \]

\[ M_u/\text{bd}^2 = \text{Very small} \]

Too small and hence provide minimum steel. \( A_{\text{stmin}} = 540 \, \text{mm}^2 \)
.

Check the force at junction of heel slab with stem
The intensity of downward force decreases due to increases in upward soil reaction. Consider m width of the slab at C
Net downward force \( = 18 \times 7.8 + 25 \times 0.45 - 143.9 = 7.75 \text{ kN/m} \).
\[ \therefore \text{Provide only minimum reinforcement. Provide # 12 mm bars at 200 mm c/c.} \]

Distribution steel
\[ A_{st} = 0.12 \times 1000 \times 450/100 = 540 \text{ mm}^2 \]
Using # 12 mm bars, spacing = 1000 x 113/468 = 241 mm.
Provide # 12 mm at 200 mm c/c., Area provided = 565 mm\(^2\)

(d) Design of Stem (Vertical Slab)
The stem acts as a continuous slab spanning between the counterforts. It is subjected to linearly varying earth pressure having maximum intensity at bottom.
Consider 1 m wide strip at bottom of stem at C.
The intensity of earth pressure
\[ = P_h = k_a \gamma h = 18 \times 7.8/3 = 46.8 \text{ kN/m}^2 \]
Area of steel on earth side near counterforts:
Maximum -ve ultimate moment,
\[ M_u = 1.5 \times ph \frac{1^2}{12} = 1.5 \times 46.8 \times 2.6^2/12 = 39.54 \text{ kN.m} \]
Required \( d = \sqrt{39.54 \times 10^6/(2.76 \times 1000)} = 119 \text{ mm} \)
However provide total depth = 250 mm
Assuming effective cover = 60 mm, \( d = 250 - 60 = 190 \text{ mm} \)
\[ M_u/\beta d^2 = 1.1, \quad P_t = 0.33, \quad A_{st} = 627 \text{ mm}^2 \]
Provide #12 mm @ 180 mm c/c,

However provide #12 mm @ 110 mm c/c from shar. Area provided = 1000 x 113/110 = 1027.27 mm\(^2\)
\[ P_t = 100 \times 1027.27/(100 \times 190) = 0.54 \%
\]
As the earth pressure decreases towards the top, the spacing of the bars is increased with decrease in height.

Distribution steel
\[ A_{sl} = 0.12 \times 1000 \times 250/100 = 300 \text{ mm}^2 \]
\[ \therefore \text{Area of steel on each face} = 300/2 = 150 \text{ mm}^2 \]
Provide # 8 mm @ 300 mm on each face in the vertical direction.
Area provided = 1000 x 50/300 = 167 mm\(^2\)
On the front face provided nominal steel φ 8 mm at 300 mm c/c to support the vertical bars.

(e) Design of Counterfort
Width of counterfort = 400 mm. The counterforts are provided at 3 m c/c. They are subjected to earth pressure and downward reaction from the heel slab.
At any section at any depth \( h \) below the top \( E \) the total horizontal earth pressure acting on the counterfort.

\[ \text{B.M. at any depth } h = 9h^2xh/3 = 3h^3 \]

B.M. at the base at \( C = 3 \times 7.8^3 = 1423.7 \text{ kN.m.} \)

Ultimate moment \( M_u = 1.5 \times 1423.7 = 2135.6 \text{ kN.m.} \)

Net downward pressure on heel slab at \( D \)

\( = \) wt. due to earth pressure + wt. of heel slab - upward soil pressure

\[ = 18 \times 7.8 + 25 \times 0.45 - 80.39 = 71.26 \text{ kN/m} \]

Net downward pressure on heel slab at \( C \)

\[ = 18 \times 7.8 + 25 \times 0.45 - 143.9 = 7.75 \text{ kN/m}^2. \]

\( \therefore \) Total downward force at \( D = 71.26 \times \text{c/c distance} = 71.26 \times 3 \)

\[ = 213.78 \text{ kN.m.} \]

Total downward force at \( C = 7.75 \times \text{c/c distance} = 7.75 \times 3 \)

\[ = 23.25 \text{ kN.m.}. \]

As mentioned earlier the counterfort acts as a T-beam. As can be seen that the depth available is much more than required from B.M. considerations.

Even assuming rectangular section,

\[ d = \sqrt{(2135.6 \times 10^6(2.76 \times 400))} = 1390 \text{ mm} \]

The available depth is obtained as under:

\[ \text{The effective depth is taken at right angle to the reinforcement.} \]

\[ \tan \theta = 7.8/4.05 = 1.93, \quad \theta = 62.5^\circ, \]

\( \therefore \) \( d = 4050 \times \sin \theta - \text{eff. cover} \)

\[ = 3535 \text{ mm} > > 1390 \text{ mm} \]

\[ M_u/bd^2 = 2135.6 \times 10^6/(400 \times 3535^2) = 0.427, \quad \rho_t = 0.12\%, \quad A_{st} = 1696 \text{ mm}^2 \]

\[ A_{st.min} = 0.85 \times bd/f_y = 0.85 \times 400 \times 3535/415 = 2896 \text{ mm}^2 \]

\( \therefore \) Provided 4- \# 22 mm + 4 - \# 22 mm, Area provided = 3041 mm\(^2\)

\[ \rho_t = 100 \times 3041/(400 \times 3535) = 0.21 \% \]
The height \( h \) where half of the reinforcement can be curtailed is approximately equal to \( \sqrt{H} = \sqrt{7.8} = 2.79 \) m
Curtail 4 bars at 2.79-L_{df} from top i.e, 2.79-1.03 =1.77m from top.

**Design of Horizontal Ties**
Due to horizontal earth pressure, the vertical stem has a tendency of separating out from the counterforts, Hence it should be tied to it by horizontal ties.
The direct pull by the wall on counterfort for 1 m height at base
= \( \gamma h k_a x \) c/c distance =18 x 7.8 x 3 x 1/3 = 140.4 kN
Area of steel required to resist the direct pull
= 1.5 x 140.4 x 10^3/(0.87 x 415) = 583 mm^2 per meter height.
Using # 8 mm 2-legged stirrups, \( A_{st} = 2 \times \pi \times 8^2/4 = 100 \) mm^2
spacing = 1000 x 100/583 = 170 mm c/c.
\( \therefore \) Provide # 8 mm 2-legged stirrups at 170 mm c/c.
Since the horizontal pressure decreases with \( h \), the spacing of stirrups can be increased from 170 mm c/c to 450 mm c/c towards the top.

**Design of Vertical Ties**
Due to net vertical downward force acting on the base slab, it has a tendency to separate out from the counterfort. This is prevented by providing vertical ties.
The maximum pull will be exerted at the end of heel slab where the net downward force = 71.26 kN/m.
Consider one meter strip
Total downward force at D
= 71.26 x c/c distance between counterforts = 71.28 x 3 = 213.78 kN.
Required \( A_{st} = 1.5 x 213.78 x 10^3/(0.87 x 415) = 888 \) mm^2
Using # 8 mm 2-legged stirrups , \( A_{st} = 100 \) mm^2
spacing = 1000 x 100/888 = 110 mm c/c.
\( \therefore \) Provide # 8 mm 2-legged stirrups at 110 mm c/c.
Total downward force at C= 3 x 7.75 = 23.25 kN
Required \( A_{st} = 1.5 x 23.25 x 10^3/(0.87 x 415) = 96.6 mm^2 \) very less.
\( \therefore \) Increase the spacing of vertical stirrups from 110 mm c/c to 450 mm c/c towards the end C.
Cross sectional details of wall through the counterfort

Cross section between counterforts
Section through stem at the junction of Base slab.