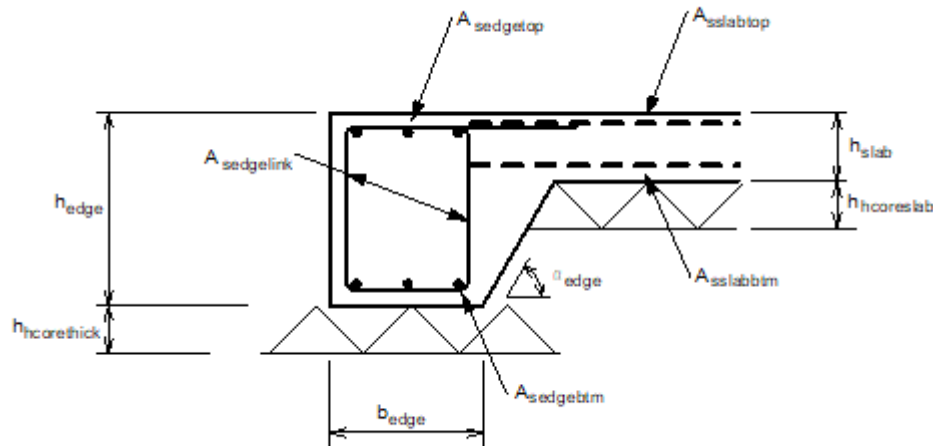


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RAFT FOUNDATION DESIGN (BS8110 : PART 1 : 1997)

Tedds calculation version 1.0.12



Soil and raft definition

Soil definition

Allowable bearing pressure
Number of types of soil forming sub-soil
Soil density
Depth of hardcore beneath slab
Depth of hardcore beneath thickenings
Density of hardcore
Basic assumed diameter of local depression
Diameter under slab modified for hardcore
Diameter under thickenings modified for hardcore

$$q_{\text{allow}} = 125.0 \text{ kN/m}^2$$

Two or more types

Firm to loose

$$h_{\text{coreslab}} = 400 \text{ mm (Dispersal allowed for bearing pressure check)}$$

$$h_{\text{corethick}} = 400 \text{ mm (Dispersal allowed for bearing pressure check)}$$

$$\gamma_{\text{hcore}} = 20.0 \text{ kN/m}^3$$

$$\phi_{\text{depbasic}} = 3500 \text{ mm}$$

$$\phi_{\text{dep slab}} = \phi_{\text{depbasic}} - h_{\text{coreslab}} = 3100 \text{ mm}$$

$$\phi_{\text{dep thick}} = \phi_{\text{depbasic}} - h_{\text{corethick}} = 3100 \text{ mm}$$

Raft slab definition

Max dimension/max dimension between joints
Slab thickness
Concrete strength
Poissons ratio of concrete
Slab mesh reinforcement strength
Partial safety factor for steel reinforcement
From C&CA document 'Concrete ground floors' Table 5

$$l_{\text{max}} = 14.750 \text{ m}$$

$$h_{\text{slab}} = 250 \text{ mm}$$

$$f_{\text{cu}} = 35 \text{ N/mm}^2$$

$$\nu = 0.2$$

$$f_{\text{yslab}} = 500 \text{ N/mm}^2$$

$$\gamma_s = 1.15$$

Minimum mesh required in top for shrinkage

A193

Actual mesh provided in top

$$\mathbf{A393 (A_{\text{sslabtop}} = 393 \text{ mm}^2/\text{m})}$$

Mesh provided in bottom

$$\mathbf{A393 (A_{\text{sslabbtm}} = 393 \text{ mm}^2/\text{m})}$$

Top mesh bar diameter

$$\phi_{\text{slabtop}} = 10 \text{ mm}$$

Bottom mesh bar diameter

$$\phi_{\text{slabbtm}} = 10 \text{ mm}$$

Cover to top reinforcement

$$C_{\text{top}} = 40 \text{ mm}$$

Cover to bottom reinforcement

$$C_{\text{btm}} = 40 \text{ mm}$$

Average effective depth of top reinforcement

$$d_{\text{tslabav}} = h_{\text{slab}} - C_{\text{top}} - \phi_{\text{slabtop}} = 200 \text{ mm}$$

Average effective depth of bottom reinforcement

$$d_{\text{bslabav}} = h_{\text{slab}} - C_{\text{btm}} - \phi_{\text{slabbtm}} = 200 \text{ mm}$$

Overall average effective depth

$$d_{\text{slabav}} = (d_{\text{tslabav}} + d_{\text{bslabav}})/2 = 200 \text{ mm}$$

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Minimum effective depth of top reinforcement

$$d_{tslabmin} = d_{tslabav} - \phi_{slabtop}/2 = \mathbf{195 \text{ mm}}$$

Minimum effective depth of bottom reinforcement

$$d_{bslabmin} = d_{bslabav} - \phi_{slabbtm}/2 = \mathbf{195 \text{ mm}}$$

Edge beam definition

Overall depth

$$h_{edge} = \mathbf{400 \text{ mm}}$$

Width

$$b_{edge} = \mathbf{400 \text{ mm}}$$

Angle of chamfer to horizontal

$$\alpha_{edge} = \mathbf{60 \text{ deg}}$$

Strength of main bar reinforcement

$$f_y = \mathbf{500 \text{ N/mm}^2}$$

Strength of link reinforcement

$$f_{ys} = \mathbf{500 \text{ N/mm}^2}$$

Reinforcement provided in top

$$\mathbf{6 \text{ H25 bars } (A_{sedge\text{top}} = 2945 \text{ mm}^2)}$$

Reinforcement provided in bottom

$$\mathbf{3 \text{ H25 bars } (A_{sedge\text{btm}} = 1473 \text{ mm}^2)}$$

Link reinforcement provided

$$\mathbf{2 \text{ H20 legs at 250 ctrs } (A_{sv}/s_v = 2.513 \text{ mm})}$$

Bottom cover to links

$$C_{beam} = \mathbf{40 \text{ mm}}$$

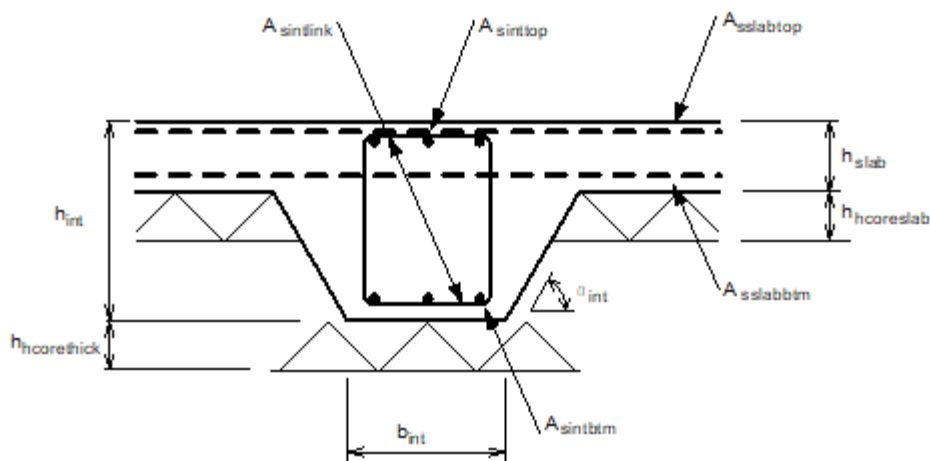
Effective depth of top reinforcement

$$d_{edgetop} = h_{edge} - C_{top} - \phi_{slabtop} - \phi_{edgeline} - \phi_{edgetop}/2 = \mathbf{318 \text{ mm}}$$

Effective depth of bottom reinforcement

$$d_{edgebtm} = h_{edge} - C_{beam} - \phi_{edgeline} - \phi_{edgebtm}/2 = \mathbf{328 \text{ mm}}$$

Internal beam definition



Overall depth

$$h_{int} = \mathbf{400 \text{ mm}}$$

Width

$$b_{int} = \mathbf{400 \text{ mm}}$$

Angle of chamfer to horizontal

$$\alpha_{int} = \mathbf{60 \text{ deg}}$$

Strength of main bar reinforcement

$$f_y = \mathbf{500 \text{ N/mm}^2}$$

Strength of link reinforcement

$$f_{ys} = \mathbf{500 \text{ N/mm}^2}$$

Reinforcement provided in top

$$\mathbf{6 \text{ H20 bars } (A_{sint\text{top}} = 1885 \text{ mm}^2)}$$

Reinforcement provided in bottom

$$\mathbf{5 \text{ H20 bars } (A_{sint\text{btm}} = 1571 \text{ mm}^2)}$$

Link reinforcement provided

$$\mathbf{2 \text{ H16 legs at 325 ctrs } (A_{sv}/s_v = 1.237 \text{ mm})}$$

Effective depth of top reinforcement

$$d_{inttop} = h_{int} - C_{top} - 2 \times \phi_{slabtop} - \phi_{inttop}/2 = \mathbf{330 \text{ mm}}$$

Effective depth of bottom reinforcement

$$d_{intbtm} = h_{int} - C_{beam} - \phi_{intlink} - \phi_{intbtm}/2 = \mathbf{334 \text{ mm}}$$

Internal slab design checks

Basic loading

Slab self weight

$$W_{slab} = 24 \text{ kN/m}^3 \times h_{slab} = \mathbf{6.0 \text{ kN/m}^2}$$

Hardcore

$$W_{hcoreslab} = \gamma_{hcore} \times h_{hcoreslab} = \mathbf{8.0 \text{ kN/m}^2}$$

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Applied loading

Uniformly distributed dead load

$$W_{Dudl} = 24.7 \text{ kN/m}^2$$

Uniformly distributed live load

$$W_{Ludl} = 5.6 \text{ kN/m}^2$$

Internal slab bearing pressure check

Total uniform load at formation level

$$W_{udl} = W_{slab} + W_{hcoreslab} + W_{Dudl} + W_{Ludl} = 44.3 \text{ kN/m}^2$$

PASS - $W_{udl} \leq q_{allow}$ - Applied bearing pressure is less than allowable

Internal slab bending and shear check

Applied bending moments

Span of slab

$$l_{slab} = \phi_{depslab} + d_{tslabav} = 3300 \text{ mm}$$

Ultimate self weight udl

$$W_{swult} = 1.4 \times W_{slab} = 8.4 \text{ kN/m}^2$$

Self weight moment at centre

$$M_{csw} = W_{swult} \times l_{slab}^2 \times (1 + \nu) / 64 = 1.7 \text{ kNm/m}$$

Self weight moment at edge

$$M_{esw} = W_{swult} \times l_{slab}^2 / 32 = 2.9 \text{ kNm/m}$$

Self weight shear force at edge

$$V_{sw} = W_{swult} \times l_{slab} / 4 = 6.9 \text{ kN/m}$$

Moments due to applied uniformly distributed loads

Ultimate applied udl

$$W_{udult} = 1.4 \times W_{Dudl} + 1.6 \times W_{Ludl} = 43.6 \text{ kN/m}^2$$

Moment at centre

$$M_{cudl} = W_{udult} \times l_{slab}^2 \times (1 + \nu) / 64 = 8.9 \text{ kNm/m}$$

Moment at edge

$$M_{eudl} = W_{udult} \times l_{slab}^2 / 32 = 14.8 \text{ kNm/m}$$

Shear force at edge

$$V_{udl} = W_{udult} \times l_{slab} / 4 = 36.0 \text{ kN/m}$$

Resultant moments and shears

Total moment at edge

$$M_{\Sigma e} = 17.7 \text{ kNm/m}$$

Total moment at centre

$$M_{\Sigma c} = 10.6 \text{ kNm/m}$$

Total shear force

$$V_{\Sigma} = 42.9 \text{ kN/m}$$

Reinforcement required in top

K factor

$$K_{slabtop} = M_{\Sigma e} / (f_{cu} \times d_{tslabav}^2) = 0.013$$

Lever arm

$$Z_{slabtop} = d_{tslabav} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabtop}/0.9)}) = 190.0 \text{ mm}$$

Area of steel required for bending

$$A_{sslabtopbend} = M_{\Sigma e} / ((1.0/\gamma_s) \times f_{yslab} \times Z_{slabtop}) = 214 \text{ mm}^2/\text{m}$$

Minimum area of steel required

$$A_{sslabmin} = 0.0013 \times h_{slab} = 325 \text{ mm}^2/\text{m}$$

Area of steel required

$$A_{sslabtopreq} = \max(A_{sslabtopbend}, A_{sslabmin}) = 325 \text{ mm}^2/\text{m}$$

PASS - $A_{sslabtopreq} \leq A_{sslabtop}$ - Area of reinforcement provided in top to span local depressions is adequate

Reinforcement required in bottom

K factor

$$K_{slabbtm} = M_{\Sigma c} / (f_{cu} \times d_{bslabav}^2) = 0.008$$

Lever arm

$$Z_{slabbtm} = d_{bslabav} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabbtm}/0.9)}) = 190.0 \text{ mm}$$

Area of steel required for bending

$$A_{sslabbtmbend} = M_{\Sigma c} / ((1.0/\gamma_s) \times f_{yslab} \times Z_{slabbtm}) = 129 \text{ mm}^2/\text{m}$$

Area of steel required

$$A_{sslabbtmreq} = \max(A_{sslabbtmbend}, A_{sslabmin}) = 325 \text{ mm}^2/\text{m}$$

PASS - $A_{sslabbtmreq} \leq A_{sslabbtm}$ - Area of reinforcement provided in bottom to span local depressions is adequate

Shear check

Applied shear stress

$$v = V_{\Sigma} / d_{tslabmin} = 0.220 \text{ N/mm}^2$$

Tension steel ratio

$$\rho = 100 \times A_{sslabtop} / d_{tslabmin} = 0.202$$

From BS8110-1:1997 - Table 3.8

Design concrete shear strength

$$v_c = 0.496 \text{ N/mm}^2$$

PASS - $v \leq v_c$ - Shear capacity of the slab is adequate

Internal slab deflection check

Basic allowable span to depth ratio

$$\text{Ratio}_{basic} = 26.0$$

Moment factor

$$M_{factor} = M_{\Sigma c} / d_{bslabav}^2 = 0.265 \text{ N/mm}^2$$

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Steel service stress

$$f_s = 2/3 \times f_{yslab} \times A_{sslabtmbend}/A_{sslabbtm} = \mathbf{109.031 \text{ N/mm}^2}$$

Modification factor

$$MF_{slab} = \min(2.0, 0.55 + [(477\text{N/mm}^2 - f_s)/(120 \times (0.9\text{N/mm}^2 + M_{factor}))])$$

$$MF_{slab} = \mathbf{2.000}$$

Modified allowable span to depth ratio

$$Ratio_{allow} = Ratio_{basic} \times MF_{slab} = \mathbf{52.000}$$

Actual span to depth ratio

$$Ratio_{actual} = l_{slab}/d_{slabav} = \mathbf{16.500}$$

PASS - Ratio_{actual} <= Ratio_{allow} - Slab span to depth ratio is adequate

Edge beam design checks

Basic loading

Hardcore

$$W_{hcorethick} = \gamma_{hcore} \times h_{hcorethick} = \mathbf{8.0 \text{ kN/m}^2}$$

Edge beam

Rectangular beam element

$$W_{beam} = 24 \text{ kN/m}^3 \times h_{edge} \times b_{edge} = \mathbf{3.8 \text{ kN/m}}$$

Chamfer element

$$W_{chamfer} = 24 \text{ kN/m}^3 \times (h_{edge} - h_{slab})^2/(2 \times \tan(\alpha_{edge})) = \mathbf{0.2 \text{ kN/m}}$$

Slab element

$$W_{slabelmt} = 24 \text{ kN/m}^3 \times h_{slab} \times (h_{edge} - h_{slab})/\tan(\alpha_{edge}) = \mathbf{0.5 \text{ kN/m}}$$

Edge beam self weight

$$W_{edge} = W_{beam} + W_{chamfer} + W_{slabelmt} = \mathbf{4.5 \text{ kN/m}}$$

Edge load number 1

Load type

Longitudinal line load

Dead load

$$W_{Dedge1} = \mathbf{58.7 \text{ kN/m}}$$

Live load

$$W_{Ledge1} = \mathbf{11.6 \text{ kN/m}}$$

Ultimate load

$$W_{ultedge1} = 1.4 \times W_{Dedge1} + 1.6 \times W_{Ledge1} = \mathbf{100.8 \text{ kN/m}}$$

Longitudinal line load width

$$b_{edge1} = \mathbf{300 \text{ mm}}$$

Centroid of load from outside face of raft

$$X_{edge1} = \mathbf{150 \text{ mm}}$$

Edge beam bearing pressure check

Effective bearing width of edge beam

$$b_{bearing} = b_{edge} + (h_{edge} - h_{slab})/\tan(\alpha_{edge}) = \mathbf{487 \text{ mm}}$$

Total uniform load at formation level

$$W_{udledge} = W_{Dudl} + W_{Ludl} + W_{edge}/b_{bearing} + W_{hcorethick} = \mathbf{47.6 \text{ kN/m}^2}$$

Centroid of longitudinal and equivalent line loads from outside face of raft

Load x distance for edge load 1

$$Moment_1 = W_{ultedge1} \times X_{edge1} = \mathbf{15.1 \text{ kN}}$$

Sum of ultimate longitud'l and equivalent line loads $\Sigma UDL = \mathbf{100.8 \text{ kN/m}}$

Sum of load x distances

$$\Sigma Moment = \mathbf{15.1 \text{ kN}}$$

Centroid of loads

$$X_{bar} = \Sigma Moment/\Sigma UDL = \mathbf{150 \text{ mm}}$$

Initially assume no moment transferred into slab due to load/reaction eccentricity

Sum of unfactored longitud'l and eff'tive line loads $\Sigma UDLsls = \mathbf{70.3 \text{ kN/m}}$

Allowable bearing width

$$b_{allow} = 2 \times X_{bar} + 2 \times h_{hcoreslab} \times \tan(30) = \mathbf{762 \text{ mm}}$$

Bearing pressure due to line/point loads

$$q_{linepoint} = \Sigma UDLsls/b_{allow} = \mathbf{92.3 \text{ kN/m}^2}$$

Total applied bearing pressure

$$q_{edge} = q_{linepoint} + W_{udledge} = \mathbf{140.0 \text{ kN/m}^2}$$

$q_{edge} > q_{allow}$ - The slab is required to resist a moment due to eccentricity

Now assume moment due to load/reaction eccentricity is resisted by slab

Bearing width required

$$b_{req} = \Sigma UDLsls/(q_{allow} - W_{udledge}) = \mathbf{909 \text{ mm}}$$

Effective bearing width at u/s of slab

$$b_{reqeff} = b_{req} - 2 \times h_{hcoreslab} \times \tan(30) = \mathbf{447 \text{ mm}}$$

Load/reaction eccentricity

$$e = b_{reqeff}/2 - X_{bar} = \mathbf{74 \text{ mm}}$$

Ultimate moment to be resisted by slab

$$M_{ecc} = \Sigma UDL \times e = \mathbf{7.4 \text{ kNm/m}}$$

From slab bending check

Moment due to depression under slab (hogging)

$$M_{\Sigma e} = \mathbf{17.7 \text{ kNm/m}}$$

Total moment to be resisted by slab top steel

$$M_{slabtop} = M_{ecc} + M_{\Sigma e} = \mathbf{25.1 \text{ kNm/m}}$$

K factor

$$K_{slab} = M_{slabtop}/(f_{cu} \times d_{slabmin}^2) = \mathbf{0.019}$$

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Lever arm

$$Z_{slab} = d_{slabmin} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slab}/0.9)}) = \mathbf{185 \text{ mm}}$$

Area of steel required

$$A_{slabreq} = M_{slabtop} / ((1.0/\gamma_s) \times f_y \times Z_{slab}) = \mathbf{312 \text{ mm}^2/\text{m}}$$

PASS - $A_{slabreq} \leq A_{slabtop}$ - Area of reinforcement provided to transfer moment into slab is adequate

The allowable bearing pressure under the edge beam will not be exceeded

Library item - B pressure with moment transfer

Edge beam bending check

Divider for moments due to udl's

$$\beta_{udl} = \mathbf{10.0}$$

Applied bending moments

Span of edge beam

$$l_{edge} = \phi_{depthick} + d_{edgetop} = \mathbf{3418 \text{ mm}}$$

Ultimate self weight udl

$$W_{edgeult} = 1.4 \times W_{edge} = \mathbf{6.3 \text{ kN/m}}$$

Ultimate slab udl (approx)

$$W_{edgeslab} = \max(0 \text{ kN/m}, 1.4 \times W_{slab} \times ((\phi_{depthick}/2 \times 3/4) - (b_{edge} + (h_{edge} - h_{slab})/\tan(\alpha_{edge}))))$$

$$W_{edgeslab} = \mathbf{5.7 \text{ kN/m}}$$

Self weight and slab bending moment

$$M_{edgesw} = (W_{edgeult} + W_{edgeslab}) \times l_{edge}^2 / \beta_{udl} = \mathbf{14.0 \text{ kNm}}$$

Self weight shear force

$$V_{edgesw} = (W_{edgeult} + W_{edgeslab}) \times l_{edge} / 2 = \mathbf{20.5 \text{ kN}}$$

Moments due to applied uniformly distributed loads

Ultimate udl (approx)

$$W_{edgeudl} = W_{udlult} \times \phi_{depthick} / 2 \times 3/4 = \mathbf{50.7 \text{ kN/m}}$$

Bending moment

$$M_{edgeudl} = W_{edgeudl} \times l_{edge}^2 / \beta_{udl} = \mathbf{59.2 \text{ kNm}}$$

Shear force

$$V_{edgeudl} = W_{edgeudl} \times l_{edge} / 2 = \mathbf{86.6 \text{ kN}}$$

Moment and shear due to load number 1

Bending moment

$$M_{edge1} = W_{ultedge1} \times l_{edge}^2 / \beta_{udl} = \mathbf{117.7 \text{ kNm}}$$

Shear force

$$V_{edge1} = W_{ultedge1} \times l_{edge} / 2 = \mathbf{172.3 \text{ kN}}$$

Resultant moments and shears

Total moment (hogging and sagging)

$$M_{\Sigma edge} = \mathbf{191.0 \text{ kNm}}$$

Maximum shear force

$$V_{\Sigma edge} = \mathbf{279.4 \text{ kN}}$$

Reinforcement required in top

Width of section in compression zone

$$b_{edgetop} = b_{edge} = \mathbf{400 \text{ mm}}$$

Average web width

$$b_w = b_{edge} + (h_{edge} / \tan(\alpha_{edge})) / 2 = \mathbf{515 \text{ mm}}$$

K factor

$$K_{edgetop} = M_{\Sigma edge} / (f_{cu} \times b_{edgetop} \times d_{edgetop}^2) = \mathbf{0.135}$$

Lever arm

$$Z_{edgetop} = d_{edgetop} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{edgetop}/0.9)}) = \mathbf{259 \text{ mm}}$$

Area of steel required for bending

$$A_{sedgetopbend} = M_{\Sigma edge} / ((1.0/\gamma_s) \times f_y \times Z_{edgetop}) = \mathbf{1696 \text{ mm}^2}$$

Minimum area of steel required

$$A_{sedgetopmin} = 0.0013 \times 1.0 \times b_w \times h_{edge} = \mathbf{268 \text{ mm}^2}$$

Area of steel required

$$A_{sedgetopreq} = \max(A_{sedgetopbend}, A_{sedgetopmin}) = \mathbf{1696 \text{ mm}^2}$$

PASS - $A_{sedgetopreq} \leq A_{sedgetop}$ - Area of reinforcement provided in top of edge beams is adequate

Reinforcement required in bottom

Width of section in compression zone

$$b_{edgebtm} = b_{edge} + (h_{edge} - h_{slab}) / \tan(\alpha_{edge}) + 0.1 \times l_{edge} = \mathbf{828 \text{ mm}}$$

K factor

$$K_{edgebtm} = M_{\Sigma edge} / (f_{cu} \times b_{edgebtm} \times d_{edgebtm}^2) = \mathbf{0.061}$$

Lever arm

$$Z_{edgebtm} = d_{edgebtm} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{edgebtm}/0.9)}) = \mathbf{303 \text{ mm}}$$

Area of steel required for bending

$$A_{sedgebtmbend} = M_{\Sigma edge} / ((1.0/\gamma_s) \times f_y \times Z_{edgebtm}) = \mathbf{1448 \text{ mm}^2}$$

Minimum area of steel required

$$A_{sedgebtmmin} = 0.0013 \times 1.0 \times b_w \times h_{edge} = \mathbf{268 \text{ mm}^2}$$

Area of steel required

$$A_{sedgebtmreq} = \max(A_{sedgebtmbend}, A_{sedgebtmmin}) = \mathbf{1448 \text{ mm}^2}$$

PASS - $A_{sedgebtmreq} \leq A_{sedgebtm}$ - Area of reinforcement provided in bottom of edge beams is adequate

Edge beam shear check

Applied shear stress

$$V_{edge} = V_{\Sigma edge} / (b_w \times d_{edgetop}) = \mathbf{1.707 \text{ N/mm}^2}$$

Tension steel ratio

$$\rho_{edge} = 100 \times A_{sedgetop} / (b_w \times d_{edgetop}) = \mathbf{1.800}$$

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From BS8110-1:1997 - Table 3.8

Design concrete shear strength

$$V_{edge} = 0.911 \text{ N/mm}^2$$

$$V_{edge} > V_{cedge} + 0.4 \text{ N/mm}^2 - \text{Therefore designed links required}$$

Link area to spacing ratio required

$$A_{sv_upon_Svreqedge} = (V_{edge} - V_{cedge}) \times b_w / ((1.0/\gamma_s) \times f_{ys}) = 0.944 \text{ mm}$$

Link area to spacing ratio provided

$$A_{sv_upon_Svprovedge} = N_{edgelink} \times \pi \times \phi_{edgelink}^2 / (4 \times S_{vedge}) = 2.513 \text{ mm}$$

$$PASS - A_{sv_upon_Svreqedge} \leq A_{sv_upon_Svprovedge} - \text{Shear reinforcement provided in edge beams is adequate}$$

Corner design checks

Basic loading

Corner load number 1

Load type

Line load in x direction

Dead load

$$W_{Dcorner1} = 58.7 \text{ kN/m}$$

Live load

$$W_{Lcorner1} = 11.6 \text{ kN/m}$$

Ultimate load

$$W_{ultcorner1} = 1.4 \times W_{Dcorner1} + 1.6 \times W_{Lcorner1} = 100.8 \text{ kN/m}$$

Centroid of load from outside face of raft

$$y_{corner1} = 150 \text{ mm}$$

Corner load number 2

Load type

Line load in y direction

Dead load

$$W_{Dcorner2} = 45.1 \text{ kN/m}$$

Live load

$$W_{Lcorner2} = 0.0 \text{ kN/m}$$

Ultimate load

$$W_{ultcorner2} = 1.4 \times W_{Dcorner2} + 1.6 \times W_{Lcorner2} = 63.1 \text{ kN/m}$$

Centroid of load from outside face of raft

$$x_{corner2} = 150 \text{ mm}$$

Corner bearing pressure check

Total uniform load at formation level

$$W_{udlcorner} = W_{Dudl} + W_{Ludl} + W_{edge}/b_{bearing} + W_{corethick} = 47.6 \text{ kN/m}^2$$

Net bearing press avail to resist line/point loads

$$q_{netcorner} = q_{allow} - W_{udlcorner} = 77.4 \text{ kN/m}^2$$

Total line/point loads

Total unfactored line load in x direction

$$W_{\Sigma linex} = 70.3 \text{ kN/m}$$

Total ultimate line load in x direction

$$W_{\Sigma ultlinex} = 100.8 \text{ kN/m}$$

Total unfactored line load in y direction

$$W_{\Sigma liney} = 45.1 \text{ kN/m}$$

Total ultimate line load in y direction

$$W_{\Sigma ultliney} = 63.1 \text{ kN/m}$$

Total unfactored point load

$$W_{\Sigma point} = 0.0 \text{ kN}$$

Total ultimate point load

$$W_{\Sigma ultpoint} = 0.0 \text{ kN}$$

Length of side of sq reqd to resist line/point loads

$$p_{corner} = [W_{\Sigma linex} + W_{\Sigma liney} + \sqrt{(W_{\Sigma linex} + W_{\Sigma liney})^2 + 4 \times q_{netcorner} \times W_{\Sigma point}}] / (2 \times q_{netcorner})$$

$$p_{corner} = 1491 \text{ mm}$$

Bending moment about x-axis due to load/reaction eccentricity

Moment due to load 1 (x line)

$$M_{x1} = \max(0 \text{ kNm}, W_{ultcorner1} \times p_{corner} \times (p_{corner}/2 - y_{corner1})) = 89.5 \text{ kNm}$$

Total moment about x axis

$$M_{\Sigma x} = 89.5 \text{ kNm}$$

Bending moment about y-axis due to load/reaction eccentricity

Moment due to load 2 (y line)

$$M_{y2} = \max(0 \text{ kNm}, W_{ultcorner2} \times p_{corner} \times (p_{corner}/2 - x_{corner2})) = 56.1 \text{ kNm}$$

Total moment about y axis

$$M_{\Sigma y} = 56.1 \text{ kNm}$$

Check top reinforcement in edge beams for load/reaction eccentric moment

Max moment due to load/reaction eccentricity

$$M_{\Sigma} = \max(M_{\Sigma x}, M_{\Sigma y}) = 89.5 \text{ kNm}$$

Assume all of this moment is resisted by edge beam

From edge beam design checks away from corners

Moment due to edge beam spanning depression

$$M_{\Sigma edge} = 191.0 \text{ kNm}$$

Total moment to be resisted

$$M_{\Sigma cornerbp} = M_{\Sigma} + M_{\Sigma edge} = 280.5 \text{ kNm}$$

Width of section in compression zone

$$b_{edgetop} = b_{edge} = 400 \text{ mm}$$

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K factor

$$K_{cornerbp} = M_{\Sigma cornerbp} / (f_{cu} \times b_{edgetop} \times d_{edgetop}^2) = \mathbf{0.199}$$

$K > 0.156$ - Therefore compression reinforcement is required
The design is outside the scope of this calculation

Corner beam bending check

Cantilever span of edge beam

$$l_{corner} = \phi_{depthick} / \sqrt{(2)} + d_{edgetop} / 2 = \mathbf{2351 \text{ mm}}$$

Moment and shear due to self weight

Ultimate self weight udl

$$W_{edgeult} = 1.4 \times W_{edge} = \mathbf{6.3 \text{ kN/m}}$$

Average ultimate slab udl (approx)

$$W_{cornerslab} = \max(0 \text{ kN/m}, 1.4 \times W_{slab} \times (\phi_{depthick} / (\sqrt{(2)} \times 2) - (b_{edge} + (h_{edge} - h_{slab}) / \tan(\alpha_{edge}))))$$

$$W_{cornerslab} = \mathbf{5.1 \text{ kN/m}}$$

Self weight and slab bending moment

$$M_{cornersw} = (W_{edgeult} + W_{cornerslab}) \times l_{corner}^2 / 2 = \mathbf{31.6 \text{ kNm}}$$

Self weight and slab shear force

$$V_{cornersw} = (W_{edgeult} + W_{cornerslab}) \times l_{corner} = \mathbf{26.9 \text{ kN}}$$

Moment and shear due to udl's

Maximum ultimate udl

$$W_{cornerudl} = ((1.4 \times W_{Dudl}) + (1.6 \times W_{Ludl})) \times \phi_{depthick} / \sqrt{(2)} = \mathbf{95.6 \text{ kN/m}}$$

Bending moment

$$M_{cornerudl} = W_{cornerudl} \times l_{corner}^2 / 6 = \mathbf{88.0 \text{ kNm}}$$

Shear force

$$V_{cornerudl} = W_{cornerudl} \times l_{corner} / 2 = \mathbf{112.4 \text{ kN}}$$

Moment and shear due to line loads in x direction

Bending moment

$$M_{cornerlinex} = W_{\Sigma ultlinex} \times l_{corner}^2 / 2 = \mathbf{278.4 \text{ kNm}}$$

Shear force

$$V_{cornerlinex} = W_{\Sigma ultlinex} \times l_{corner} = \mathbf{236.9 \text{ kN}}$$

Moment and shear due to line loads in y direction

Bending moment

$$M_{cornerliney} = W_{\Sigma ultliney} \times l_{corner}^2 / 2 = \mathbf{174.5 \text{ kNm}}$$

Shear force

$$V_{cornerliney} = W_{\Sigma ultliney} \times l_{corner} = \mathbf{148.4 \text{ kN}}$$

Total moments and shears due to point loads

Bending moment about x axis

$$M_{cornerpointx} = \mathbf{0.0 \text{ kNm}}$$

Bending moment about y axis

$$M_{cornerpointy} = \mathbf{0.0 \text{ kNm}}$$

Shear force

$$V_{cornerpoint} = \mathbf{0.0 \text{ kN}}$$

Resultant moments and shears

Total moment about x axis

$$M_{\Sigma cornerx} = M_{cornersw} + M_{cornerudl} + M_{cornerlinex} + M_{cornerpointx} = \mathbf{294.1 \text{ kNm}}$$

Total shear force about x axis

$$V_{\Sigma cornerx} = V_{cornersw} + V_{cornerudl} + V_{cornerlinex} + V_{cornerpoint} = \mathbf{287.7 \text{ kN}}$$

Total moment about y axis

$$M_{\Sigma cornery} = M_{cornersw} + M_{cornerudl} + M_{cornerliney} + M_{cornerpointy} = \mathbf{398.0 \text{ kNm}}$$

Total shear force about y axis

$$V_{\Sigma cornery} = V_{cornersw} + V_{cornerudl} + V_{cornerliney} + V_{cornerpoint} = \mathbf{376.1 \text{ kN}}$$

Deflection of both edge beams at corner will be the same therefore design for average of these moments and shears

Design bending moment

$$M_{\Sigma corner} = (M_{\Sigma cornerx} + M_{\Sigma cornery}) / 2 = \mathbf{346.1 \text{ kNm}}$$

Design shear force

$$V_{\Sigma corner} = (V_{\Sigma cornerx} + V_{\Sigma cornery}) / 2 = \mathbf{331.9 \text{ kN}}$$

Reinforcement required in top of edge beam

K factor

$$K_{corner} = M_{\Sigma corner} / (f_{cu} \times b_{edgetop} \times d_{edgetop}^2) = \mathbf{0.245}$$

$K > 0.156$ - Therefore compression reinforcement is required
The design is outside the scope of this calculation

Corner beam shear check

Average web width

$$b_w = b_{edge} + (h_{edge} / \tan(\alpha_{edge})) / 2 = \mathbf{515 \text{ mm}}$$

Applied shear stress

$$v_{corner} = V_{\Sigma corner} / (b_w \times d_{edgetop}) = \mathbf{2.028 \text{ N/mm}^2}$$

Tension steel ratio

$$\rho_{corner} = 100 \times A_{sedgetop} / (b_w \times d_{edgetop}) = \mathbf{1.800}$$

From BS8110-1:1997 - Table 3.8

Design concrete shear strength

$$V_{ccorner} = \mathbf{0.860 \text{ N/mm}^2}$$

$V_{corner} > V_{ccorner} + 0.4 \text{ N/mm}^2$ - Therefore designed links required

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Link area to spacing ratio required

$$A_{sv_upon_Svreqcorner} = (V_{corner} - V_{ccorner}) \times b_w / ((1.0/\gamma_s) \times f_{ys}) = \mathbf{1.385 \text{ mm}}$$

Link area to spacing ratio provided

$$A_{sv_upon_Svprovedge} = N_{edgelink} \times \pi \times \phi_{edgelink}^2 / (4 \times S_{vedge}) = \mathbf{2.513 \text{ mm}}$$

PASS - $A_{sv_upon_Svreqcorner} \leq A_{sv_upon_Svprovedge}$ - Shear reinforcement provided in edge beams at corners is adequate

Internal beam design checks

Basic loading

Hardcore

$$W_{hcorethick} = \gamma_{hcore} \times h_{hcorethick} = \mathbf{8.0 \text{ kN/m}^2}$$

Internal beam self weight

$$W_{int} = 24 \text{ kN/m}^3 \times [(h_{int} \times b_{int}) + (h_{int} - h_{slab})^2 / \tan(\alpha_{int}) + 2 \times h_{slab} \times (h_{int} - h_{slab}) / \tan(\alpha_{int})]$$

$$W_{int} = \mathbf{5.2 \text{ kN/m}}$$

Internal beam load number 1

Load type

Longitudinal line load

Dead load

$$W_{Dint1} = \mathbf{24.0 \text{ kN/m}}$$

Live load

$$W_{Lint1} = \mathbf{11.6 \text{ kN/m}}$$

Ultimate load

$$W_{ultint1} = 1.4 \times W_{Dint1} + 1.6 \times W_{Lint1} = \mathbf{52.2 \text{ kN/m}}$$

Longitudinal line load width

$$b_{int1} = \mathbf{100 \text{ mm}}$$

Centroid of load from centreline of beam

$$X_{int1} = \mathbf{50 \text{ mm}}$$

Internal beam bearing pressure check

Total uniform load at formation level

$$W_{udlint} = W_{Dudl} + W_{Ludl} + W_{hcorethick} + 24 \text{ kN/m}^3 \times h_{int} = \mathbf{47.9 \text{ kN/m}^2}$$

Sum of factored longitud'l and eff'tive line loads

$$\Sigma U_{DLint} = \mathbf{52.2 \text{ kN/m}}$$

Sum of unfactored longitud'l and eff'tive line loads

$$\Sigma U_{DLsint} = \mathbf{35.6 \text{ kN/m}}$$

Centroid of loads from centreline of internal beam

Load x distance for internal load 1

$$\text{Moment}_{int1} = W_{ultint1} \times X_{int1} = \mathbf{2.6 \text{ kN}}$$

Sum of load x distances

$$\Sigma \text{Moment}_{int} = \mathbf{2.6 \text{ kN}}$$

Centroid of loads

$$X_{barint} = \Sigma \text{Moment}_{int} / \Sigma U_{DLint} = \mathbf{50.0 \text{ mm}}$$

Moment due to eccentricity to be resisted by slab

$$M_{eccint} = \Sigma U_{DLint} \times \text{abs}(X_{barint}) = \mathbf{2.6 \text{ kNm/m}}$$

Assume moment due to eccentricity is resisted equally by top steel of slab on one side and bottom steel of slab on other
From slab bending check

Moment due to depression under slab (hogging)

$$M_{\Sigma e} = \mathbf{17.7 \text{ kNm/m}}$$

Total moment to be resisted by slab top steel

$$M_{slabtopint} = M_{eccint}/2 + M_{\Sigma e} = \mathbf{19.0 \text{ kNm/m}}$$

K factor

$$K_{slabtopint} = M_{slabtopint} / (f_{cu} \times d_{slabmin}^2) = \mathbf{0.014}$$

Lever arm

$$Z_{slabtopint} = d_{slabmin} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabtopint}/0.9)}) = \mathbf{185 \text{ mm}}$$

Area of steel required

$$A_{slabtopintreq} = M_{slabtopint} / ((1.0/\gamma_s) \times f_{yslab} \times Z_{slabtopint}) = \mathbf{236 \text{ mm}^2/\text{m}}$$

PASS - $A_{slabtopintreq} \leq A_{slabtop}$ - Area of reinforcement in top of slab is adequate to transfer moment into slab

Mt to be resisted by slab btm stl due to load ecc'ty

$$M_{slabbtmint} = M_{eccint}/2 = \mathbf{1.3 \text{ kNm/m}}$$

K factor

$$K_{slabbtmint} = M_{slabbtmint} / (f_{cu} \times d_{slabmin}^2) = \mathbf{0.001}$$

Lever arm

$$Z_{slabbtmint} = d_{slabmin} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabbtmint}/0.9)}) = \mathbf{185 \text{ mm}}$$

Area of steel required in bottom

$$A_{slabbtmintreq} = M_{slabbtmint} / ((1.0/\gamma_s) \times f_{yslab} \times Z_{slabbtmint}) = \mathbf{16 \text{ mm}^2/\text{m}}$$

PASS - $A_{slabbtmintreq} \leq A_{slabbtm}$ - Area of reinforcement in bottom of slab is adequate to transfer moment into slab

Bearing pressure

Initially check bearing pressure based on beam soffit/chamfer width only

Allowable bearing width

$$b_{bearint} = \min(2 \times (b_{int}/2 \times \tan(\alpha_{int}) + h_{int}) / (1 + \tan(\alpha_{int})), b_{int} + 2 \times (h_{int} - h_{slab}) / \tan(\alpha_{int}))$$

$$b_{bearint} = \mathbf{546 \text{ mm}}$$

Bearing pressure due to line/point loads

$$q_{linepointint} = \Sigma U_{DLsint} / b_{bearint} = \mathbf{65.2 \text{ kN/m}^2}$$

Total applied bearing pressure

$$q_{int} = q_{linepointint} + W_{udlint} = \mathbf{113.1 \text{ kN/m}^2}$$

PASS - $q_{int} \leq q_{allow}$ - Allowable bearing pressure is not exceeded

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Internal beam bending check

Divider for moments due to udl's $\beta_{udl} = 10.0$

Applied bending moments

Span of internal beam $l_{int} = \phi_{depthick} + d_{inttop} = 3430 \text{ mm}$

Ultimate self weight udl $W_{intult} = 1.4 \times W_{int} = 7.3 \text{ kN/m}$

Ultimate slab udl (approx) $W_{intslab} = \max(0 \text{ kN/m}, 1.4 \times W_{slab} \times ((\phi_{depthick} \times 3/4) - (b_{int} + 2 \times (h_{int} - h_{slab}) / \tan(\alpha_{int}))))$

$W_{intslab} = 14.7 \text{ kN/m}$

Self weight and slab bending moment $M_{intsw} = (W_{intult} + W_{intslab}) \times l_{int}^2 / \beta_{udl} = 25.9 \text{ kNm}$

Self weight shear force $V_{intsw} = (W_{intult} + W_{intslab}) \times l_{int} / 2 = 37.7 \text{ kN}$

Moments due to applied uniformly distributed loads

Ultimate udl (approx) $W_{intudl} = W_{udlult} \times \phi_{depthick} \times 3/4 = 101.4 \text{ kN/m}$

Bending moment $M_{intudl} = W_{intudl} \times l_{int}^2 / \beta_{udl} = 119.3 \text{ kNm}$

Shear force $V_{intudl} = W_{intudl} \times l_{int} / 2 = 173.9 \text{ kN}$

Moment and shear due to load number 1

Bending moment $M_{int1} = W_{ultint1} \times l_{int}^2 / \beta_{udl} = 61.4 \text{ kNm}$

Shear force $V_{int1} = W_{ultint1} \times l_{int} / 2 = 89.5 \text{ kN}$

Resultant moments and shears

Total moment (hogging and sagging) $M_{\Sigma int} = 206.5 \text{ kNm}$

Maximum shear force $V_{\Sigma int} = 301.0 \text{ kN}$

Reinforcement required in top

Width of section in compression zone $d_{inttop} = b_{int} = 400 \text{ mm}$

Average web width $b_{wint} = b_{int} + h_{int} / \tan(\alpha_{int}) = 631 \text{ mm}$

K factor $K_{inttop} = M_{\Sigma int} / (f_{cu} \times b_{inttop} \times d_{inttop}^2) = 0.135$

Lever arm $Z_{inttop} = d_{inttop} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{inttop} / 0.9)}) = 269 \text{ mm}$

Area of steel required for bending $A_{sinttopbend} = M_{\Sigma int} / ((1.0 / \gamma_s) \times f_y \times Z_{inttop}) = 1765 \text{ mm}^2$

Minimum area of steel $A_{sinttopmin} = 0.0013 \times b_{wint} \times h_{int} = 328 \text{ mm}^2$

Area of steel required $A_{sinttopreq} = \max(A_{sinttopbend}, A_{sinttopmin}) = 1765 \text{ mm}^2$

PASS - $A_{sinttopreq} \leq A_{sinttop}$ - Area of reinforcement provided in top of internal beams is adequate

Reinforcement required in bottom

Width of section in compression zone $b_{intbtm} = b_{int} + 2 \times (h_{int} - h_{slab}) / \tan(\alpha_{int}) + 0.2 \times l_{int} = 1259 \text{ mm}$

K factor $K_{intbtm} = M_{\Sigma int} / (f_{cu} \times b_{intbtm} \times d_{intbtm}^2) = 0.042$

Lever arm $Z_{intbtm} = d_{intbtm} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{intbtm} / 0.9)}) = 317 \text{ mm}$

Area of steel required for bending $A_{sintbtmbend} = M_{\Sigma int} / ((1.0 / \gamma_s) \times f_y \times Z_{intbtm}) = 1497 \text{ mm}^2$

Minimum area of steel required $A_{sintbtmmin} = 0.0013 \times 1.0 \times b_{wint} \times h_{int} = 328 \text{ mm}^2$

Area of steel required $A_{sintbtmreq} = \max(A_{sintbtmbend}, A_{sintbtmmin}) = 1497 \text{ mm}^2$

PASS - $A_{sintbtmreq} \leq A_{sintbtm}$ - Area of reinforcement provided in bottom of internal beams is adequate

Internal beam shear check

Applied shear stress $V_{int} = V_{\Sigma int} / (b_{wint} \times d_{inttop}) = 1.446 \text{ N/mm}^2$

Tension steel ratio $\rho_{int} = 100 \times A_{sinttop} / (b_{wint} \times d_{inttop}) = 0.905$

From BS8110-1:1997 - Table 3.8

Design concrete shear strength $V_{cint} = 0.684 \text{ N/mm}^2$

$V_{int} > V_{cint} + 0.4 \text{ N/mm}^2$ - Therefore designed links required

Link area to spacing ratio required $A_{sv_upon_Svreqint} = (V_{int} - V_{cint}) \times b_{wint} / ((1.0 / \gamma_s) \times f_{ys}) = 1.106 \text{ mm}$

Link area to spacing ratio provided $A_{sv_upon_Svprovinc} = N_{intlink} \times \pi \times \phi_{intlink}^2 / (4 \times S_{vinc}) = 1.237 \text{ mm}$

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PASS - $A_{sv_upon_S_{vreqint}} \leq A_{sv_upon_S_{vprovint}}$ - Shear reinforcement provided in internal beams is adequate