APPENDIX 1

A-1 DESIGN OF BLAST RESISTANT RESIDENTIAL BUILDING

As a case study a four storied residential building was designed to withstand a blast overpressure of 8 psi (56 Kpa). The Analysis of the structure and the design procedure for designing the structural elements are given below.

Design of Slab

Step 1: Data

Size of the span = 2728×4550 mm
Concrete grade, f_{ck}(assuming) = 30 N/mm²
Steel grade, f_{y}(assuming) = 415 N/mm²
Shorter side of panel, L_{x} = 2.728 m
Longer side of panel, L_{y} = 4.550 m
L_{y}/L_{x} = 4.550/2.728
= 1.67 < 2

Hence it is **two way** slab.

Assume depth, D = 150 mm
Clear cover, C_{c} = 20 mm
Diameter of bar, \( \phi \) = 10 mm
Effective depth, d = 150-20-(10/2) = 125 mm
Step 2: Load Calculation

Self weight = 25×0.15 = 3.75 kN/m²

Live load, L.L = 2 kN/m²

Floor finish, F.F = 1 kN/m²

Total load, w = 3.75+2+1 = 6.75kN/m²

Service load, w = 6.75*1.5 =10.13 kN/m²

Step 3: To Find Ultimate Design Moment (Refer Table 26 and 27 of IS 456)

Type of panel: one short edge continuous

\[ M_{ux}(-ve) = \alpha_x (-ve)wl_x^2 \text{ (Clauses D-1.1 of IS 456:2000)} \]

\[ = 4.82 \text{ KNm} \]

\[ M_{ux}(+ve) = \alpha_x (+ve)wl_x^2 \text{ (Clauses D-1.1 of IS 456:2000)} \]

\[ = 3.62 \text{ KNm} \]

\[ M_{uy}(-ve) = \alpha_y(-ve)wl_x^2 \text{ (Clauses D-1.1 of IS 456:2000)} \]

\[ = 2.78 \text{ KNm} \]

\[ M_{uy}(+ve) = \alpha_y(+ve)wl_x^2 \text{ (Clauses D-1.1 of IS 456:2000)} \]

\[ = 2.11 \text{ KNm} \]

Step 4: Check for depth

\[ M_U = 0.36 \times (X_u \text{ max}/d) \times (1-0.42(X_u \text{ max}/d)) \times f_{ck} \times b \times d^2 \]

\[ \text{ (Annex G-1.1.c of IS 456:2000)} \]

\[ d_{\text{required}} = 34.12 \text{ mm}^2 \]
\(d_{\text{provided}} = 125 \text{ mm}^2\)

\(d_{\text{required}} < d_{\text{provided}}\)

Hence it is safe.

**Step 5: Check for reinforcement**

\[M_U = 0.87 \times F_y \times A_{st} \times d \times (1 - (A_{st} \times f_y) / (b \times d \times f_{ck}))\]

(Annex G-1.1.c of IS 456:2000)

\[(4.82 \times 10^6) = (0.87 \times 415 \times A_{st} \times 125 \times (1 - (415 \times A_{st}) / (1000 \times 30 \times 125))\]

\(A_{st}(-\text{ve}) = 108.1 \text{ mm}^2\)

\[(3.62 \times 10^6) = (0.87 \times 415 \times A_{st} \times 125 \times (1 - (415 \times A_{st}) / (1000 \times 30 \times 125))\]

\(A_{st}(+\text{ve}) = 80.9 \text{ mm}^2\)

\(d = 125 - 10 = 115 \text{ mm}\)

\[(2.78 \times 10^6) = (0.87 \times 415 \times A_{st} \times 115 \times (1 - (415 \times A_{st}) / (1000 \times 30 \times 115))\]

\(A_{st}(-\text{ve}) = 67.5 \text{ mm}^2\)

\[(2.11 \times 10^6) = (0.87 \times 415 \times A_{st} \times 115 \times (1 - (415 \times A_{st}) / (1000 \times 30 \times 115))\]

\(A_{st}(+\text{ve}) = 51.13 \text{ mm}^2\)

**Minimum area of steel:**

\(A_{st}(\text{min}) = 0.12\% BD = 180 \text{ mm}^2\) (Clause 26.5.1.1.a of IS 456:2000)

**Maximum area of steel:**

\(A_{st}(\text{max}) = 4\% BD = 6000 \text{ mm}^2\) (Clause 26.5.1.1.b of IS 456:2000)
Step 6: Spacing

\[ S = \frac{((\pi/4) d^2)}{A_{st}(\text{min})}\times1000 \]

\[ S_1, S_2, S_3, S_4 = \frac{((\pi/4) 10^2)}{180}\times1000 = 440 \text{ mm} \]

Minimum spacing permitted = 3d = 375 mm > 300 mm

Hence provide 10mm φ bar at 300mm c/c.

Step 7: Check for deflection

\[ A_{st} = \frac{((\pi/4) 10^2)}{300}\times1000 \]

\[ A_{st} = 261.8 \text{ mm}^2 \]

\[ P_t = \frac{(261.8)/(1000\times125)}{100} = 0.21 \]

\[ F_s = 0.58\times415\times180/261.8 = 165.5 \]

Modification factor (mf) = 2

(Source: Pg. No.38, Figure 4 of IS 456:2000)

\[ k = 1.3 \]

\[ (L/D)\times mf = 26 = 52 \]

\[ d_{\text{required}} = 87.5 \text{ mm} \]

\[ d_{\text{provided}} = 125 \text{ mm} \]

Hence it is safe.

Step 8: Check for shear

\[ V_u = w_u l_s/2 = 13.82 \text{ KN/m} \]

\[ \tau_v = 0.11 \text{ N/mm}^2 \]
\[ P_t = 0.21 \text{ (Refer Table 19 of IS 456)} \]

\[ K\tau_c = 1.3 \times 0.338 = 0.439 \text{ N/mm}^2 \]

\[ \tau_c < K\tau_c \text{ Hence it is safe.} \]

**Step 9: Torsion steel**

At corner where slab is discontinuous over the edge

\[ A_{st\ (Torsion)} = \frac{3}{8} \times A_{st\ (min)} = 67.5 \text{ mm}^2 \]

\[ S_2 = \frac{((\pi/4) \times 8^2)/67.5} \times 1000 \]

\[ = 744.67 \text{ mm} > 300 \text{ mm} \sim 300 \text{ mm} \]

Hence provide 8 mm \( \varphi \) bar at 300mm c/c.

**Design of beam**

**Step 1: Data**

Length of Clear span \( = 3 \text{ m} \)

Assume width of supports \( = 300 \text{ mm} \)

Concrete grade, \( f_{ck}(assuming) = 30 \text{ N/mm}^2 \)

Steel grade, \( f_y(assuming) = 415 \text{ N/mm}^2 \)

**Cross sectional dimensions:**

Overall depth \( D = 500 \text{ mm} \)

Effective depth \( d = 475 \text{ mm} \)

Breadth \( b = (D/2) = 250 \text{ mm} \)
Width of support = 300 mm
Effective end span = Clear span + Width of support = 4.55 + 0.3 = 4.85 m
Effective interior span = 4.7 m
Imposed load on end span = 16.95 KN/m
Imposed load on interior span = 16.73 KN/m
Design imposed load on end span = 1.5 * 16.95 = 25.425 KN/m
Design Imposed load on Interior span = 16.73 * 1.5 = 25.09 KN/m

Step 2: Loads
Self weight of beam = 0.5 * 0.25 * 25 = 3.125 kN/m
Design self weight = 3.125 * 1.5 = 4.68 KN/m
Total load on end span = 30.1 KNm
Total load on interior span = 30 KNm

Step 3: Ultimate moments
Assume both impose load and dead load are fixed.

Bending moment near middle of end span,

\[ M_1 = \frac{WL^2}{12} = 58.81 \text{ KNm} \]

Bending moment at middle of interior span,

\[ M_2 = \frac{WL^2}{16} = 41.42 \text{ KNm} \]
Bending moment at support next to the end support,
\[ M_3 = \frac{(-1/2 \times (WL^2) / 10) + ((WL^2) / 10)}{} = -68.415 \text{ KNm} \]

Bending moment at interior span,
\[ M_4 = \frac{-(WL^2)}{12} = 55.225 \text{ KNm} \]

**Step 4: Ultimate Shear force**

Shear force at end support
\[ = 0.4 \times w l = 58.25 \text{ KN} \]

Shear force at support next to end support
\[ = 0.6 \times w l = 87.3 \text{ KN} \]

\[ M_{U(\text{lim})} = 0.36 \times (X_{\text{umax}/d}) \times (1 - 0.42 (X_{\text{umax}/d}) \times f_{\text{ck}} \times b \times d^2) \]

(Annex G-1.1.c of IS 456:2000)

\[ = 233.52 \text{ KNm} \]

\[ M_U < M_{U(\text{lim})} \]

Hence the section is under reinforced section.

For support moment:
\[ M_U = 0.87 \times F_y \times A_{st} \times d \times (1 - (A_{st} \times F_y) / (b \times d \times F_{ck})) \]

(Annex G-1.1.c of IS 456:2000)

\[ A_{st} = 419.41 \text{ mm}^2 \]

Provide 4 bars of 12mm φ

For mid span moment:
\[ M_U = 0.87 \times F_y \times A_{st} \times d \times (1 - (A_{st} \times F_y) / (b \times d \times F_{ck})) \]

\[ (58.81 \times 10^6) = 0.87 \times 415 \times A_{st} \times 475 \times 1 - (415 \times A_{st}) / (250 \times 30 \times 475) \]
\[ A_{st} = 357.83 \text{ mm}^2 \]

Provide 4 bars of 12mm \( \phi \).

**Step 5: Check for shear**

\[ \tau_v = \frac{V_u}{b\times d} \]

\[ \tau_v = \frac{87.3 \times 10^3}{250 \times 475} \]

\[ P_t = \frac{(A_{st})}{(b\times d)} \times 100 = 0.35 \] (Refer Table 19 of IS 456)

\[ \tau_c = 0.442 \]

\[ \tau_v > \tau_c \]

Hence shear reinforcement are required.

Use 8mm dia of 2 legged stirrups of 300mm spacing.

**Step 6: Check for Deflection**

\[ (L/d)_{max} = (L/d)_{basic} \times k_i \times k_c \times k_t \] (From Figures 4,5,6 of IS 456)

\[ (L/d)_{max} = 10 \times 1.9 \times 1 \times 1 = 19 \text{ mm} \]

\[ (L/d)_{actual} = \frac{4550}{475} \]

Hence it is safe.

**Design of Column**

**Step 1: Data**

Effective length, \( L_{eff} \) \quad = 3 + 0.25 = 3.25 \text{ m} \]

Load, \( P \) \quad = 932 \text{ KN}
Assume breadth, \( b \) = 250 mm.

Percentage of steel = 2

Concrete grade, \( f_{ck} \) (assuming) = 30 N/mm\(^2\)

Steel grade, \( f_y \) (assuming) = 415 N/mm\(^2\)

Design load, \( P_U \) = 932 \times 1.5 = 1398 KN

Slenderness Ratio \( (\lambda) \) = \( \frac{L_{eff}}{b} = 7.8 < 12 \)

Hence the column is short column.

**Step 2: Size of the column**

Area of the steel reinforcement \( (A_{sc}) \) = 0.02 \( A_g \)

Area of concrete, \( A_c \) = \( A_g - A_{sc} = 0.98 A_g \)

Let us assume column is subjected to axial member

\[ P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc} \] (Clause 39.3 of IS 456)

\[ A_g = 80711.27 \text{ mm}^2 \]

Breadth of column, \( b = 250 \text{ mm} \)

Depth of column, \( D = 322.8 \sim 350 \text{ mm} \)

Hence size of the column = 250 \times 350

**Step 3: Longitudinal reinforcement**

Area of steel reinforcement, \( A_{sc reqd} = 0.02 A_{g reqd} = 1614.22 \text{ mm}^2 \)

Provide 6 number of 20 mm \( \varnothing \) bar and 2 number of 16 mm \( \varnothing \) bar as the main (Longitudinal reinforcement).
Step 4: Transverse reinforcement

Minimum permitted pitch (Take least value) (Clause 26.5.3.2.d of IS 456)

\( \phi/4 = 20/4 = 5\text{mm} \)

6mm
Say 8mm

Maximum permitted pitch (Take least value) (Clause 26.5.3.2.b of IS 456)

Least lateral dimension of columns = 250mm
16 \times \text{dia of smallest longitudinal bar} = 16 \times 16 = 256mm
300 mm
Say 255mm

Provide 8mm of laterals at 255 mm c/c.

Design of Footing

Step 1: Data

Load carried by the column (P) \( = 932 \text{ KN} \) or 940 KN

Concrete grade, \( f_{ck} \) (assuming) \( = 30 \text{ N/mm}^2 \)

Steel grade, \( f_y \) (assuming) \( = 415 \text{ N/mm}^2 \)

Assume the self weight of the footing as 10% of column load

Total load on soil \( = 1030 \text{ KN} \)

Assume safe bearing capacity of soil \( = 120 \text{ KN/m}^2 \)

Area of footing required \( = 1030/120 \text{ m}^2 \)
\[ = 8.58 \text{ m}^2 \]
Step 2: Side Ratio

Side ratio of column, L/B  = 350/250 = 1.4

For footing, L  = 1.4 B

Area of footing  = 8.58

Breadth of footing, B  = 2.5

Length of footing, L  = 3.8

Provide a footing of 2.5m × 3.8m

Net upward design pressure at base  = 148.42 KN/m²

Step 3: Bending Moment

\[ M_u = 0.87 \times F_y \times A_{st} \times d \times (1 - (A_{st} \times F_y)/(b \times d \times f_{ck})) \]

(Annex G-1.1.c of IS 456:2000)

In long direction, \( M_{x_1} \)  = 529.97 KNm

In short direction, \( M_{x_2} \)  = 338.12 KNm

Max B.M  = 529.97 Nmm

Effective depth required, \( d \)  = 248 mm

Step 4: Tension Reinforcement

Long direction, \( M_{x_1} \)  = 529.97 KNm

\[ M_{x_1} = 0.87 \times F_y \times A_{st} \times d \times (1 - (A_{st} \times F_y)/(b \times d \times f_{ck})) \]

\( A_{st} \)  = 3065.73 mm²

\( A_{st(min)} = 0.12\% BD \)  = 1674 mm²
Provide 13 no of 16 mm $\varnothing$ bar in long direction at uniform spacing

$$M_{x2} = 0.87 \times F_y \times A_{st} \times d \times (1 - (A_{st} \times F_y) / (b \times d \times F_{ck}))$$

$$A_{st} = 1971.9 \text{mm}^2$$

Provide 10 no of 16 mm $\varnothing$ bar at central band and 4 no of 16 mm $\varnothing$ bar two each at outer portion in shorter direction.

**Step 5: Check for Development Length**

$$L_d = \varnothing \sigma st / 4 \tau bd = 410.71 \text{mm}$$

Allowing an end cover of 40 mm for the bars straight length of bar beyond the face of column in ling direction.

$$= 1725 - 50$$

$$= 1675 \text{mm} > 410 \text{mm}$$

Straight length of bar in short direction

$$= 1125 - 40$$

$$= 1085 \text{mm} > 410 \text{mm}$$

Hence Hooks are not necessary at the end of bars.

**Step 6: Check for Transverse Shear**

The critical section for transverse shear will be at a distance of 484 mm from the faces of the column.

Transverse shear in long direction,

$$V_{y1} = (1.725 - 0.484) \times 2.5 \times 148.42 = 460.47 \text{KN}$$
Nominal shear stress in long direction, \( \tau_{v1} = \frac{(460.73 \times 10^3)}{2500 \times 484} = 0.38 \) N/mm\(^2\)

Transverse shear in short direction,
\( V_{y1} = (1.125 - 0.484) \times 3.8 \times 148.42 = 361.52 \text{ KN} \)

Nominal shear stress in short direction, \( \tau_{v2} = \frac{(361.52 \times 10^3)}{3600 \times 484} = 0.19\text{N/mm}^2 \leq 0.38 \text{ N/mm}^2 \)

Hence it is safe.

**Step 7: Check for Punching Shear**

The critical section for punching shear is at a distance of 242 mm from the faces of the column around.

Nominal shear stress across \( zz \),
\( \tau V_z = \frac{1319.13 \times 10^3}{2(834 + 734) \times 484} = 0.869 \text{ N/mm}^2 \)

Permissible shear stress in concrete \( = K_s \tau_{cz} \approx 1 \)

\( \tau_v < K \tau_c \)

Hence safe.

**Step 8: Check for safe bearing capacity of soil**

Column load \( = 940 \text{ KN} \)

Weight of footing \( = 2.5 \times 3.8 \times 0.558 \times 25 \)

\( = 132.525 \text{ KN} \)

Total load on soil \( = 940 + 133 = 1073 \text{ KN} \)

Pressure on soil \( = \frac{1073}{(2.5 \times 3.8)} = 112.94 < 120 \)

Hence it is safe.
A-2 DESIGN OF THE UNDER GROUND BUNKER STRUCTURE

(i) **Design of Roof Slab**

The roof slab is divided into three types of slab panels based on their span and edge support conditions.

S1-Interior Panel
S2-One edge discontinuous
S3-Two adjacent discontinuous

Design of Slab Panel S1-Interior Panel

The following are the data to design the roof slab,

Size of the slab panel: 10m x 10m (Interior Panel)

Characteristic strength of concrete, $f_c = 25 \text{ N/mm}^2$

Yield strength of steel, $f_y = 415 \text{ N/mm}^2$

Effective spans, $\{l_x = 10 \text{ m}\}
\{l_y = 10 \text{ m}\}$

Aspect ratio $= l_y/l_x = 10/10 = 1 < 2$

Since the aspect ratio is less than 2, the slab should be designed as two way slab with provision for torsion at corners.

Assumed overall depth of the slab, $D = 300\text{mm}$

Diameter of bar, $\varnothing = 12\text{mm}$

Clear cover, $cc = 20\text{mm}$

Effective depth, $d = D - cc - \frac{\varnothing}{2} = 300 - 20 - 6 = 274 \text{ mm}$

**Step 1: Calculation of Loads**

Self weight of slab $= 0.3 \times 25 = 7.5 \text{ kN/m}^2$
Dead Load = 3.75 kN/m²

Live Load = 6 kN/m²

Total Load = 17.25 kN/m²

Total Design Load, \( w_u \) = 1.5 * 17.25 kN/m² = 25.875 kN/m²

Step 2: Ultimate Design Moments (using IS-456, 2000 moment efficient for restrained slab, page no 90&91)

\[ M_{ux}(-ve) = \alpha_x(-ve)w_u l_x^2 = 82.80 \times 10^6 \text{Nmm} \]

\[ M_{ux}(+ve) = \alpha_x(+ve)w_u l_x^2 = 0.0240 \times 25.875 \times 10^2 = 62.10 \times 10^6 \text{Nmm} \]

\[ M_{uy}(-ve) = \alpha_x(-ve)w_u l_x^2 = 0.0320 \times 25.875 \times 10^2 = 82.80 \times 10^6 \text{Nmm} \]

\[ M_{uy}(+ve) = \alpha_x(+ve)w_u l_x^2 = 0.0240 \times 25.875 \times 10^2 = 62.10 \times 10^6 \text{Nmm} \]

Step 3: Check for Depth

Maximum bending moment = 82.80 \times 10^6 \text{Nmm}

For Balanced section

(Using IS-456: 2000, page no 96)

\[ M_{ultim} = 0.36 \frac{x_{ulmax}}{d} \left(1 - 0.42 \frac{x_{ulmax}}{d}\right)bd^2f_{ck} \]

Effective depth required, \( d = 200 \text{mm} \) < effective depth provided, \( d = 274 \text{mm} \).

Hence, the effective depth selected is sufficient to resist the ultimate design bending moment.

Step 4: Design of Reinforcement

Area of tension reinforcement and spacing required

(using IS456: 2000 page no 96)
\[ A_{st} = \frac{bdf_{ck}}{2f_y} \left[ 1 - \sqrt{1 - \frac{4.598M_{ul}}{f_{ck}bd^2}} \right] \]

\[ S = \frac{\text{Area of one bar}}{\text{Area of tension reinforcement}} \times 1000 \]

\[ A_{st}(-ve) = 885\text{mm}^2, S(-ve) = 128\text{mm} \]

\[ A_{st}(+ve) = 654\text{mm}^2, S(+ve) = 173\text{mm} \]

\[ A_{st}(-ve) = 931\text{mm}^2, S(-ve) = 122\text{mm} \]

\[ A_{st}(+ve) = 687\text{mm}^2, S(+ve) = 165\text{mm} \]

Maximum spacing permitted = 3d = 3 \times 274 = 822\text{mm}, but < 300\text{mm}

Provide 12Ø @ 120 c/c in both directions.

Step 5: Check for Deflection

Percentage of tension reinforcement at mid span = 0.28

(Using IS-456: 2000 page no 38)

\[ f_s = 0.58f_y \frac{\text{Area of steel required}}{\text{Area of steel provided}} = 161 \text{ N/mm}^2 \]

(Using IS-456: 2000, page no 37)

Basic span/effective depth ratio for continuous slabs = 26

Modification factor = 2

Effective depth required = 194\text{mm} < 274\text{mm}

The section is safe for deflection.
Step 6:  Design of Slab Panel S2 and S3

Slab Panels S2 and S3 are designed as similar to the panel S1 by using IS456:2000 code.

(ii)  Design of Beam

All the beams are designed as simply supported beam which is having zero bending moment at the ends and maximum at the mid span. The following data are used to design the beam of the bunker.

Size of the beam: 450mm x 900mm x 10000mm

Characteristic strength of concrete, \( f_{ck} = 25 \ N/mm^2 \)

Yield strength of steel, \( f_y = 415 \ N/mm^2 \)

Effective span, \( l = 10m \)

Assumed overall depth of the slab, \( D = 900mm \)

Diameter of bar, \( \varphi = 25mm \)

Clear cover, \( cc = 25mm \)

Effective depth, \( d = D - cc - \frac{\varphi}{2} \)

\[ = 900-25-12.5 \]
\[ = 862.5mm \]

Step 1:  Load Calculations

Self weight of beam \( = 0.45*0.9*25 \)

\( = 10.125 \ kN/m \)

Total Load from slab \( = 2 * w \# \frac{3}{3} = 2 * 17.25 * 10 / 3 = 115 \ kN/m \)

Total Load \( = 125.125 \ kN/m \)

Total Design Load, \( w_{tu} = 1.5 * 125.125 \ kN/m = 187.6875 \ kN/m \)
Step 2: Ultimate Design Moment

\[ M_u = w_u l^2 / 8 = 187.6875 \times 10^2 / 8 = 2346.09 \times 10^6 N mm \]

Step 3: Design of Reinforcement (Using SP:16, Table 51)

- Flexural reinforcement:

\[ k = \frac{M_u}{bd^2} = 7.01, \quad \frac{d'}{d} = 0.05 \]

Percentage tension reinforcement, \( p_e = 2.084\% \)

Percentage compression reinforcement, \( p_c = 0.93\% \)

Area of tension steel required, \( A_{st} = 8088.525 \ mm^2 \)

No of bars required = 17

No of bars provided = 18

Area of tension steel provided, \( A_{st} = 8381.25 \ mm^2 \)

Area of compression steel required, \( A_{sc} = 3621.21 mm^2 \)

No of bars required = 7.34

No of bars provided = 8

Area of tension steel provided, \( A_{sc} = 3925 mm^2 \)

- Shear reinforcement:

Maximum shear force, \( V_u = \frac{w_u l}{2} = 187.6875 \times \frac{10}{2} \)

= 938.437 kN

Nominal shear stress, \( \tau_v = 2.42 \ N/mm^2 \)

(Using IS- 456: 2000, page no 73)
Design concrete shear strength, \( \tau_c = 0.83 \, \text{N/mm}^2 \)

Provide 4-legged 10mm diameter stirrups at 300mm centre to centre.

- Side face reinforcement:

Provide side face reinforcement 2-12mm diameter bars on each side of the beam.

(iii) **Design of Column**

Columns are designed as short axially loaded columns and also checked for uniaxial and biaxial moments. The following data are used to design the columns of the bunker.

Size of the column: 600mm x 600mm

Characteristic strength of concrete, \( f_{ck} = 25 \, \text{N/mm}^2 \)

Yield strength of steel, \( f_y = 415 \, \text{N/mm}^2 \)

Effective length, \( l = 3\text{m} \)

Step 1: Load Calculations

Total design load from beams = \( 4 \times 940 = 3760 \, \text{kN} \)

Step 2: Design of Reinforcement (Using IS- 456: 2000)

Assume percentage of reinforcement, \( p = 1\% \)

Gross Area, \( A = 600 \times 600 = 360000 \, \text{mm}^2 \)

Area of steel, \( A_{sc} = \frac{1}{180} \times 360000 = 3600 \, \text{mm}^2 \)

Area of concrete, \( A_c = 360000 - 3600 = 356400 \, \text{mm}^2 \)
Axial Load capacity, assuming column as axial load member

(Using IS- 456: 2000, page no 71)

\[ P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc} \]

\[ = 4564980 \text{ N} \]

\[ = 4564.98 kN > 3760 kN \]

Hence, the column is safe for the axial load.

Provide 8-25mm diameter bars and 8-16mm diameter bars.

Step 3: Lateral Ties (refer IS 456, 2000 pg no.49)

Provide lateral ties of 4-legged 10mm diameter bars at 200 centre to centre.

(iv) Design of Retaining Wall

Step 1: Data used

Unit weight of concrete = 25 kN/m\(^3\)

Characteristic strength of concrete, \( f_{ck} = 25 \text{ N/mm}^2 \)

Yield strength of steel, \( f_y = 415 \text{ N/mm}^2 \)

Surcharge on retaining wall = 9 kN/m\(^2\)

Angle of repose, \( \phi = 30^0 \)

Coefficient of friction between soil & concrete, \( \mu = 0.5 \)
Safe bearing capacity of the soil, $SBC = 150 \text{ kN/m}^2$

Unit weight of soil $= 18 \text{ kN/m}^3$

Total height of the wall $= 3.25m$

Base width, $B = 2.0m$

Resultant active pressure on the wall $= 41.46 \text{ kN/m}^2$

Heel Projection $= 1.35m$

Toe Projection $= 0.35m$

Thickness of stem, heel and toe $= 300 \text{mm}$

Resultant Weight, $W = 121 \text{kN}$

Resultant moment, $M = 155 \text{kNm}$

Eccentricity, $e = z - \frac{B}{2} = 0.28m$

Pressure distribution at base

$$p_{\text{max}} = \frac{W}{B} \left(1 + \frac{6e}{B}\right) = 111.32 \text{ kN/m}^2 < 150 \text{ kN/m}^2$$

$$p_{\text{min}} = \frac{W}{B} \left(1 + \frac{6e}{B}\right) = 9.45 \text{ kN/m}^2 < 150 \text{ kN/m}^2$$

Step 2: Design of Stem of Retaining Wall

Factored bending moment at the bottom of the stem, $M_{tu} = 5752 * 10^6 \text{Nm}$

Factored shear force at the bottom of the stem, $V_{tu} = 56.45 \text{kN}$

For Balanced section,
Effective depth required, \( d = 144.38 \text{mm} < \text{effective depth provided}, d = 250 \text{mm}. \)

Hence, the effective depth selected is sufficient to resist the ultimate design bending moment.

Provide thickness of stem, \( D = 300 \text{mm} \)

Assume the diameter of reinforcement as 16mm

Area of tension reinforcement and spacing required,

\[
A_{st} = \frac{bd f_{ck}}{2f_y} \left[ 1 - \sqrt{1 - \frac{4.598 M_{ut}}{f_{ck} bd^2}} \right] = 675.15 \text{mm}^2
\]

\[
S = \frac{\text{Area of one bar}}{\text{Area of tension reinforcement}} \times 1000 = 297.65 \text{mm}
\]

Provide 16mm diameter bar at 250mm c/c.

- Check for Shear:

Nominal shear stress, \( \tau_v = 0.23 \text{ N/mm}^2 \)

(Using IS-456: 2000, page no 73)

Design concrete shear strength, \( \tau_c = 0.38 \text{ N/mm}^2 \)

Section is safe for shear.

Step 3: Design of Heel
Factored bending moment at the edge of the stem, $M_u = 96.63 \times 10^6 Nmm$

Factored shear force at the bottom of the stem, $V_u = 34.70 kN$

For Balanced section,

(Using IS- 456: 2000, page no 96)

$$M_{u,lim} = 0.36 \frac{x_{u,max}}{d} \left(1 - 0.42 \frac{x_{u,max}}{d}\right) b d^2 f_{ck}$$

Effective depth required, $d = 187.14 < $ effective depth provided, $d = 250 mm$.

Hence, the effective depth selected is sufficient to resist the ultimate design bending moment.

Provide thickness of stem, $D = 300 mm$

Assume the diameter of reinforcement as 16mm.

Area of tension reinforcement and spacing required,

$$A_{st} = \frac{b d f_{ck}}{2 f_y} \left[1 - \sqrt{1 - \frac{4.598 M_u}{f_{ck} b d^2}}\right] = 1187.74 mm^2$$

$$S = \frac{Area \ of \ one \ bar}{Area \ of \ tension \ reinforcement} \times 1000 = 169.19 mm$$

Provide 16mm diameter bar at 150mm c/c.

- Check for Shear:

  Nominal shear stress, $\tau_v = 0.21 \ N/mm^2$
(Using IS-456: 2000, page no 73)

Design concrete shear strength, $\tau_c = 0.5 \ N/mm^2$ (from SP16)

Section is safe for shear.

Step 4: Design of Toe

Factored bending moment at the edge of the stem, $M_{ul} = 77.14 \times 10^6 N/mm$

Factored shear force at the bottom of the stem, $V_{ul} = 99.95 kN$

For Balanced section, (Using IS-456: 2000, page no 96)

$$M_{ul,lim} = 0.36 \frac{X_{\mu,\text{max}}}{d} \left( 1 - 0.42 \frac{X_{\mu,\text{max}}}{d} \right) bd^2 f_{ck}$$

Effective depth required, $d = 167.2 < \text{effective depth provided, } d = 250 mm$.

Hence, the effective depth selected is sufficient to resist the ultimate design bending moment.

Provide thickness of stem, $D = 300 mm$

Assume the diameter of reinforcement as 16mm.

Area of tension reinforcement and spacing required,

$$A_{st} = \frac{bd f_{ck}}{2 f_y} \left[ 1 - \sqrt{1 - \frac{4.598 M_{ul}}{f_{ck} bd^2}} \right] = 925.82 mm^2$$

$$S = \frac{\text{Area of one bar}}{\text{Area of tension reinforcement}} \times 1000 = 217.6 mm$$
Provide 16mm diameter bar at 150mm c/c.

- Check for Shear:

  Nominal shear stress, \( \tau_n = 0.4 \ N/mm^2 \)

  (Using IS- 456: 2000, page no 73)

  Design concrete shear strength, \( \tau_c = 0.42 \ N/mm^2 \)

  Section is safe for shear.

The plan, elevation and reinforcement details and drawings were generated using AUTO CAD software and presented in chapter 5, Results and Discussion. The cost estimation for the bunkers were calculated based on the square area method and cubic volume methods with 2011 year schedule of rates (INR) issued by Public Works Department of Tamilnadu, India.
## APPENDIX 2

### Table A2.1 Detailed Estimation for Underground Bunker

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<th>D rft</th>
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**Table A 2.2 Abstract Estimate for the Bunker**
**Table A 2.3 Abstract Estimation for G+2 Storeyed Building**

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**Table A2.4 Abstract Estimation for Stilt +5 Storeyed Building with 4 Blocks – A, B, C&D**

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## BLOCK - C

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**BLOCK - D**

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**Total Cost**

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