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BS Floor Slab A

\_exVarsLib = "$(SysLbrDir)8110chks.lbr"

\_exVarsItem = "Example variables 03"

Eval( if( GetVar("\_exFirstCalculate", True) == True, "EvalCalcItem(\_exVarsLib, \_exVarsItem)", "" )) = **0.000**

\_exFirstCalculate = False

RC Slab design (BS8110:Part1:1997)

TEDDS calculation version 1.0.04

(Library item – RC slab design title)

Two Way Spanning Slab Definition - Restrained

; Overall depth of slab; h = **400** mm

Outer sagging steel

; Cover to outer tension reinforcement resisting sagging; csag = **20** mm

; Trial bar diameter; Dtryx = **16** mm

Depth to outer tension steel (resisting sagging)

dx = h - csag - Dtryx/2 = **372** mm

Inner sagging steel

; Trial bar diameter; Dtryy = **16** mm

Depth to inner tension steel (resisting sagging)

dy = h - csag - Dtryx - Dtryy/2 = **356** mm

Outer hogging steel

; Cover to outer tension reinforcement resisting hogging; chog = **20** mm

; Trial bar diameter; Dtryxhog = **16** mm

Depth to outer tension steel (resisting hogging)

dxhog = h - chog - Dtryxhog/2 = **372** mm

Inner hogging steel

; Trial bar diameter; Dtryyhog = **16** mm

Depth to inner tension steel (resisting hogging)

dyhog = h - chog - Dtryxhog - Dtryyhog/2 = **356** mm

Materials

; Characteristic strength of reinforcement; fy = **500** N/mm2

; Characteristic strength of concrete; fcu = **40** N/mm2

****

Restrained – 2 way Spanning (Cl 3.5.3)

Maximum Design Moments

; Length of shorter side of slab; lx = **5.000** m

; Length of longer side of slab; ly = **5.000** m

; Design ultimate load per unit area; ns = **15.0** kN/m2

; Edge condition shorter side (1); Edge1 = **"C"**

; Edge condition other shorter side (2); Edge2 = **"D"**

; Edge condition longer side (3); Edge3 = **"C"**

; Edge condition other longer side (4); Edge4 = **"D"**

Number of discontinuous edges

Nd = if(Edge1 == "C", 0,1) + if(Edge2 == "C", 0,1) + if(Edge3 == "C", 0,1) + if(Edge4 == "C", 0,1)

Nd = **2**

Moment coefficients

sy = (24 + 2Nd + 1.5Nd2) / 1000 =**0.034**

1 = if(Edge1 == "C", 4/3sy,0) = **0.045**

2 = if(Edge2 == "C", 4/3sy,0) = **0.000**

 = 2/9  [3 - (18)  lx/ly  ((sy + 1) + (sy + 2))] = **0.227**

3x = if(Edge3 == "C", 4/3,0) = **1.333**

4x = if(Edge4 == "C", 4/3,0) = **0.000**

sx =  / [(1+3x)0.5 + (1+4x)0.5]2 = **0.036**

3 = 3x  sx = **0.047**

4 = 4x  sx = **0.000**

Maximum span moments per unit width - restrained slabs

msx = sx  ns  lx2 = **13.3** kNm/m

msy = sy  ns  lx2 =**12.8** kNm/m

Maximum support moments per unit width - restrained slabs

msxhog = max( 3 , 4 )  ns  lx2 = **17.8** kNm/m

msyhog = max( 1 , 2 )  ns  lx2 =**17.0** kNm/m

Concrete Slab Design – Sagging – Outer Layer of Steel (Cl 3.5.4)

; Design sagging moment (per m width of slab); msx = **13.3** kNm/m

; Moment Redistribution Factor; bx = **1.0**

**Area of reinforcement required**

;; Kx = abs(msx) / ( dx2  fcu ) = **0.002**

K'x = min (0.156 , (0.402  (bx - 0.4)) - (0.18  (bx - 0.4)2 )) = **0.156**

Outer compression steel not required to resist sagging

Slab requiring outer tension steel only - bars (sagging)

;; zx = min (( 0.95  dx),(dx(0.5+0.25-Kx/0.9)))) = **353** mm

Neutral axis depth; xx = (dx - zx) / 0.45 = **41** mm

Area of tension steel required

;;; Asx\_req = abs(msx) / (1/ms fy  zx) = **87** mm2/m

**Tension steel**

;;Provide 20 dia bars @ 300 centres; outer tension steel resisting sagging

Asx\_prov = Asx = **1050** mm2/m

Area of outer tension steel provided sufficient to resist sagging

Concrete Slab Design - Sagging - Inner layer of steel (cl. 3.5.4)

; Design sagging moment (per m width of slab); msy = **12.8** kNm/m

; Moment Redistribution Factor; by = **1.0**

**Area of reinforcement required**

;; Ky = abs(msy) / ( dy2  fcu ) = **0.003**

K'y = min (0.156 , (0.402  (by - 0.4)) - (0.18  (by - 0.4)2 )) = **0.156**

Inner compression steel not required to resist sagging

Slab requiring inner tension steel only - bars (sagging)

;; zy = min (( 0.95  dy),(dy(0.5+0.25-Ky/0.9)))) = **338** mm

Neutral axis depth; xy = (dy - zy) / 0.45 = **40** mm

Area of tension steel required

;;; Asy\_req = abs(msy) / (1/ms fy  zy) = **87** mm2/m

**Tension steel**

;;Provide 20 dia bars @ 300 centres; inner tension steel resisting sagging

Asy\_prov = Asy = **1050** mm2/m

Area of inner tension steel provided sufficient to resist sagging

Concrete Slab Design – HOgging – Outer Layer of Steel (Cl 3.5.4)

; Design hogging moment (per m width of slab); msxhog = **17.8** kNm/m

; Moment Redistribution Factor; bx = **1.0**

**Area of reinforcement required**

;; Kxhog = abs(msxhog) / ( dxhog2  fcu ) = **0.003**

K'x = min (0.156 , (0.402  (bx - 0.4)) - (0.18  (bx - 0.4)2 )) = **0.156**

Outer compression steel not required to resist hogging

Slab requiring outer tension steel only - bars (hogging)

;; zxhog = min (( 0.95  dxhog),(dxhog(0.5+0.25-Kxhog/0.9)))) = **353** mm

Neutral axis depth; xxhog = (dxhog - zxhog) / 0.45 = **41** mm

Area of tension steel required

;;; Asxhog\_req = abs(msxhog) / (1/ms fy  zxhog) = **116** mm2/m

**Tension steel**

;;Provide 20 dia bars @ 300 centres; outer tension steel resisting hogging

Asxhog\_prov = Asxhog = **1050** mm2/m

Area of outer tension steel provided sufficient to resist hogging

Concrete Slab Design - hogging - Inner layer of steel (cl. 3.5.4)

; Design hogging moment (per m width of slab); msyhog = **17.0** kNm/m

; Moment Redistribution Factor; by = **1.0**

**Area of reinforcement required**

;; Kyhog = abs(msyhog) / ( dyhog2  fcu ) = **0.003**

K'y = min (0.156 , (0.402  (by - 0.4)) - (0.18  (by - 0.4)2 )) = **0.156**

Inner compression steel not required to resist hogging

Slab requiring inner tension steel only - bars (hogging)

;; zyhog = min (( 0.95  dyhog),(dyhog(0.5+0.25-Kyhog/0.9)))) = **338** mm

Neutral axis depth; xyhog = (dyhog - zyhog) / 0.45 = **40** mm

Area of tension steel required

;;; Asyhog\_req = abs(msyhog) / (1/ms fy  zyhog) = **116** mm2/m

**Tension steel**

;;Provide 20 dia bars @ 300 centres; inner tension steel resisting hogging

Asyhog\_prov = Asyhog = **1050** mm2/m

Area of inner tension steel provided sufficient to resist hogging

Check min and max areas of steel resisting sagging

;Total area of concrete; Ac = h = **400000** mm2/m

; Minimum % reinforcement; k = **0.13** %

Ast\_min = k  Ac = **520** mm2/m

Ast\_max = 4 %  Ac = **16000** mm2/m

Steel defined:

; Outer steel resisting sagging; Asx\_prov = **1050** mm2/m

Area of outer steel provided (sagging) OK

; Inner steel resisting sagging; Asy\_prov = **1050** mm2/m

Area of inner steel provided (sagging) OK

Check min and max areas of steel resisting hogging

;Total area of concrete; Ac = h = **400000** mm2/m

; Minimum % reinforcement; k = **0.13** %

Ast\_min = k  Ac = **520** mm2/m

Ast\_max = 4 %  Ac = **16000** mm2/m

Steel defined:

; Outer steel resisting hogging; Asxhog\_prov = **1050** mm2/m

Area of outer steel provided (hogging) OK

; Inner steel resisting hogging ; Asyhog\_prov = **1050** mm2/m

Area of inner steel provided (hogging) OK

Shear Resistance of Concrete Slabs (Cl 3.5.5)

**Outer tension steel resisting sagging moments**

; Depth to tension steel from compression face; dx = **372** mm

; Area of tension reinforcement provided (per m width of slab); Asx\_prov = **1050** mm2/m

; Design ultimate shear force (per m width of slab); Vx = **25** kN/m

; Characteristic strength of concrete; fcu = **40** N/mm2

**Applied shear stress**

vx = Vx / dx = **0.07** N/mm2

**Check shear stress to clause 3.5.5.2**

vallowable = min ((0.8 N1/2/mm)  (fcu ), 5 N/mm2 ) = **5.00** N/mm2

Shear stress - OK

**Shear stresses to clause 3.5.5.3**

**Design shear stress**

fcu\_ratio = if (fcu > 40 N/mm2 , 40/25 , fcu/(25 N/mm2)) = **1.600**

vcx = 0.79 N/mm2   min(3,100  Asx\_prov / dx)1/3  max(0.67,(400 mm / dx)1/4) / 1.25  fcu\_ratio1/3

vcx = **0.49** N/mm2

Applied shear stress

vx = **0.07** N/mm2

No shear reinforcement required

Library item - Calcs - shear steel? (outer/sag)

Shear Resistance of Concrete Slabs (Cl 3.5.5)

**Inner tension steel resisting sagging moments**

; Depth to tension steel from compression face; dy = **356** mm

; Area of tension reinforcement provided (per m width of slab); Asy\_prov = **1050** mm2/m

; Design ultimate shear force (per m width of slab); Vy = **20** kN/m

; Characteristic strength of concrete; fcu = **40** N/mm2

**Applied shear stress**

vy = Vy / dy = **0.06** N/mm2

**Check shear stress to clause 3.5.5.2**

vallowable = min ((0.8 N1/2/mm)  (fcu ), 5 N/mm2 ) = **5.00** N/mm2

Shear stress - OK

**Shear stresses to clause 3.5.5.3**

**Design shear stress**

fcu\_ratio = if (fcu > 40 N/mm2 , 40/25 , fcu/(25 N/mm2)) = **1.600**

vcy = 0.79 N/mm2   min(3,100  Asy\_prov / dy)1/3  max(0.67,(400 mm) / dy)1/4 / 1.25  fcu\_ratio1/3

vcy = **0.51** N/mm2

Applied shear stress

vy = **0.06** N/mm2

No shear reinforcement required

Shear perimeters for a rectangular concentrated load (cl 3.7.7)

; Length of loaded rectangle; l = **300** mm

; Width of loaded rectangle; w = **300** mm

; Depth to tension steel; dx = **372** mm

; Dimension from edge of load to shear perimeter; lp = kp  dx = **558** mm; where; kp = **1.50**

For punching shear cases not affected by free edges or holes:

Total length of inner perimeter at edge of loaded area; u0\_gen = 2  (l + w) = **1200** mm

Total length of outer perimeter at lp from loaded area; ugen = 2  (l + w) + 8  lp  **5664** mm

Punching shear at concentrated loads (cl 3.7.7)

**Tension steel resisting sagging**

; Total length of inner perimeter at edge of loaded area; u0 = **1200** mm

; Total length of outer perimeter at dimension lp from loaded area; u = **5664** mm

; Depth to outer steel; dx = **372** mm

; Depth to inner steel; dy = **356** mm

Average depth to "tension" steel; dav = (dx + dy)/2 = **364.0** mm

; Area of outer steel per m effective through the perimeter; Asx\_prov = **1050** mm2 /m

; Area of inner steel per m effective through the perimeter; Asy\_prov = **1050** mm2 /m

; Max shear effective across either perimeter under consideration; Vp = **50** kN

; Characteristic strength of concrete; fcu = **40** N/mm2

**Applied shear stress**

Stress around loaded area; vmax = Vp / (u0  dav) = **0.114** N/mm2

Stress around perimeter; v = Vp / (u  dav) = **0.024** N/mm2

**Check shear stress to clause 3.7.7.2**

vallowable = min ((0.8 N1/2/mm)  (fcu ), 5 N/mm2 ) = **5.000** N/mm2

Shear stress - OK

**Shear stresses to clause 3.7.7.4**

**Design shear stress**

fcu\_ratio = if (fcu > 40 N/mm2 , 40/25 , fcu/(25 N/mm2)) = **1.600**

; Effective steel area for shear strength determination:; As\_eff =**201** mm2/m;

vc = 0.79 N/mm2  min( 3, 100( As\_eff / dav ) )1/3  max(0.67, (400 mm / dav )1/4) / 1.25  fcu\_ratio1/3

vc = **0.288** N/mm2

No shear reinforcement required

Library item - Calcs slab punching shear (sag)

Concrete Slab Deflection Check (cl 3.5.7)

; Slab span length; lx = **5.000** m

; Design ultimate moment in shorter span per m width; msx = **13** kNm/m

; Depth to outer tension steel; dx = **372** mm

**Tension steel**

; Area of outer tension reinforcement provided; Asx\_prov = **1050** mm2/m

; Area of tension reinforcement required; Asx\_req = **87** mm2/m

; Moment Redistribution Factor; bx = **1.00**

**Modification Factors**

;Basic span / effective depth ratio (Table 3.9); ratiospan\_depth = **60**

The modification factor for spans in excess of 10m (ref. cl 3.4.6.4) has not been included.

;fs = 2 × fy  Asx\_req / (3  Asx\_prov  bx ) = **27.6** N/mm2

factortens = min ( 2 , 0.55 + ( 477 N/mm2 - fs ) / ( 120  ( 0.9 N/mm2 + msx / dx2))) = **2.000**

**Calculate Maximum Span**

This is a simplified approach and further attention should be given where special circumstances exist. Refer to clauses 3.4.6.4 and 3.4.6.7.

Maximum span; lmax = ratiospan\_depth  factortens  dx = **44.64** m

**Check the actual beam span**

Actual span/depth ratio; lx / dx = **13.44**

Span depth limit; ratiospan\_depth  factortens = **120.00**

Span/Depth ratio check satisfied

Check of Nominal Cover (Sagging) – (BS8110:Pt 1, Table 3.4)

; Slab thickness; h = **400** mm

; Effective depth to bottom outer tension reinforcement; dx = **372.0** mm

; Diameter of tension reinforcement; Dx = **20** mm

; Diameter of links; Ldiax = **0** mm

Cover to outer tension reinforcement

ctenx = h - dx - Dx / 2 = **18.0** mm

Nominal cover to links steel

cnomx = ctenx - Ldiax = **18.0** mm

Permissable minimum nominal cover to all reinforcement (Table 3.4)

; cmin = **15** mm

Cover over steel resisting sagging OK

Check of Nominal Cover (HOgging) – (BS8110:Pt 1, Table 3.4)

; Slab thickness; h = **400** mm

; Effective depth to bottom outer tension reinforcement; dxhog = **372.0** mm

; Diameter of tension reinforcement; Dxhog = **20** mm

; Diameter of links; Ldiaxhog = **0** mm

Cover to outer tension reinforcement

ctenxhog = h - dxhog - Dxhog / 2 = **18.0** mm

Nominal cover to links steel

cnomxhog = ctenxhog - Ldiaxhog = **18.0** mm

Permissable minimum nominal cover to all reinforcement (Table 3.4)

; cmin = **15** mm

Cover OK over steel resisting hogging

;

;

BS Floor Slab B

\_exVarsLib = "$(SysLbrDir)8110chks.lbr"

\_exVarsItem = "Example variables 03"

Eval( if( GetVar("\_exFirstCalculate", True) == True, "EvalCalcItem(\_exVarsLib, \_exVarsItem)", "" )) = **0.000**

\_exFirstCalculate = False

RC Slab design (BS8110:Part1:1997)

TEDDS calculation version 1.0.04

(Library item – RC slab design title)

Two Way Spanning Slab Definition - Restrained

; Overall depth of slab; h = **400** mm

Outer sagging steel

; Cover to outer tension reinforcement resisting sagging; csag = **20** mm

; Trial bar diameter; Dtryx = **16** mm

Depth to outer tension steel (resisting sagging)

dx = h - csag - Dtryx/2 = **372** mm

Inner sagging steel

; Trial bar diameter; Dtryy = **16** mm

Depth to inner tension steel (resisting sagging)

dy = h - csag - Dtryx - Dtryy/2 = **356** mm

Outer hogging steel

; Cover to outer tension reinforcement resisting hogging; chog = **20** mm

; Trial bar diameter; Dtryxhog = **16** mm

Depth to outer tension steel (resisting hogging)

dxhog = h - chog - Dtryxhog/2 = **372** mm

Inner hogging steel

; Trial bar diameter; Dtryyhog = **16** mm

Depth to inner tension steel (resisting hogging)

dyhog = h - chog - Dtryxhog - Dtryyhog/2 = **356** mm

Materials

; Characteristic strength of reinforcement; fy = **500** N/mm2

; Characteristic strength of concrete; fcu = **40** N/mm2

****

Restrained – 2 way Spanning (Cl 3.5.3)

Maximum Design Moments

; Length of shorter side of slab; lx = **5.000** m

; Length of longer side of slab; ly = **5.000** m

; Design ultimate load per unit area; ns = **15.0** kN/m2

; Edge condition shorter side (1); Edge1 = **"C"**

; Edge condition other shorter side (2); Edge2 = **"D"**

; Edge condition longer side (3); Edge3 = **"D"**

; Edge condition other longer side (4); Edge4 = **"D"**

Number of discontinuous edges

Nd = if(Edge1 == "C", 0,1) + if(Edge2 == "C", 0,1) + if(Edge3 == "C", 0,1) + if(Edge4 == "C", 0,1)

Nd = **3**

Moment coefficients

sy = (24 + 2Nd + 1.5Nd2) / 1000 =**0.044**

1 = if(Edge1 == "C", 4/3sy,0) = **0.058**

2 = if(Edge2 == "C", 4/3sy,0) = **0.000**

 = 2/9  [3 - (18)  lx/ly  ((sy + 1) + (sy + 2))] = **0.170**

3x = if(Edge3 == "C", 4/3,0) = **0.000**

4x = if(Edge4 == "C", 4/3,0) = **0.000**

sx =  / [(1+3x)0.5 + (1+4x)0.5]2 = **0.042**

3 = 3x  sx = **0.000**

4 = 4x  sx = **0.000**

Maximum span moments per unit width - restrained slabs

msx = sx  ns  lx2 = **15.9** kNm/m

msy = sy  ns  lx2 =**16.3** kNm/m

Maximum support moments per unit width - restrained slabs

msxhog = max( 3 , 4 )  ns  lx2 = **0.0** kNm/m

msyhog = max( 1 , 2 )  ns  lx2 =**21.7** kNm/m

Concrete Slab Design – Sagging – Outer Layer of Steel (Cl 3.5.4)

; Design sagging moment (per m width of slab); msx = **15.9** kNm/m

; Moment Redistribution Factor; bx = **1.0**

**Area of reinforcement required**

;; Kx = abs(msx) / ( dx2  fcu ) = **0.003**

K'x = min (0.156 , (0.402  (bx - 0.4)) - (0.18  (bx - 0.4)2 )) = **0.156**

Outer compression steel not required to resist sagging

Slab requiring outer tension steel only - bars (sagging)

;; zx = min (( 0.95  dx),(dx(0.5+0.25-Kx/0.9)))) = **353** mm

Neutral axis depth; xx = (dx - zx) / 0.45 = **41** mm

Area of tension steel required

;;; Asx\_req = abs(msx) / (1/ms fy  zx) = **104** mm2/m

**Tension steel**

;;Provide 20 dia bars @ 250 centres; outer tension steel resisting sagging

Asx\_prov = Asx = **1260** mm2/m

Area of outer tension steel provided sufficient to resist sagging

Concrete Slab Design - Sagging - Inner layer of steel (cl. 3.5.4)

; Design sagging moment (per m width of slab); msy = **16.3** kNm/m

; Moment Redistribution Factor; by = **1.0**

**Area of reinforcement required**

;; Ky = abs(msy) / ( dy2  fcu ) = **0.003**

K'y = min (0.156 , (0.402  (by - 0.4)) - (0.18  (by - 0.4)2 )) = **0.156**

Inner compression steel not required to resist sagging

Slab requiring inner tension steel only - bars (sagging)

;; zy = min (( 0.95  dy),(dy(0.5+0.25-Ky/0.9)))) = **338** mm

Neutral axis depth; xy = (dy - zy) / 0.45 = **40** mm

Area of tension steel required

;;; Asy\_req = abs(msy) / (1/ms fy  zy) = **111** mm2/m

**Tension steel**

;;Provide 20 dia bars @ 250 centres; inner tension steel resisting sagging

Asy\_prov = Asy = **1260** mm2/m

Area of inner tension steel provided sufficient to resist sagging

Concrete Slab Design – HOgging – Outer Layer of Steel (Cl 3.5.4)

; Design hogging moment (per m width of slab); msxhog = **0.0** kNm/m

; Moment Redistribution Factor; bx = **1.0**

**Area of reinforcement required**

;; Kxhog = abs(msxhog) / ( dxhog2  fcu ) = **0.000**

K'x = min (0.156 , (0.402  (bx - 0.4)) - (0.18  (bx - 0.4)2 )) = **0.156**

Outer compression steel not required to resist hogging

Slab requiring outer tension steel only - bars (hogging)

;; zxhog = min (( 0.95  dxhog),(dxhog(0.5+0.25-Kxhog/0.9)))) = **353** mm

Neutral axis depth; xxhog = (dxhog - zxhog) / 0.45 = **41** mm

Area of tension steel required

;;; Asxhog\_req = abs(msxhog) / (1/ms fy  zxhog) = **0** mm2/m

**Tension steel**

;;Provide 20 dia bars @ 250 centres; outer tension steel resisting hogging

Asxhog\_prov = Asxhog = **1260** mm2/m

Area of outer tension steel provided sufficient to resist hogging

Concrete Slab Design - hogging - Inner layer of steel (cl. 3.5.4)

; Design hogging moment (per m width of slab); msyhog = **21.7** kNm/m

; Moment Redistribution Factor; by = **1.0**

**Area of reinforcement required**

;; Kyhog = abs(msyhog) / ( dyhog2  fcu ) = **0.004**

K'y = min (0.156 , (0.402  (by - 0.4)) - (0.18  (by - 0.4)2 )) = **0.156**

Inner compression steel not required to resist hogging

Slab requiring inner tension steel only - bars (hogging)

;; zyhog = min (( 0.95  dyhog),(dyhog(0.5+0.25-Kyhog/0.9)))) = **338** mm

Neutral axis depth; xyhog = (dyhog - zyhog) / 0.45 = **40** mm

Area of tension steel required

;;; Asyhog\_req = abs(msyhog) / (1/ms fy  zyhog) = **148** mm2/m

**Tension steel**

;;Provide 20 dia bars @ 250 centres; inner tension steel resisting hogging

Asyhog\_prov = Asyhog = **1260** mm2/m

Area of inner tension steel provided sufficient to resist hogging

Check min and max areas of steel resisting sagging

;Total area of concrete; Ac = h = **400000** mm2/m

; Minimum % reinforcement; k = **0.13** %

Ast\_min = k  Ac = **520** mm2/m

Ast\_max = 4 %  Ac = **16000** mm2/m

Steel defined:

; Outer steel resisting sagging; Asx\_prov = **1260** mm2/m

Area of outer steel provided (sagging) OK

; Inner steel resisting sagging; Asy\_prov = **1260** mm2/m

Area of inner steel provided (sagging) OK

Check min and max areas of steel resisting hogging

;Total area of concrete; Ac = h = **400000** mm2/m

; Minimum % reinforcement; k = **0.13** %

Ast\_min = k  Ac = **520** mm2/m

Ast\_max = 4 %  Ac = **16000** mm2/m

Steel defined:

; Outer steel resisting hogging; Asxhog\_prov = **1260** mm2/m

Area of outer steel provided (hogging) OK

; Inner steel resisting hogging ; Asyhog\_prov = **1260** mm2/m

Area of inner steel provided (hogging) OK

Shear Resistance of Concrete Slabs (Cl 3.5.5)

**Outer tension steel resisting sagging moments**

; Depth to tension steel from compression face; dx = **372** mm

; Area of tension reinforcement provided (per m width of slab); Asx\_prov = **1260** mm2/m

; Design ultimate shear force (per m width of slab); Vx = **25** kN/m

; Characteristic strength of concrete; fcu = **40** N/mm2

**Applied shear stress**

vx = Vx / dx = **0.07** N/mm2

**Check shear stress to clause 3.5.5.2**

vallowable = min ((0.8 N1/2/mm)  (fcu ), 5 N/mm2 ) = **5.00** N/mm2

Shear stress - OK

**Shear stresses to clause 3.5.5.3**

**Design shear stress**

fcu\_ratio = if (fcu > 40 N/mm2 , 40/25 , fcu/(25 N/mm2)) = **1.600**

vcx = 0.79 N/mm2   min(3,100  Asx\_prov / dx)1/3  max(0.67,(400 mm / dx)1/4) / 1.25  fcu\_ratio1/3

vcx = **0.52** N/mm2

Applied shear stress

vx = **0.07** N/mm2

No shear reinforcement required

Library item - Calcs - shear steel? (outer/sag)

Shear Resistance of Concrete Slabs (Cl 3.5.5)

**Inner tension steel resisting sagging moments**

; Depth to tension steel from compression face; dy = **356** mm

; Area of tension reinforcement provided (per m width of slab); Asy\_prov = **1260** mm2/m

; Design ultimate shear force (per m width of slab); Vy = **20** kN/m

; Characteristic strength of concrete; fcu = **40** N/mm2

**Applied shear stress**

vy = Vy / dy = **0.06** N/mm2

**Check shear stress to clause 3.5.5.2**

vallowable = min ((0.8 N1/2/mm)  (fcu ), 5 N/mm2 ) = **5.00** N/mm2

Shear stress - OK

**Shear stresses to clause 3.5.5.3**

**Design shear stress**

fcu\_ratio = if (fcu > 40 N/mm2 , 40/25 , fcu/(25 N/mm2)) = **1.600**

vcy = 0.79 N/mm2   min(3,100  Asy\_prov / dy)1/3  max(0.67,(400 mm) / dy)1/4 / 1.25  fcu\_ratio1/3

vcy = **0.54** N/mm2

Applied shear stress

vy = **0.06** N/mm2

No shear reinforcement required

Shear perimeters for a rectangular concentrated load (cl 3.7.7)

; Length of loaded rectangle; l = **300** mm

; Width of loaded rectangle; w = **300** mm

; Depth to tension steel; dx = **372** mm

; Dimension from edge of load to shear perimeter; lp = kp  dx = **558** mm; where; kp = **1.50**

For punching shear cases not affected by free edges or holes:

Total length of inner perimeter at edge of loaded area; u0\_gen = 2  (l + w) = **1200** mm

Total length of outer perimeter at lp from loaded area; ugen = 2  (l + w) + 8  lp  **5664** mm

Punching shear at concentrated loads (cl 3.7.7)

**Tension steel resisting sagging**

; Total length of inner perimeter at edge of loaded area; u0 = **1200** mm

; Total length of outer perimeter at dimension lp from loaded area; u = **5664** mm

; Depth to outer steel; dx = **372** mm

; Depth to inner steel; dy = **356** mm

Average depth to "tension" steel; dav = (dx + dy)/2 = **364.0** mm

; Area of outer steel per m effective through the perimeter; Asx\_prov = **1260** mm2 /m

; Area of inner steel per m effective through the perimeter; Asy\_prov = **1260** mm2 /m

; Max shear effective across either perimeter under consideration; Vp = **50** kN

; Characteristic strength of concrete; fcu = **40** N/mm2

**Applied shear stress**

Stress around loaded area; vmax = Vp / (u0  dav) = **0.114** N/mm2

Stress around perimeter; v = Vp / (u  dav) = **0.024** N/mm2

**Check shear stress to clause 3.7.7.2**

vallowable = min ((0.8 N1/2/mm)  (fcu ), 5 N/mm2 ) = **5.000** N/mm2

Shear stress - OK

**Shear stresses to clause 3.7.7.4**

**Design shear stress**

fcu\_ratio = if (fcu > 40 N/mm2 , 40/25 , fcu/(25 N/mm2)) = **1.600**

; Effective steel area for shear strength determination:; As\_eff =**201** mm2/m;

vc = 0.79 N/mm2  min( 3, 100( As\_eff / dav ) )1/3  max(0.67, (400 mm / dav )1/4) / 1.25  fcu\_ratio1/3

vc = **0.288** N/mm2

No shear reinforcement required

Library item - Calcs slab punching shear (sag)

Concrete Slab Deflection Check (cl 3.5.7)

; Slab span length; lx = **5.000** m

; Design ultimate moment in shorter span per m width; msx = **16** kNm/m

; Depth to outer tension steel; dx = **372** mm

**Tension steel**

; Area of outer tension reinforcement provided; Asx\_prov = **1260** mm2/m

; Area of tension reinforcement required; Asx\_req = **104** mm2/m

; Moment Redistribution Factor; bx = **1.00**

**Modification Factors**

;Basic span / effective depth ratio (Table 3.9); ratiospan\_depth = **60**

The modification factor for spans in excess of 10m (ref. cl 3.4.6.4) has not been included.

;fs = 2 × fy  Asx\_req / (3  Asx\_prov  bx ) = **27.4** N/mm2

factortens = min ( 2 , 0.55 + ( 477 N/mm2 - fs ) / ( 120  ( 0.9 N/mm2 + msx / dx2))) = **2.000**

**Calculate Maximum Span**

This is a simplified approach and further attention should be given where special circumstances exist. Refer to clauses 3.4.6.4 and 3.4.6.7.

Maximum span; lmax = ratiospan\_depth  factortens  dx = **44.64** m

**Check the actual beam span**

Actual span/depth ratio; lx / dx = **13.44**

Span depth limit; ratiospan\_depth  factortens = **120.00**

Span/Depth ratio check satisfied

Check of Nominal Cover (Sagging) – (BS8110:Pt 1, Table 3.4)

; Slab thickness; h = **400** mm

; Effective depth to bottom outer tension reinforcement; dx = **372.0** mm

; Diameter of tension reinforcement; Dx = **20** mm

; Diameter of links; Ldiax = **0** mm

Cover to outer tension reinforcement

ctenx = h - dx - Dx / 2 = **18.0** mm

Nominal cover to links steel

cnomx = ctenx - Ldiax = **18.0** mm

Permissable minimum nominal cover to all reinforcement (Table 3.4)

; cmin = **15** mm

Cover over steel resisting sagging OK

Check of Nominal Cover (HOgging) – (BS8110:Pt 1, Table 3.4)

; Slab thickness; h = **400** mm

; Effective depth to bottom outer tension reinforcement; dxhog = **372.0** mm

; Diameter of tension reinforcement; Dxhog = **20** mm

; Diameter of links; Ldiaxhog = **0** mm

Cover to outer tension reinforcement

ctenxhog = h - dxhog - Dxhog / 2 = **18.0** mm

Nominal cover to links steel

cnomxhog = ctenxhog - Ldiaxhog = **18.0** mm

Permissable minimum nominal cover to all reinforcement (Table 3.4)

; cmin = **15** mm

Cover OK over steel resisting hogging

;

;

BS Floor Slab C

\_exVarsLib = "$(SysLbrDir)8110chks.lbr"

\_exVarsItem = "Example variables 03"

Eval( if( GetVar("\_exFirstCalculate", True) == True, "EvalCalcItem(\_exVarsLib, \_exVarsItem)", "" )) = **0.000**

\_exFirstCalculate = False

RC Slab design (BS8110:Part1:1997)

TEDDS calculation version 1.0.04

(Library item – RC slab design title)

Two Way Spanning Slab Definition - Restrained

; Overall depth of slab; h = **250** mm

Outer sagging steel

; Cover to outer tension reinforcement resisting sagging; csag = **20** mm

; Trial bar diameter; Dtryx = **16** mm

Depth to outer tension steel (resisting sagging)

dx = h - csag - Dtryx/2 = **222** mm

Inner sagging steel

; Trial bar diameter; Dtryy = **16** mm

Depth to inner tension steel (resisting sagging)

dy = h - csag - Dtryx - Dtryy/2 = **206** mm

Outer hogging steel

; Cover to outer tension reinforcement resisting hogging; chog = **20** mm

; Trial bar diameter; Dtryxhog = **16** mm

Depth to outer tension steel (resisting hogging)

dxhog = h - chog - Dtryxhog/2 = **222** mm

Inner hogging steel

; Trial bar diameter; Dtryyhog = **16** mm

Depth to inner tension steel (resisting hogging)

dyhog = h - chog - Dtryxhog - Dtryyhog/2 = **206** mm

Materials

; Characteristic strength of reinforcement; fy = **500** N/mm2

; Characteristic strength of concrete; fcu = **40** N/mm2

****

Restrained – 2 way Spanning (Cl 3.5.3)

Maximum Design Moments

; Length of shorter side of slab; lx = **5.000** m

; Length of longer side of slab; ly = **5.000** m

; Design ultimate load per unit area; ns = **15.0** kN/m2

; Edge condition shorter side (1); Edge1 = **"C"**

; Edge condition other shorter side (2); Edge2 = **"C"**

; Edge condition longer side (3); Edge3 = **"C"**

; Edge condition other longer side (4); Edge4 = **"C"**

Number of discontinuous edges

Nd = if(Edge1 == "C", 0,1) + if(Edge2 == "C", 0,1) + if(Edge3 == "C", 0,1) + if(Edge4 == "C", 0,1)

Nd = **0**

Moment coefficients

sy = (24 + 2Nd + 1.5Nd2) / 1000 =**0.024**

1 = if(Edge1 == "C", 4/3sy,0) = **0.032**

2 = if(Edge2 == "C", 4/3sy,0) = **0.032**

 = 2/9  [3 - (18)  lx/ly  ((sy + 1) + (sy + 2))] = **0.220**

3x = if(Edge3 == "C", 4/3,0) = **1.333**

4x = if(Edge4 == "C", 4/3,0) = **1.333**

sx =  / [(1+3x)0.5 + (1+4x)0.5]2 = **0.024**

3 = 3x  sx = **0.031**

4 = 4x  sx = **0.031**

Maximum span moments per unit width - restrained slabs

msx = sx  ns  lx2 = **8.9** kNm/m

msy = sy  ns  lx2 =**9.0** kNm/m

Maximum support moments per unit width - restrained slabs

msxhog = max( 3 , 4 )  ns  lx2 = **11.8** kNm/m

msyhog = max( 1 , 2 )  ns  lx2 =**12.0** kNm/m

Concrete Slab Design – Sagging – Outer Layer of Steel (Cl 3.5.4)

; Design sagging moment (per m width of slab); msx = **8.9** kNm/m

; Moment Redistribution Factor; bx = **1.0**

**Area of reinforcement required**

;; Kx = abs(msx) / ( dx2  fcu ) = **0.004**

K'x = min (0.156 , (0.402  (bx - 0.4)) - (0.18  (bx - 0.4)2 )) = **0.156**

Outer compression steel not required to resist sagging

Slab requiring outer tension steel only - bars (sagging)

;; zx = min (( 0.95  dx),(dx(0.5+0.25-Kx/0.9)))) = **211** mm

Neutral axis depth; xx = (dx - zx) / 0.45 = **25** mm

Area of tension steel required

;;; Asx\_req = abs(msx) / (1/ms fy  zx) = **97** mm2/m

**Tension steel**

;;Provide 20 dia bars @ 300 centres; outer tension steel resisting sagging

Asx\_prov = Asx = **1050** mm2/m

Area of outer tension steel provided sufficient to resist sagging

Concrete Slab Design - Sagging - Inner layer of steel (cl. 3.5.4)

; Design sagging moment (per m width of slab); msy = **9.0** kNm/m

; Moment Redistribution Factor; by = **1.0**

**Area of reinforcement required**

;; Ky = abs(msy) / ( dy2  fcu ) = **0.005**

K'y = min (0.156 , (0.402  (by - 0.4)) - (0.18  (by - 0.4)2 )) = **0.156**

Inner compression steel not required to resist sagging

Slab requiring inner tension steel only - bars (sagging)

;; zy = min (( 0.95  dy),(dy(0.5+0.25-Ky/0.9)))) = **196** mm

Neutral axis depth; xy = (dy - zy) / 0.45 = **23** mm

Area of tension steel required

;;; Asy\_req = abs(msy) / (1/ms fy  zy) = **106** mm2/m

**Tension steel**

;;Provide 20 dia bars @ 300 centres; inner tension steel resisting sagging

Asy\_prov = Asy = **1050** mm2/m

Area of inner tension steel provided sufficient to resist sagging

Concrete Slab Design – HOgging – Outer Layer of Steel (Cl 3.5.4)

; Design hogging moment (per m width of slab); msxhog = **11.8** kNm/m

; Moment Redistribution Factor; bx = **1.0**

**Area of reinforcement required**

;; Kxhog = abs(msxhog) / ( dxhog2  fcu ) = **0.006**

K'x = min (0.156 , (0.402  (bx - 0.4)) - (0.18  (bx - 0.4)2 )) = **0.156**

Outer compression steel not required to resist hogging

Slab requiring outer tension steel only - bars (hogging)

;; zxhog = min (( 0.95  dxhog),(dxhog(0.5+0.25-Kxhog/0.9)))) = **211** mm

Neutral axis depth; xxhog = (dxhog - zxhog) / 0.45 = **25** mm

Area of tension steel required

;;; Asxhog\_req = abs(msxhog) / (1/ms fy  zxhog) = **129** mm2/m

**Tension steel**

;;Provide 20 dia bars @ 300 centres; outer tension steel resisting hogging

Asxhog\_prov = Asxhog = **1050** mm2/m

Area of outer tension steel provided sufficient to resist hogging

Concrete Slab Design - hogging - Inner layer of steel (cl. 3.5.4)

; Design hogging moment (per m width of slab); msyhog = **12.0** kNm/m

; Moment Redistribution Factor; by = **1.0**

**Area of reinforcement required**

;; Kyhog = abs(msyhog) / ( dyhog2  fcu ) = **0.007**

K'y = min (0.156 , (0.402  (by - 0.4)) - (0.18  (by - 0.4)2 )) = **0.156**

Inner compression steel not required to resist hogging

Slab requiring inner tension steel only - bars (hogging)

;; zyhog = min (( 0.95  dyhog),(dyhog(0.5+0.25-Kyhog/0.9)))) = **196** mm

Neutral axis depth; xyhog = (dyhog - zyhog) / 0.45 = **23** mm

Area of tension steel required

;;; Asyhog\_req = abs(msyhog) / (1/ms fy  zyhog) = **141** mm2/m

**Tension steel**

;;Provide 20 dia bars @ 300 centres; inner tension steel resisting hogging

Asyhog\_prov = Asyhog = **1050** mm2/m

Area of inner tension steel provided sufficient to resist hogging

Check min and max areas of steel resisting sagging

;Total area of concrete; Ac = h = **250000** mm2/m

; Minimum % reinforcement; k = **0.13** %

Ast\_min = k  Ac = **325** mm2/m

Ast\_max = 4 %  Ac = **10000** mm2/m

Steel defined:

; Outer steel resisting sagging; Asx\_prov = **1050** mm2/m

Area of outer steel provided (sagging) OK

; Inner steel resisting sagging; Asy\_prov = **1050** mm2/m

Area of inner steel provided (sagging) OK

Check min and max areas of steel resisting hogging

;Total area of concrete; Ac = h = **250000** mm2/m

; Minimum % reinforcement; k = **0.13** %

Ast\_min = k  Ac = **325** mm2/m

Ast\_max = 4 %  Ac = **10000** mm2/m

Steel defined:

; Outer steel resisting hogging; Asxhog\_prov = **1050** mm2/m

Area of outer steel provided (hogging) OK

; Inner steel resisting hogging ; Asyhog\_prov = **1050** mm2/m

Area of inner steel provided (hogging) OK

Shear Resistance of Concrete Slabs (Cl 3.5.5)

**Outer tension steel resisting sagging moments**

; Depth to tension steel from compression face; dx = **222** mm

; Area of tension reinforcement provided (per m width of slab); Asx\_prov = **1050** mm2/m

; Design ultimate shear force (per m width of slab); Vx = **25** kN/m

; Characteristic strength of concrete; fcu = **40** N/mm2

**Applied shear stress**

vx = Vx / dx = **0.11** N/mm2

**Check shear stress to clause 3.5.5.2**

vallowable = min ((0.8 N1/2/mm)  (fcu ), 5 N/mm2 ) = **5.00** N/mm2

Shear stress - OK

**Shear stresses to clause 3.5.5.3**

**Design shear stress**

fcu\_ratio = if (fcu > 40 N/mm2 , 40/25 , fcu/(25 N/mm2)) = **1.600**

vcx = 0.79 N/mm2   min(3,100  Asx\_prov / dx)1/3  max(0.67,(400 mm / dx)1/4) / 1.25  fcu\_ratio1/3

vcx = **0.67** N/mm2

Applied shear stress

vx = **0.11** N/mm2

No shear reinforcement required

Library item - Calcs - shear steel? (outer/sag)

Shear Resistance of Concrete Slabs (Cl 3.5.5)

**Inner tension steel resisting sagging moments**

; Depth to tension steel from compression face; dy = **206** mm

; Area of tension reinforcement provided (per m width of slab); Asy\_prov = **1050** mm2/m

; Design ultimate shear force (per m width of slab); Vy = **20** kN/m

; Characteristic strength of concrete; fcu = **40** N/mm2

**Applied shear stress**

vy = Vy / dy = **0.10** N/mm2

**Check shear stress to clause 3.5.5.2**

vallowable = min ((0.8 N1/2/mm)  (fcu ), 5 N/mm2 ) = **5.00** N/mm2

Shear stress - OK

**Shear stresses to clause 3.5.5.3**

**Design shear stress**

fcu\_ratio = if (fcu > 40 N/mm2 , 40/25 , fcu/(25 N/mm2)) = **1.600**

vcy = 0.79 N/mm2   min(3,100  Asy\_prov / dy)1/3  max(0.67,(400 mm) / dy)1/4 / 1.25  fcu\_ratio1/3

vcy = **0.70** N/mm2

Applied shear stress

vy = **0.10** N/mm2

No shear reinforcement required

Shear perimeters for a rectangular concentrated load (cl 3.7.7)

; Length of loaded rectangle; l = **300** mm

; Width of loaded rectangle; w = **300** mm

; Depth to tension steel; dx = **222** mm

; Dimension from edge of load to shear perimeter; lp = kp  dx = **333** mm; where; kp = **1.50**

For punching shear cases not affected by free edges or holes:

Total length of inner perimeter at edge of loaded area; u0\_gen = 2  (l + w) = **1200** mm

Total length of outer perimeter at lp from loaded area; ugen = 2  (l + w) + 8  lp  **3864** mm

Punching shear at concentrated loads (cl 3.7.7)

**Tension steel resisting sagging**

; Total length of inner perimeter at edge of loaded area; u0 = **1200** mm

; Total length of outer perimeter at dimension lp from loaded area; u = **3864** mm

; Depth to outer steel; dx = **222** mm

; Depth to inner steel; dy = **206** mm

Average depth to "tension" steel; dav = (dx + dy)/2 = **214.0** mm

; Area of outer steel per m effective through the perimeter; Asx\_prov = **1050** mm2 /m

; Area of inner steel per m effective through the perimeter; Asy\_prov = **1050** mm2 /m

; Max shear effective across either perimeter under consideration; Vp = **50** kN

; Characteristic strength of concrete; fcu = **40** N/mm2

**Applied shear stress**

Stress around loaded area; vmax = Vp / (u0  dav) = **0.195** N/mm2

Stress around perimeter; v = Vp / (u  dav) = **0.060** N/mm2

**Check shear stress to clause 3.7.7.2**

vallowable = min ((0.8 N1/2/mm)  (fcu ), 5 N/mm2 ) = **5.000** N/mm2

Shear stress - OK

**Shear stresses to clause 3.7.7.4**

**Design shear stress**

fcu\_ratio = if (fcu > 40 N/mm2 , 40/25 , fcu/(25 N/mm2)) = **1.600**

; Effective steel area for shear strength determination:; As\_eff =**201** mm2/m;

vc = 0.79 N/mm2  min( 3, 100( As\_eff / dav ) )1/3  max(0.67, (400 mm / dav )1/4) / 1.25  fcu\_ratio1/3

vc = **0.393** N/mm2

No shear reinforcement required

Library item - Calcs slab punching shear (sag)

Concrete Slab Deflection Check (cl 3.5.7)

; Slab span length; lx = **5.000** m

; Design ultimate moment in shorter span per m width; msx = **9** kNm/m

; Depth to outer tension steel; dx = **222** mm

**Tension steel**

; Area of outer tension reinforcement provided; Asx\_prov = **1050** mm2/m

; Area of tension reinforcement required; Asx\_req = **97** mm2/m

; Moment Redistribution Factor; bx = **1.00**

**Modification Factors**

;Basic span / effective depth ratio (Table 3.9); ratiospan\_depth = **60**

The modification factor for spans in excess of 10m (ref. cl 3.4.6.4) has not been included.

;fs = 2 × fy  Asx\_req / (3  Asx\_prov  bx ) = **30.7** N/mm2

factortens = min ( 2 , 0.55 + ( 477 N/mm2 - fs ) / ( 120  ( 0.9 N/mm2 + msx / dx2))) = **2.000**

**Calculate Maximum Span**

This is a simplified approach and further attention should be given where special circumstances exist. Refer to clauses 3.4.6.4 and 3.4.6.7.

Maximum span; lmax = ratiospan\_depth  factortens  dx = **26.64** m

**Check the actual beam span**

Actual span/depth ratio; lx / dx = **22.52**

Span depth limit; ratiospan\_depth  factortens = **120.00**

Span/Depth ratio check satisfied

Check of Nominal Cover (Sagging) – (BS8110:Pt 1, Table 3.4)

; Slab thickness; h = **250** mm

; Effective depth to bottom outer tension reinforcement; dx = **222.0** mm

; Diameter of tension reinforcement; Dx = **20** mm

; Diameter of links; Ldiax = **0** mm

Cover to outer tension reinforcement

ctenx = h - dx - Dx / 2 = **18.0** mm

Nominal cover to links steel

cnomx = ctenx - Ldiax = **18.0** mm

Permissable minimum nominal cover to all reinforcement (Table 3.4)

; cmin = **15** mm

Cover over steel resisting sagging OK

Check of Nominal Cover (HOgging) – (BS8110:Pt 1, Table 3.4)

; Slab thickness; h = **250** mm

; Effective depth to bottom outer tension reinforcement; dxhog = **222.0** mm

; Diameter of tension reinforcement; Dxhog = **20** mm

; Diameter of links; Ldiaxhog = **0** mm

Cover to outer tension reinforcement

ctenxhog = h - dxhog - Dxhog / 2 = **18.0** mm

Nominal cover to links steel

cnomxhog = ctenxhog - Ldiaxhog = **18.0** mm

Permissable minimum nominal cover to all reinforcement (Table 3.4)

; cmin = **15** mm

Cover OK over steel resisting hogging

;

;

BS Main Parking Beam

\_exVarsLib = "$(SysLbrDir)RC beam design-BS8110-si-engb.lbr"

\_exVarsItem = "Example variables 01"

Eval( if( GetVar("\_exFirstCalculate", True) == True, "EvalCalcItem(\_exVarsLib, \_exVarsItem)", "" )) = **0.000**

\_exFirstCalculate = False

RC beam analysis & design BS8110

TEDDS calculation version 2.1.14

Library item – RC beam design title



Library item: Show beam analysis drawing



Library item: Show beam analysis drawing



Library item: Show beam analysis drawing



Library item: Show beam analysis drawing



Library item: Show beam analysis drawing



Library item: Show beam analysis drawing

Support conditions

(Library item – Support conditions header)

Support A Vertically restrained

Rotationally restrained

Library item: Support conditions description

Support B Vertically restrained

Rotationally restrained

Library item: Support conditions description

Applied loading

Library item: Applied load heading

Dead full UDL 25.8 kN/m

Library item: Beam load description

Dead full UDL 20 kN/m

Library item: Beam load description

Load combinations

Library item: Load combinations heading

Load combination 1 Support A Dead  1.40

Library item: Load combination output

Imposed  1.60

Library item: Load combination output

Span 1 Dead  1.40

Library item: Load combination output

Imposed  1.60

Library item: Load combination output

Support B Dead  1.40

Library item: Load combination output

Imposed  1.60

Library item: Load combination output

Analysis results

(Library item – Analysis results header)

Maximum moment support A; MA\_max = **-134** kNm; MA\_red = **-134** kNm;

(Library item – Moment analysis results)

Maximum moment span 1 at 2500 mm; Ms1\_max = **67** kNm; Ms1\_red = **67** kNm;

(Library item – Moment analysis results)

Maximum moment support B; MB\_max = **-134** kNm; MB\_red = **-134** kNm;

(Library item – Moment analysis results)

Maximum shear support A; VA\_max = **160** kN; VA\_red = **160** kN

(Library item – Shear analysis results)

Maximum shear support A span 1 at 452 mm; VA\_s1\_max = **131** kN; VA\_s1\_red = **131** kN

(Library item – Shear analysis results)

Maximum shear support B; VB\_max = **-160** kN; VB\_red = **-160** kN

(Library item – Shear analysis results)

Maximum shear support B span 1 at 4548 mm; VB\_s1\_max = **-131** kN; VB\_s1\_red = **-131** kN

(Library item – Shear analysis results)

Maximum reaction at support A; RA = **160** kN

(Library item – Reaction analysis results)

Unfactored dead load reaction at support A; RA\_Dead = **115** kN

(Library item – Support reaction by load)

Maximum reaction at support B; RB = **160** kN

(Library item – Reaction analysis results)

Unfactored dead load reaction at support B; RB\_Dead = **115** kN

(Library item – Support reaction by load)

Rectangular section details

Section width; b = **300** mm

Section depth; h = **500** mm

Library item – Rectangular section details



Library item : Show section preview sketch

Concrete details

Concrete strength class; **C25/30**

Characteristic compressive cube strength; fcu = **30** N/mm2

Modulus of elasticity of concrete; Ec = 20kN/mm2 + 200 × fcu = **26000** N/mm2

Maximum aggregate size; hagg = **20** mm

Reinforcement details

Characteristic yield strength of reinforcement; fy = **500** N/mm2

Characteristic yield strength of shear reinforcement; fyv = **500** N/mm2

Nominal cover to reinforcement

Nominal cover to top reinforcement; cnom\_t = **30** mm

Nominal cover to bottom reinforcement; cnom\_b = **30** mm

Nominal cover to side reinforcement; cnom\_s = **30** mm

Library item – Material details

Support A

Library item;Section header



Library item : Show section preview sketch

Rectangular section in flexure (cl.3.4.4)

Design bending moment; M = abs(MA\_red) = **134** kNm

Depth to tension reinforcement; d = h - cnom\_t - v - top / 2 = **452** mm

Redistribution ratio; b = min(1 - mrA, 1) = **1.000**

K = M / (b × d2 × fcu) = **0.073**

K' = 0.156

K' > K - No compression reinforcement is required

Library item: Rectangular end check output

Lever arm; z = min(d  (0.5 + (0.25 - K / 0.9)0.5), 0.95  d) = **412** mm

Depth of neutral axis; x = (d - z) / 0.45 = **89** mm

Area of tension reinforcement required; As,req = M / (0.87  fy  z) = **745** mm2

Tension reinforcement provided; 3  20 bars

Area of tension reinforcement provided; As,prov = **942** mm2

Minimum area of reinforcement; As,min = 0.0013  b  h = **195** mm2

Maximum area of reinforcement; As,max = 0.04  b  h = **6000** mm2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Library item: Rectangular single output

Rectangular section in shear

Library item: Rectangular shear header

Design shear force span 1 at 452 mm; V = max(VA\_s1\_max, VA\_s1\_red) = **131** kN

Design shear stress; v = V / (b × d) = **0.968** N/mm2

Design concrete shear stress; vc = 0.79N/mm2 × min(3,[100 × As,prov / (b × d)]1/3) × max(1, (400mm /d)1/4) × (min(fcu, 40N/mm2) / 25N/mm2)1/3 / m

vc = **0.595** N/mm2

Allowable design shear stress; vmax = min(0.8 N/mm2 × (fcu/1 N/mm2)0.5, 5 N/mm2) = **4.382** N/mm2

PASS - Design shear stress is less than maximum allowable

Value of v from Table 3.7; 0.5  vc < v < (vc + 0.4 N/mm2)

Design shear resistance required; vs = max(v - vc, 0.4 N/mm2) = **0.400** N/mm2

Area of shear reinforcement required; Asv,req = vs × b / (0.87 × fyv) = **276** mm2/m

Shear reinforcement provided; 2  8 legs at 300 c/c

Area of shear reinforcement provided; Asv,prov = **335** mm2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing; svl,max = 0.75  d = **339** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Library item: Rectangular shear output

Spacing of reinforcement (cl 3.12.11)

Actual distance between bars in tension; s = (b - 2  (cnom\_s + v + top/2)) /(Ntop - 1) - top = **82** mm

Minimum distance between bars in tension (cl 3.12.11.1)

Minimum distance between bars in tension; smin = hagg + 5 mm = **25** mm

PASS - Satisfies the minimum spacing criteria

Maximum distance between bars in tension (cl 3.12.11.2)

Design service stress; fs = (2 × fy × As,req) / (3 × As,prov × b) = **263.6** N/mm2

Maximum distance between bars in tension; smax = min(47000 N/mm / fs, 300 mm) = **178** mm

PASS - Satisfies the maximum spacing criteria

Library item – Crack control output

Mid span 1

Library item;Section header



Library item : Show section preview sketch

Design moment resistance of rectangular section (cl. 3.4.4) - Positive moment

Design bending moment; M = abs(Ms1\_red) = **67** kNm

Depth to tension reinforcement; d = h - cnom\_b - v - bot / 2 = **454** mm

Redistribution ratio; b = min(1 - mrs1, 1) = **1.000**

K = M / (b × d2 × fcu) = **0.036**

K' = 0.156

K' > K - No compression reinforcement is required

Library item: Rectangular check output

Lever arm; z = min(d  (0.5 + (0.25 - K / 0.9)0.5), 0.95  d) = **431** mm

Depth of neutral axis; x = (d - z) / 0.45 = **50** mm

Area of tension reinforcement required; As,req = M / (0.87  fy  z) = **356** mm2

Tension reinforcement provided; 3  16 bars

Area of tension reinforcement provided; As,prov = **603** mm2

Minimum area of reinforcement; As,min = 0.0013  b  h = **195** mm2

Maximum area of reinforcement; As,max = 0.04  b  h = **6000** mm2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Library item: Rectangular single output

Rectangular section in shear

Library item: Rectangular shear header

Shear reinforcement provided; 2  8 legs at 300 c/c

Area of shear reinforcement provided; Asv,prov = **335** mm2/m

Minimum area of shear reinforcement (Table 3.7); Asv,min = 0.4N/mm2  b / (0.87  fyv) = **276** mm2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing (cl. 3.4.5.5); svl,max = 0.75  d = **340** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Design concrete shear stress; vc = 0.79N/mm2  min(3,[100  As,prov / (b  d)]1/3)  max(1, (400mm /d)1/4)  (min(fcu, 40N/mm2) / 25N/mm2)1/3 / m = **0.512** N/mm2

Design shear resistance provided; vs,prov = Asv,prov  0.87  fyv / b = **0.486** N/mm2

Design shear stress provided; vprov = vs,prov + vc = **0.998** N/mm2

Design shear resistance; Vprov = vprov  (b  d) = **135.9** kN

Shear links provided valid between 400 mm and 4600 mm with tension reinforcement of 603 mm2

Library item: Rectangular min shear output

Spacing of reinforcement (cl 3.12.11)

Actual distance between bars in tension; s = (b - 2  (cnom\_s + v + bot/2)) /(Nbot - 1) - bot = **88** mm

Minimum distance between bars in tension (cl 3.12.11.1)

Minimum distance between bars in tension; smin = hagg + 5 mm = **25** mm

PASS - Satisfies the minimum spacing criteria

Maximum distance between bars in tension (cl 3.12.11.2)

Design service stress; fs = (2 × fy × As,req) / (3 × As,prov × b) = **196.7** N/mm2

Maximum distance between bars in tension; smax = min(47000 N/mm / fs, 300 mm) = **239** mm

PASS - Satisfies the maximum spacing criteria

Library item – Crack control output

Span to depth ratio (cl. 3.4.6)

Basic span to depth ratio (Table 3.9); span\_to\_depthbasic = **20.0**

Design service stress in tension reinforcement; fs = (2  fy  As,req)/ (3  As,prov  b) = **196.7** N/mm2

Modification for tension reinforcement

ftens = min(2.0, 0.55 + (477N/mm2 - fs) / (120  (0.9N/mm2 + (M / (b  d2))))) = **1.729**

Modification for compression reinforcement

fcomp = min(1.5, 1 + (100  As2,prov / (b  d)) / (3 + (100  As2,prov / (b  d)))) = **1.052**

Modification for span length; flong = 1.000

Allowable span to depth ratio; span\_to\_depthallow = span\_to\_depthbasic  ftens  fcomp = **36.4**

Actual span to depth ratio; span\_to\_depthactual = Ls1 / d = **11.0**

PASS - Actual span to depth ratio is within the allowable limit

Library item – Deflection control output

Support B

Library item;Section header



Library item : Show section preview sketch

Rectangular section in flexure (cl.3.4.4)

Design bending moment; M = abs(MB\_red) = **134** kNm

Depth to tension reinforcement; d = h - cnom\_t - v - top / 2 = **452** mm

Redistribution ratio; b = min(1 - mrB, 1) = **1.000**

K = M / (b × d2 × fcu) = **0.073**

K' = 0.156

K' > K - No compression reinforcement is required

Library item: Rectangular end check output

Lever arm; z = min(d  (0.5 + (0.25 - K / 0.9)0.5), 0.95  d) = **412** mm

Depth of neutral axis; x = (d - z) / 0.45 = **89** mm

Area of tension reinforcement required; As,req = M / (0.87  fy  z) = **745** mm2

Tension reinforcement provided; 3  20 bars

Area of tension reinforcement provided; As,prov = **942** mm2

Minimum area of reinforcement; As,min = 0.0013  b  h = **195** mm2

Maximum area of reinforcement; As,max = 0.04  b  h = **6000** mm2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Library item: Rectangular single output

Rectangular section in shear

Library item: Rectangular shear header

Design shear force span 1 at 4548 mm; V = abs(min(VB\_s1\_max, VB\_s1\_red)) = **131** kN

Design shear stress; v = V / (b × d) = **0.968** N/mm2

Design concrete shear stress; vc = 0.79N/mm2 × min(3,[100 × As,prov / (b × d)]1/3) × max(1, (400mm /d)1/4) × (min(fcu, 40N/mm2) / 25N/mm2)1/3 / m

vc = **0.595** N/mm2

Allowable design shear stress; vmax = min(0.8 N/mm2 × (fcu/1 N/mm2)0.5, 5 N/mm2) = **4.382** N/mm2

PASS - Design shear stress is less than maximum allowable

Value of v from Table 3.7; 0.5  vc < v < (vc + 0.4 N/mm2)

Design shear resistance required; vs = max(v - vc, 0.4 N/mm2) = **0.400** N/mm2

Area of shear reinforcement required; Asv,req = vs × b / (0.87 × fyv) = **276** mm2/m

Shear reinforcement provided; 2  8 legs at 300 c/c

Area of shear reinforcement provided; Asv,prov = **335** mm2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing; svl,max = 0.75  d = **339** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Library item: Rectangular shear output

Spacing of reinforcement (cl 3.12.11)

Actual distance between bars in tension; s = (b - 2  (cnom\_s + v + top/2)) /(Ntop - 1) - top = **82** mm

Minimum distance between bars in tension (cl 3.12.11.1)

Minimum distance between bars in tension; smin = hagg + 5 mm = **25** mm

PASS - Satisfies the minimum spacing criteria

Maximum distance between bars in tension (cl 3.12.11.2)

Design service stress; fs = (2 × fy × As,req) / (3 × As,prov × b) = **263.6** N/mm2

Maximum distance between bars in tension; smax = min(47000 N/mm / fs, 300 mm) = **178** mm

PASS - Satisfies the maximum spacing criteria

Library item – Crack control output

;

;

BS RC parking column

\_exVarsLib = “$(SysLbrDir)RC column design-BS8110-si-engb.lbr”

\_exVarsItem = ”Example variables 03”

Eval( if( GetVar(“\_exFirstCalculate”, True) == True, "EvalCalcItem(\_exVarsLib, \_exVarsItem)", "" )) = **0.000**

\_exFirstCalculate = False

RC column design (Bs8110:Part1:1997)

TEDDS calculation version 2.0.08

Library item: RC column design (BS8110)

Column definition

Column dimensions, depth to steel (assumed symmetrical)

Column depth (larger column dim); h = **550** mm

Nominal cover to all reinforcement (longer dim); ch = **50** mm

Trial bar diameter; Dcol = **25** mm

Trial link diameter; Ldia = **8** mm

Depth to tension steel; h' = h - ch – Ldia – Dcol/2 = **480** mm

Column width (smaller column dim); b = **350** mm

Nominal cover to all reinforcement (shorter dim); cb = **50** mm

Depth to tension steel; b' = b - cb - Ldia – Dcol/2 = **280** mm

Characteristic strength of reinforcement; fy = **500** N/mm2

Characteristic strength of concrete; fcu = **40** N/mm2

Partial safety factor for strength of concrete; mc = **1.50**

Partial safety factor for strength of steel; ms = **1.15**

Library item : Definition of rectangular column data



Library item : Show sketch

Unbraced Column Design to cl 3.8.4

Check on overall column dimensions

Column OK - h < 4b

Unbraced column slenderness check

Unbraced column clear height; lo = **3000** mm

Slenderness limit; llimit = 60  b = **21000** mm

Slenderness limit; llimit1 = 100  b2 / h = **22273** mm

Column slenderness limit OK

If column is unrestrained, slenderness limit OK

Short column check for unbraced columns

Column clear height; lo = **3000** mm

Effect height factor for unbraced columns - maj axis; x = **1.20**

BS8110:Table 3.20

Effective height (major axis); lex = x  lo = **3.600** m

Slenderness check; lex/h = **6.55**

The unbraced column is short (major axis)

Effect height factor for unbraced columns - min axis; y = **1.20**

BS8110:Table 3.20

Effective height (minor axis); ley = y  lo = **3.600** m

Slenderness check; ley/b = **10.29**

The unbraced column is slender (minor axis)

Unbraced slender column (minor axis) - bi-axial bending

Define column reinforcement

Main reinforcement in column

Assumed diameter of main reinforcement; Dcol = **25** mm

Assumed no. of bars in one face (assumed sym); Lncol = **4**

Area of "tension" steel; Ast = Lncol    Dcol2 / 4 = **1963** mm2

Area of compression steel; Asc = Ast = **1963** mm2

Total area of steel ; Ascol =   Dcol2 / 4  2  (Lncol + (Lncol -2)) = **5890.5** mm2

Percentage of steel; Ascol / (b  h) = **3.1** %

Design ultimate loading

Design ultimate axial load; N = **2000** kN

Initial end moment (major axis); Mx1 = **100** kNm

Initial end moment (minor axis); My1 = **100** kNm

Initial moment approx (major axis); Mix =abs( Mx1 ) = **100.0** kNm

Initial moment approx (minor axis); Miy = abs( My1 ) = **100.0** kNm

Additional moment

ay = ley2 / (2000  b2 ) = **0.05**

Reduction factor to correct deflection for axial load

Nuz = 0.45  fcu  h  b + 1/ms fy  Ascol = **6026.1** kN

Nbal = 0.25  fcu  h  b' = **1537.3** kN

K = min((Nuz - N)/(Nuz - Nbal), 1.0) = **0.90**

auy = ay  K  b = **16.6** mm

Additional moment; Maddy = N  auy = **33.2** kNm

Minimum design moments

Min design moment (Major axis); Mxmin = min(0.05  h, 20 mm)  N = **40.0** kNm

Min design moment (Minor axis); Mymin = min(0.05  b, 20 mm)  N = **35.0** kNm

Design moments

Design moment (major axis); Mxdes = max (Mix , Mxmin) = **100.0** kNm

Design moment (minor axis); Mydes = max (Miy + Maddy, Mymin) = **133.2** kNm

Simplified method for dealing with bi-axial bending:-

h' = **480** mm; b' = **280** mm

Approx uniaxial design moment (Cl 3.8.4.5)

 = 1 - 1.165  min(0.6, N/(bhfcu)) = **0.70**

Design moment;

Mdesign = if(Mxdes/h' < Mydes/b' , Mydes +   b'/h'  Mxdes , Mxdes +   h'/b'  Mydes ) = **173.9** kNm

Set up section dimensions for design:-

Section depth; D = if(Mxdes/h' < Mydes/b' , b, h) = **350.0** mm

Depth to "tension" steel; d = if(Mxdes/h' < Mydes/b' , b', h') = **279.5** mm

Section width; B = if(Mxdes/h' < Mydes/b' , h, b) = **550.0** mm

Library item - Calcs – unbra sl col N+Mmaj+Mmin\*

Check of design forces - symmetrically reinforced section

Note

The following calculations determine the bending capacity of the column given that an axial load N is present. They result in the same solution as obtained from the design charts in BS8110:Pt 3 which assume a symmetrically reinforced column.

The calculations assume a simplified concrete stress block of 0.9X deep and have a lower limit of axial load applied such that the compression steel never goes into tension.

The calculations look at 7 potential solutions with the concrete at its ultimate strain and determine which one is applicable.

Note:- the section dimensions used in the following calculation are:-

Section width (parallel to axis of bending); B = **550** mm

Section depth perpendicular to axis of bending); D = **350** mm

Depth to "tension" steel (symmetrical); d = **280** mm

Material limits

Yield strain for steel; y = fy / (ms ES8110) = **0.0022**

"Yield" strain for concrete; 0 = 2.4  10-4   (fcu/((1 N/mm2)  mc)) = **0.0012**

Strain limit; slimit = fy / (ms  ES8110) = **0.0022**

Determine the mode of failure: Assuming simplified stress block

Case 1 - Assume tension failure in steel

Solve for x in equation N equals (0.67fcu/mc) B 0.9 x + Asc fsc + Astfst assuming fst is fym

A1 = (0.67  fcu / mc)  0.9  B = **8844.0** N/mm

B1 = ES8110  0.0035  Asc - fy / ms  Ast - N = **-1479.2** kN

C1 = ES8110  0.0035  Asc  (d-D) = **-96.9** kNm

xtry1 = [-1  B1 +  (B12 - 4  A1  C1)] / (2  A1) = **217.6** mm

Resulting steel strains

sttry1 = 0.0035  (d - xtry1) / xtry1 = **0.0010**

sctry1 = 0.0035  (xtry1 - (D - d)) / xtry1 = **0.0024**

Chk1 = if( sctry1>slimit ,"Not OK",if( sttry1<slimit ,"Not OK", if( sctry1<-slimit ,"Not OK","OK"))) = **"Not OK"**

Case 2 - Assume both tension and compression failure in steel

Solve for x in equation N equals (0.67fcu/mc) B 0.9x + Asc fsc + Astfst assuming fst and fsc are fym

xtry2 = [ N + fy / ms  Ast - fy / ms  Asc ] / ((0.67  fcu / mc)  0.9  B) = **226.1** mm

Resulting steel strains

sttry2 = 0.0035  (d - xtry2) / xtry2 = **0.0008**

sctry2 = 0.0035  (xtry2 - (D - d)) / xtry2 = **0.0024**

Chk2 = if( sctry2<slimit ,"Not OK", if( sttry2<slimit ,"Not OK","OK")) = **"Not OK"**

Case 3 - Assume Compression steel failure (0.9x<D)

Solve for x in equation N equals (0.67fcu/mc) B 0.9x + Asc fsc + Astfst assuming fst < fym and fsc is fym

A3 = (0.67  fcu / mc)  0.9  B = **8844.0** N/mm

B3 = fy / ms  Asc + 0.0035  ES8110  Ast - N = **228.1** kN

C3 = -1  ES8110  0.0035  d  Ast = **-384.2** kNm

xtry3 = [-1  B3 +  (B32 - 4  A3  C3)] / (2  A3) = **195.9** mm

Resulting steel strains

sttry3 = 0.0035  (d - xtry3)/ xtry3 = **0.0015**

sctry3 = 0.0035  (xtry3 - (D - d)) / xtry3 = **0.0022**

Chk3 = if(xtry3>D/0.9,"Not OK",if( sttry3>slimit ,"Not OK",if( sctry3<slimit ,"Not OK","OK"))) = **"OK"**

Case 4 - Assume Compression Steel failure (0.9x>D)

Solve for x in equation N equals (0.67fcu/mc) B D + Asc fsc + Astfst assuming fst < fym and fsc is fym

xtry4 = 0.0035  d  ES8110  Ast / ( DB0.67fcu/mc + Ascfy/ms + 0.0035ES8110Ast -N) = **104.7** mm

Resulting steel strains

sttry4 = 0.0035  (d - xtry4)/ xtry4 = **0.0058**

sctry4 = 0.0035  (xtry4 - (D - d)) / xtry4 = **0.0011**

Chk4 = if(xtry4<D/0.9 ,"Not OK",if( sttry4>slimit ,"Not OK",if( sctry4<slimit ,"Not OK","OK"))) = **"Not OK"**

Case 5 - Assume no steel yields at failure (0.9x≤D)

Solve for x in equation N equals (0.67fcu/mc) B 0.9x + Asc fsc + Astfst assuming fst < fym and fsc < fym

A5 = (0.67  fcu / mc)  0.9  B = **8844.0** N/mm

B5 = 0.0035  ES8110  (Asc + Ast) - N = **748.9** kN

C5 = -1  ES8110  0.0035  (d  Ast + (D - d) × Asc) = **-481.1** kNm

xtry5 = [-1  B5 +  (B52 - 4  A5  C5)] / (2  A5) = **194.7** mm

Resulting steel strains

sttry5 = 0.0035  (d - xtry5)/ xtry5 = **0.0015**

sctry5 = 0.0035  (xtry5 - (D - d)) / xtry5 = **0.0022**

Chk5 = if(xtry5>D/0.9,"Not OK",if( sttry5>slimit ,"Not OK",if( sctry5>slimit ,"Not OK","OK"))) = **"Not OK"**

Case 6 - Assume no steel yields at failure (0.9x>D)

Solve for x in equation N equals (0.67fcu/mc) B D + Asc fsc + Astfst assuming fst < fym and fsc < fym

xtry6 = 0.0035ES8110(dAst+(D-d)×Asc)/(DB0.67fcu/mc+ 0.0035×ES8110×Asc+0.0035ES8110Ast-N)

ie; xtry6 = **114.9** mm

Resulting steel strains

sttry6 = 0.0035  (d - xtry6)/ xtry6 = **0.0050**

sctry6 = 0.0035  (xtry6 - (D - d)) / xtry6 = **0.0014**

Chk6 = if(xtry6<D/0.9,"Not OK",if( sttry6>slimit ,"Not OK",if( sctry6>slimit ,"Not OK","OK"))) = **"Not OK"**

Case 7 - Assume both tension and compression steel fail in tension

Solve for x in equation N equals (0.67fcu/mc) B 0.9x - Asc fsc + Astfst assuming fst and fsc are fym

xtry7 = [ N + fy / ms  Ast + fy / ms  Asc ] / ((0.67  fcu / mc)  0.9  B) = **419.2** mm

Resulting steel strains

sttry7 = 0.0035  (d - xtry7) / xtry7 = **-0.0012**

sctry7 = 0.0035  (xtry7 - (D - d)) / xtry7 = **0.0029**

Chk7 = if( sctry7>-slimit ,"Not OK", if( sttry7<slimit ,"Not OK","OK")) = **"Not OK"**

Case 8 - Assume All Steel Compression Steel failure

Limiting case occurs when

xtry8 = 0.0035  d / ( 0.0035 - slimit ) = **737.7** mm

Resulting steel strains

sttry8 = 0.0035  (d - xtry8) / xtry8 = **-0.0022**

sctry8 = 0.0035  (xtry8 - (D - d)) / xtry8 = **0.0032**

Determine true solution

≤Compression steel yields (0.9x<D)

Use the correct value of fst , fsc and x to determine MR

xcalc = if(Chk1 == "OK", xtry1 ,if(Chk2 == "OK", xtry2 ,if(Chk3 == "OK", xtry3 ,if(Chk4 == "OK", xtry4 ,if(Chk5 == "OK", xtry5,if(Chk6 == "OK", xtry6 , if(Chk7 == "OK" , xtry7 , xtry8 )))))))

xcalc = **195.9** mm

st = if(Chk1 == "OK", sttry1 ,if(Chk2 == "OK", sttry2 ,if(Chk3 == "OK", sttry3 ,if(Chk4 == "OK", sttry4 ,if(Chk5 == "OK", sttry5 ,if(Chk6 == "OK", sttry6 , if(Chk7 == "OK" , sttry7 , sttry8 )))))))

st = **0.0015**

sc = if(Chk1 == "OK", sctry1 ,if(Chk2 == "OK", sctry2 ,if(Chk3 == "OK", sctry3 ,if(Chk4 == "OK", sctry4 ,if(Chk5 == "OK", sctry5 ,if(Chk6 =="OK", sctry6 , if(Chk7 == "OK", sctry7 , sctry8 )))))))

sc = **0.0022**

Resulting steel stresses

fst = max( -fy / ms , min(ES8110  st , fy / ms) ) = **298.6** N/mm2

fsc = max( -fy / ms , min(ES8110  sc , fy / ms) ) = **434.8** N/mm2

For NA depth xcalc < 0.9D

NR1 = 0.67 × fcu / mc  0.9  xcalc  B + Asc  fsc - Ast  fst = **2000.0** kN

Substitute the relevant values in equation for M (about centre of section)

MR1 = (0.67×fcu/mc)0.9xcalc(D/2-(0.9xcalc)/2)B+Ascfsc(d-D/2)+Astfst(d-D/2) = **300.9** kNm

For NA depth xcalc > 0.9D

NR2 = (0.67 × fcu / mc) D  B + Asc  fsc - Ast  fst = **3706.6** kN

Substitute the relevant values in equation for M (about centre of section)

MR2 = Ascfsc(d-D/2) + Astfst(d-D/2) = **150.5** kNm

Determine correct moment of resistance

NR =ceiling(if(xcalc<D/0.9, NR1 , NR2 ),0.001kN) = **2000.0** kN

MR = ceiling(if(xcalc<D/0.9, MR1 , MR2 ) ,0.001kNm) = **301.0** kNm

Applied axial load; N = **2000.0** kN

Check for moment; Mdesign = **173.9** kNm

Moment check satisfied

Moment capacity with no axial load

Solve for x in equation (0.67fcu/mc) B 0.9 x + Asc fsc + Ast fym equals zero

A9 = (0.67  fcu / mc)  0.9  B = **8844.0** N/mm

B9 = ES8110  0.0035  Asc - fy / ms  Ast = **520.8** kN

C9 = ES8110  0.0035  Asc  (d-D) = **-96.9** kNm

x9 = [-1  B9 +  (B92 - 4  A9  C9)] / (2  A9) = **79.3** mm

Resulting steel strains

st9 = 0.0035  (d - x9) / x9 = **0.0088**

sc9 = 0.0035  (x9 - (D - d)) / x9 = **0.0004**

Resulting steel stresses

fst9 = min(ES8110  st9 , fy / ms) = **434.8** N/mm2

fsc9 = min(ES8110  sc9 , fy / ms) = **77.6** N/mm2

MR9 = 0.45fcu0.9  x9  (D/2-(0.9x9)/2)  B + Ascfsc9(d-D/2) + Astfst9(d-D/2)

MR9 = **203.6** kNm

Check min and max areas of steel

Total area of concrete; Aconc = b  h = **192500** mm2

Area of steel (symmetrical); Ascol = **5890** mm2

Minimum percentage of compression reinforcement; kc = **0.40** %

Minimum steel area; Asc\_min = kc  Aconc = **770** mm2

Maximum steel area; As\_max = 6 %  Aconc = **11550** mm2

Area of compression steel provided OK

Major axis Shear Resistance of Concrete Columns - (cl 3.8.4.6)

Column width; b = **350** mm

Column depth; h = **550** mm

Effective depth to steel; h' = **480** mm

Area of concrete; Aconc = b  h = **192500** mm2

Design ultimate shear force (major axis); Vx = **75** kN

Characteristic strength of concrete; fcu = **40** N/mm2

Is a check required? (3.8.4.6)

Axial load; N = **2000.0** kN

Major axis moment; Mx = **173.9** kNm

Eccentricity; e = Mx / N = **86.9** mm

Limit to eccentricity; elimit = 0.6  h = **330.0** mm

Actual shear stress; vx = Vx / (b  h') = **0.4** N/mm2

Allowable stress; vallowable = min ((0.8 N1/2/mm)  (fcu ), 5 N/mm2 ) = **5.000** N/mm2

No shear check required

**Define Containment Links Provided**

Link spacing; sv = **250** mm; Link diameter; Ldia = **8** mm; No of links in each group; Ln = **12**

Minimum Containment Steel (Cl 3.12.7)

Shear steel

Link spacing; sv = **250** mm

Link diameter; Ldia = **8** mm

Column steel

Diameter; Dcol = **25** mm

Min diameter; Llimit = max( (6 mm), Dcol/4) = **6.3** mm

Link diameter OK

Max spacing; slimit = 12  Dcol = **300.0** mm

Link spacing OK

Crack Control in columns - is a check required? (Cl 3.8.6)

Column design ultimate axial load; N = **2000.0** kN

Column dimensions

Column depth (larger column dimension); h = **550** mm

Column width (smaller column dimension); b = **350** mm

Column area; Ac = h  b = **192500** mm2

Limit for crack check; Nlimit = 0.2  fcu  Ac = **1540.0** kN

No crack checks required

Serviceability Limit State - Cracking in Columns

Bent about the major axis

(BS8110:Pt 2, Cl. 3.8 & BS8007 Cl 2.6 & Appendix B)

The following calculations ignore the presence of compression steel and axial load.

Design serviceability moment about the major axis; MX\_SLS = **50** kNm

Column dimensions, depth to steel (assumed symmetrical)

Column depth (larger column dimension); h = **550** mm

Depth to steel; h' = **480** mm

Column width (smaller column dimension); b = **350** mm

Characteristic strength of concrete; fcu = **40** N/mm2

Characteristic strength of reinforcement; fy = **500** N/mm2

BS8110:Pt 1:Table 3.1

Diameter of links; Ldia = **8** mm

Diameter of tension reinforcement; Dcol = **25** mm

Number of tension reinforcement bars; Lncol = **4**

Area of tension reinforcement; Ast =   Dcol2 /4  Lncol = **1963** mm2

Nominal cover to reinforcement; cnom = h - h' - Dcol/2 - Ldia = **50** mm

Cover to tension reinforcement; cten = cnom + Ldia = **58.0** mm

Effective depth to tension reinforcement; h' = **479.5** mm

Tension bar centres; barcrs = (b - 2(cnom + Ldia) - Dcol ) / (Lncol - 1) = **69.7** mm

Modular Ratio

Modulus of elasticity for reinforcement; Es = 200 kN/mm2

BS8110:Pt 1:Cl 2.5.4

Modulus of elast for conc (half the instanteneous); Ec = ((20 kN/mm2) + 200fcu) / 2 = **14** kN/mm2

BS8110:Pt 2:Equation 17

Modular ratio; m = Es / Ec = **14.286**

Neutral Axis position

For equilibrium; Fst equates Fc

Therefore: m  Ast  [ fc(h'-x)/x ] equates to 0.5  fc  b  x

Solving for x gives the position of the neutral axis in the section:-

x = h'  [ -1EsAst/(Ecbh') + ( EsAst/(Ecbh')  (2+EsAst/(Ecbh')))] = **208.4** mm

Depth of concrete in compression; x = **208.4** mm

Concrete and Steel stresses

The serviceability limit state moment; MX\_SLS = **50** kNm

Taking moments about the centreline of the reinforcement:-

Moment of resistance of concrete is 0.5  fc  b  x  (h' - x/3)

Solving for concrete stress fc gives; fc = 2  MX\_SLS / ( b  x  (h' - x/3)) = **3.34** N/mm2

Allowable stress; 0.45  fcu = **18.00** N/mm2

Concrete stress OK

Taking moments about the centre of action of the concrete force:-

Moment of resistance of steel is fst  As  (h' - x/3)

Solving for steel stress fst gives; fst = MX\_SLS / ( Ast  (h' - x/3)) = **62.11** N/mm2

Concrete and Steel strains

Strain in the reinforcement; s = fst / Es = **310.510-6**

Allowable steel strain; 0.8  fy / Es = **2.00010-3**

Steel strain OK

BS8007:App B.4

Strain in the concrete at the level at which crack width is required

Level of crack; a' = h = **550** mm

1 = s  (a' - x)/(h' - x) = **391.310-6**

Strain in the concrete at the level at which crack width is required adjusted for stiffening of the concrete tension zone

Allowable crack width; CrackAllowable = **2.0** mm

BS8007:Cl 2.2.3.3

Factor for stiffening based on limitiing crack width; factor = if(CrackAllowable <= 0.1 mm, 1.5 N/mm2, if(CrackAllowable <= 0.2 mm, 1.0 N/mm2, 1.0 N/mm2)) = **1.0** N/mm2

Breadth of tension face; bt = b = **350** mm

m = min( 1 ,max(0, 1 - [ factor  bt (h - x)  (a' - x) / (3Es Ast (h' - x))])) = **263.410-6**

BS8007:App B.3

Distance from tension bar to crack in tension face between tension bars

acr1 =  ( (barcrs/2)2 + (cnom + Ldia + Dcol/2)2) - Dcol/2 = **66.1** mm

Distance from tension bar to crack in tension face at corner of column

acr2 = (2)  (cnom + Ldia + Dcol/2 ) - Dcol/2 = **87.2** mm

Critical distance from tension bar; acr = max(acr1, acr2 ) = **87.2** mm

Design crack width; Crackdesign = 3  acr  m /(1 + 2(acr - cten)/(h - x)) = **0.059** mm

Crackdesignmaj = Crackdesign = **0.000**

BS8007:App B.2

Max allowable crack width; CrackAllowable = **2.00** mm

BS8007:Cl 2.2.3.3

Library item – Calcs – column cracking (major)

Serviceability Limit State - Cracking in Columns

Bent about the minor axis

(BS8110:Pt 2, Cl. 3.8 & BS8007 Cl 2.6 & Appendix B)

The following calculations ignore the presence of compression steel and axial load.

Design serviceability moment about the minor axis; MY\_SLS = **50** kNm

Column dimensions, depth to steel (assumed symmetrical)

Column depth (larger column dimension); h = **550** mm

Column width (smaller column dimension); b = **350** mm

Depth to steel; b' = **280** mm

Characteristic strength of concrete; fcu = **40** N/mm2

Characteristic strength of reinforcement; fy = **500** N/mm2

BS8110:Pt 1:Table 3.1

Diameter of links; Ldia = **8** mm

Diameter of tension reinforcement; Dcol = **25** mm

Number of tension reinforcement bars; Lncol = **4**

Area of tension reinforcement; Ast =   Dcol2 /4  Lncol = **1963** mm2

Nominal cover to reinforcement; cnom = b - b' - Dcol/2 - Ldia = **50** mm

Cover to tension reinforcement; cten = cnom + Ldia = **58.0** mm

Effective depth to tension reinforcement; b' = **279.5** mm

Tension bar centres; barcrs = (h - 2(cnom + Ldia) - Dcol ) / (Lncol - 1) = **136.3** mm

Modular Ratio

Modulus of elasticity for reinforcement; Es = 200 kN/mm2

BS8110:Pt 1:Cl 2.5.4

Modulus of elasticity for conc (half instanteneous); Ec = ((20 kN/mm2) + 200fcu) / 2 = **14** kN/mm2

BS8110:Pt 2:Equation 17

Modular ratio; m = Es / Ec = **14.286**

Neutral Axis position

For equilibrium; Fst equates Fc

Therefore: m  Ast  [ fc(b'-x)/x ] equates to 0.5  fc  h  x

Solving for x gives the position of the neutral axis in the section:-

x = b'  [ -1EsAst/(Echb') + ( EsAst/(Echb')  (2+EsAst/(Echb')))] = **125.4** mm

Depth of concrete in compression; x = **125.4** mm

Concrete and Steel stresses

The serviceability limit state moment; MY\_SLS = **50** kNm

Taking moments about the centreline of the reinforcement:-

Moment of resistance of concrete is 0.5  fc  h  x  (b' - x/3)

Solving for concrete stress fc gives; fc = 2  MY\_SLS / ( h  x  (b' - x/3)) = **6.10** N/mm2

Allowable stress; 0.45  fcu = **18.00** N/mm2

Concrete stress OK

Taking moments about the centre of action of the concrete force:-

Moment of resistance of steel; fst  As  (b' - x/3)

Solving for steel stress fst gives; fst = MY\_SLS / ( Ast  (b' - x/3)) = **107.13** N/mm2

Concrete and Steel strains

Strain in the reinforcement; s = fst / Es = **535.610-6**

Allowable steel strain; 0.8  fy / Es = **2.00010-3**

Steel strain OK

BS8007:App B.4

Strain in the concrete at the level at which crack width is required

Level of crack; a' = b = **350** mm

1 = s  (a' - x)/(b' - x) = **780.710-6**

Strain in the concrete at the level at which crack width is required adjusted for stiffening of the concrete tension zone

Allowable crack width; CrackAllowable = **2.0** mm

BS8007:Cl 2.2.3.3

Factor for stiffening based on limitiing crack width; factor = if(CrackAllowable <= 0.1 mm, 1.5 N/mm2, if(CrackAllowable <= 0.2 mm, 1.0 N/mm2, 1.0 N/mm2)) = **1.0** N/mm2

Breadth of tension face; ht = h = **550** mm

m = min( 1 , max(0, 1 - [ factor  ht (b - x)  (a' - x) / (3Es Ast (b' - x))])) = **627.810-6**

BS8007:App B.3

Distance from tension bar to crack in tension face between tension bars

acr1 =  ( (barcrs/2)2 + (cnom + Ldia + Dcol/2)2) - Dcol/2 = **85.6** mm

Distance from tension bar to crack in tension face at corner of column

acr2 = (2)  (cnom + Ldia + Dcol/2 ) - Dcol/2 = **87.2** mm

Critical distance from tension bar; acr = max(acr1, acr2 ) = **87.2** mm

Design crack width; Crackdesign = 3  acr  m /(1 + 2(acr - cten)/(b - x)) = **0.130** mm

Crackdesignmin = Crackdesign = **0.000**

BS8007:App B.2

Max allowable crack width; CrackAllowable = **2.00** mm

BS8007:Cl 2.2.3.3

Library item – Calcs – column cracking (minor)

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BS Pad footing

Pad footing analysis and design (BS8110-1:1997)

Tedds calculation version 2.0.07

Library item: Pad footing analysis title



Library item : Show found output sketch

Pad footing details

Length of pad footing; L = **1800** mm

Width of pad footing; B = **1800** mm

Area of pad footing; A = L × B = **3.240** m2

Depth of pad footing; h = **1800** mm

Depth of soil over pad footing; hsoil = **700** mm

Density of concrete; conc = **24.5** kN/m3

Library item: Pad footing details

Column details

Column base length; lA = **550** mm

Column base width; bA = **350** mm

Column eccentricity in x; ePxA = **20** mm

Column eccentricity in y; ePyA = **20** mm

Library item: Column details

Soil details

Library item: Soil details title

Density of soil; soil = **20.0** kN/m3

Design shear strength; ’ = **25.0** deg

Design base friction;  = **19.3** deg

Allowable bearing pressure; Pbearing = **150** kN/m2

Library item: Soil details

Foundation loads

Dead surcharge load; FGsur = **0.000** kN/m2

Imposed surcharge load; FQsur = **0.000** kN/m2

Pad footing self weight; Fswt = h × conc = **44.100** kN/m2

Soil self weight; Fsoil = hsoil × soil = **14.000** kN/m2

Total foundation load; F = A × (FGsur + FQsur + Fswt + Fsoil) = **188.2** kN

Library item: Foundation load on pad

Calculate pad base reaction

Total base reaction; T = F + PA = **188.2** kN

Eccentricity of base reaction in x; eTx = (PA  ePxA + MxA + HxA  h) / T = **0** mm

Eccentricity of base reaction in y; eTy = (PA  ePyA + MyA + HyA  h) / T = **0** mm

Check pad base reaction eccentricity

abs(eTx) / L + abs(eTy) / B = **0.000**

Base reaction acts within middle third of base

Library item: Calculate pad base reaction

Calculate pad base pressures

q1 = T / A - 6  T  eTx / (L  A) - 6  T  eTy / (B  A) = **58.100** kN/m2

q2 = T / A - 6  T  eTx / (L  A) + 6  T  eTy / (B  A) = **58.100** kN/m2

q3 = T / A + 6  T  eTx / (L  A) - 6  T  eTy / (B  A) = **58.100** kN/m2

q4 = T / A + 6  T  eTx / (L  A) + 6  T  eTy / (B  A) = **58.100** kN/m2

Minimum base pressure; qmin = min(q1, q2, q3, q4) = **58.100** kN/m2

Maximum base pressure; qmax = max(q1, q2, q3, q4) = **58.100** kN/m2

PASS - Maximum base pressure is less than allowable bearing pressure

Library item: Calculate pad base pressures



Library item : Show pad found pressure sketch

Partial safety factors for loads

Partial safety factor for dead loads; fG = **1.40**

Partial safety factor for imposed loads; fQ = **1.60**

Partial safety factor for wind loads; fW = **0.00**

Library item: Partial safety factors

Ultimate axial loading on column

Ultimate axial load on column; PuA = PGA × fG + PQA × fQ + PWA × fW = **0.0** kN

Ultimate foundation loads

Ultimate foundation load; Fu = A × [(FGsur + Fswt + Fsoil) × fG + FQsur × fQ] = **263.5** kN

Ultimate horizontal loading on column

Ultimate horizontal load in x direction; HxuA = HGxA × fG + HQxA × fQ + HWxA × fW = **0.0** kN

Ultimate horizontal load in y direction; HyuA = HGyA × fG + HQyA × fQ + HWyA × fW = **0.0** kN

Ultimate moment on column

Ultimate moment on column in x direction; MxuA = MGxA × fG + MQxA × fQ + MWxA × fW = **0.000** kNm

Ultimate moment on column in y direction; MyuA = MGyA × fG + MQyA × fQ + MWyA × fW = **0.000** kNm

Library item: Ultimate loads column

Calculate ultimate pad base reaction

Ultimate base reaction; Tu = Fu + PuA = **263.5** kN

Eccentricity of ultimate base reaction in x; eTxu = (PuA × ePxA + MxuA + HxuA × h) / Tu = **0** mm

Eccentricity of ultimate base reaction in y; eTyu = (PuA × ePyA + MyuA + HyuA × h) / Tu = **0** mm

Library item: Ultimate base reaction

Calculate ultimate pad base pressures

q1u = Tu/A - 6TueTxu/(LA) - 6TueTyu/(BA) = **81.340** kN/m2

q2u = Tu/A - 6TueTxu/(LA) + 6Tu eTyu/(BA) = **81.340** kN/m2

q3u = Tu/A + 6TueTxu/(LA) - 6TueTyu/(BA) = **81.340** kN/m2

q4u = Tu/A + 6TueTxu/(LA) + 6TueTyu/(BA) = **81.340** kN/m2

Minimum ultimate base pressure; qminu = min(q1u, q2u, q3u, q4u) = **81.340** kN/m2

Maximum ultimate base pressure; qmaxu = max(q1u, q2u, q3u, q4u) = **81.340** kN/m2

Library item: Ultimate pad base pressures

Calculate rate of change of base pressure in x direction

Left hand base reaction; fuL = (q1u + q2u) × B / 2 = **146.412** kN/m

Right hand base reaction; fuR = (q3u + q4u) × B / 2 = **146.412** kN/m

Length of base reaction; Lx = L = **1800** mm

Rate of change of base pressure; Cx = (fuR - fuL) / Lx = **0.000** kN/m/m

Library item: Ultimate pad base reactions in x

Calculate pad lengths in x direction

Left hand length; LL = L / 2 + ePxA = **920** mm

Right hand length; LR = L / 2 - ePxA = **880** mm

Library item: Calculate pad lengths in x

Calculate ultimate moments in x direction

Ultimate positive moment in x direction; Mx = fuL  LL2 / 2 + Cx  LL3 / 6 - Fu  LL2 / (2  L) = **0.000** kNm

Position of maximum negative moment; Lz = **920** mm

Ultimate negative moment in x direction; Mxneg = fuL  LL2 / 2 + Cx  LL3 / 6 - Fu  LL2 / (2  L)

Mxneg = **0.000** kNm

Library item: Multiple moments in x direction

Calculate rate of change of base pressure in y direction

Top edge base reaction; fuT = (q2u + q4u) × L / 2 = **146.412** kN/m

Bottom edge base reaction; fuB = (q1u + q3u) × L / 2 = **146.412** kN/m

Length of base reaction; Ly = B = **1800** mm

Rate of change of base pressure; Cy = (fuB - fuT) / Ly = **0.000** kN/m/m

Library item: Ultimate pad base reactions in y

Calculate pad lengths in y direction

Top length; LT = B / 2 - ePyA = **880** mm

Bottom length; LB = B / 2 + ePyA = **920** mm

Library item: Calculate pad lengths in y

Calculate ultimate moments in y direction

Ultimate positive moment in y direction; My = fuT  LT2 / 2 + Cy  LT3 / 6 - Fu  LT2 / (2  B) = **0.000** kNm

Position of maximum negative moment; Lz = **920** mm

Ultimate negative moment in y direction; Myneg = fuT  LT2 / 2 + Cy  LT3 / 6 - Fu  LT2 / (2  B)

Myneg = **0.000** kNm

Library item: Multiple moments in y direction

Material details

Characteristic strength of concrete; fcu = **40** N/mm2

Characteristic strength of reinforcement; fy = **500** N/mm2

Characteristic strength of shear reinforcement; fyv = **500** N/mm2

Nominal cover to reinforcement; cnom = **50** mm

Library item: Material details

Moment design in x direction

Diameter of tension reinforcement; xB = **25** mm

Depth of tension reinforcement; dx = h - cnom - xB / 2 = **1738** mm

Design formula for rectangular beams (cl 3.4.4.4)

Kx = Mx / (B × dx2 × fcu) = **0.000**

Kx’ = 0.156

Kx < Kx' compression reinforcement is not required

Lever arm; zx = dx × min([0.5 + √(0.25 - Kx / 0.9)], 0.95) = **1651** mm

Area of tension reinforcement required; As\_x\_req = Mx / (0.87 × fy × zx) = **0** mm2

Minimum area of tension reinforcement; As\_x\_min = 0.0013  B  h = **4212** mm2

Tension reinforcement provided; ×××**12 No. 25 dia. bars bottom (150 centres)**

Area of tension reinforcement provided; As\_xB\_prov = NxB ×  × xB2 / 4 = **5890** mm2

PASS - Tension reinforcement provided exceeds tension reinforcement required

Library item - Output tension design in x

Negative moment design in x direction

Diameter of tension reinforcement; xT = **12** mm

Depth of tension reinforcement; dx = h - cnom - xT / 2 = **1744** mm

Design formula for rectangular beams (cl 3.4.4.4)

Kx = -Mxneg / (B × dx2 × fcu) = **0.000**

Kx’ = 0.156

Kx < Kx' compression reinforcement is not required

Lever arm; zx = dx × min([0.5 + √(0.25 - Kx / 0.9)], 0.95) = **1657** mm

Area of tension reinforcement required; As\_x\_req = -Mxneg / (0.87 × fy × zx) = **0** mm2

Minimum area of tension reinforcement; As\_x\_min = 0.0013  B  h = **4212** mm2

Tension reinforcement provided; ×××**0 No. 12 dia. bars top**

Area of tension reinforcement provided; As\_xT\_prov = NxT ×  × xT2 / 4 = **0** mm2

FAIL - Tension reinforcement provided is less than tension reinforcement required

Library item - Output neg tension design in x

Moment design in y direction

Diameter of tension reinforcement; yB = **25** mm

Depth of tension reinforcement; dy = h - cnom - xB - yB / 2 = **1713** mm

Design formula for rectangular beams (cl 3.4.4.4)

Ky = My / (L × dy2 × fcu) = **0.000**

Ky’ = 0.156

Ky < Ky' compression reinforcement is not required

Lever arm; zy = dy × min([0.5 + √(0.25 - Ky / 0.9)], 0.95) = **1627** mm

Area of tension reinforcement required; As\_y\_req = My / (0.87 × fy × zy) = **0** mm2

Minimum area of tension reinforcement; As\_y\_min = 0.0013  L  h = **4212** mm2

Tension reinforcement provided; ×××**12 No. 25 dia. bars bottom (150 centres)**

Area of tension reinforcement provided; As\_yB\_prov = NyB ×  × yB2 / 4 = **5890** mm2

PASS - Tension reinforcement provided exceeds tension reinforcement required

Library item - Output tension design in y

Negative moment design in y direction

Diameter of tension reinforcement; yT = **12** mm

Depth of tension reinforcement; dy = h - cnom - xT - yT / 2 = **1732** mm

Design formula for rectangular beams (cl 3.4.4.4)

Ky = -Myneg / (L × dy2 × fcu) = **0.000**

Ky’ = 0.156

Ky < Ky' compression reinforcement is not required

Lever arm; zy = dy × min([0.5 + √(0.25 - Ky / 0.9)], 0.95) = **1645** mm

Area of tension reinforcement required; As\_y\_req = -Myneg / (0.87 × fy × zy) = **0** mm2

Minimum area of tension reinforcement; As\_y\_min = 0.0013  L  h = **4212** mm2

Tension reinforcement provided; ×××**0 No. 12 dia. bars top**

Area of tension reinforcement provided; As\_yT\_prov = NyT ×  × yT2 / 4 = **0** mm2

FAIL - Tension reinforcement provided is less than tension reinforcement required

Library item - Output neg tension design in y

Calculate ultimate punching shear force at face of column

Ultimate pressure for punching shear; qpuA = q1u+[(L/2+ePxA-lA/2)+(lA)/2]Cx/B-[(B/2+ePyA-bA/2)+(bA)/2]Cy/L = **81.340** kN/m2

Library item: Ultimate punching shear pressure

Average effective depth of reinforcement; d = (dx + dy) / 2 = **1738** mm

Area loaded for punching shear at column; ApA = (lA)(bA) = **0.193** m2

Length of punching shear perimeter; upA = 2(lA)+2(bA) = **1800** mm

Ultimate shear force at shear perimeter; VpuA = PuA + (Fu / A - qpuA) × ApA = **0.000** kN

Effective shear force at shear perimeter; VpuAeff = VpuA = **0.000** kN

Library item: Output punching shear force

Punching shear stresses at face of column (cl 3.7.7.2)

Design shear stress; vpuA = VpuAeff / (upA × d) = **0.000** N/mm2

Allowable design shear stress; vmax = min(0.8N/mm2 × √(fcu / 1 N/mm2), 5 N/mm2) = **5.000** N/mm2

PASS - Design shear stress is less than allowable design shear stress

Library item: Output punching shear face



Library item : Show pad reinforcement sketch

;

BS RC wall design



RC Wall Design (BS8110);Wall Design to cl 3.9.3

TEDDS calculation version 1.0.04

Wall definition

; Wall thickness; h = **300** mm

; Cover to tension reinforcement; cw = **60** mm

; Trial bar diameter; Dtry = **25** mm

Depth to tension steel

h' = h - cw - Dtry/2 = **227** mm

Materials

; Characteristic strength of reinforcement; fy = **500** N/mm2

; Characteristic strength of concrete; fcu = **35** N/mm2

Library item - Calcs - wall definition

UnBraced Wall Design to cl 3.9.3 (Simply supported construction)

**Stocky check for unbraced walls**

; Wall clear height; lo = **3000** mm

;; Effective height factor for simply supported unbraced walls (assessed for a plain wall)

 = **1.00**

; le =   lo = **3.000** m; le/h = **10.00**

The unbraced wall is stocky

**Unbraced wall slenderness check**

Effective wall height; le = **3000** mm

Slenderness limit; llimit = 30  h = **9000** mm

Wall slenderness limit OK

**Define wall reinforcement**

Main reinforcement in wall

;;

Provide 20 dia bars @ 250 centres; in each face

Area of "tension" steel; Ast = Asvert = **1260** mm2/m

Area of compression steel; Asc = Ast = **1260** mm2/m

Total area of steel ; Awall = Ast + Asc = **2520.0** mm2/m

;Percentage of steel; (Ast + Asc) / h = **0.84** %

Horizontal wall steel

; Wall thickness; h = **300** mm

;Area of vertical steel provided; Awall = **2520** mm2/m

Percentage of vertical steel; pvwall = Awall / h = **0.84** %

;Minimum diameter of horizontal steel; Dmin = max(Dvert/4 , 6 mm) = **6** mm

Minimum area of horizontal steel

; AHmin = If(fy>=(460 N/mm2),if(pvwall>2 %,0.13 %,0.25%),if(pvwall>2 %,0.24 %, 0.30 %))  h/2

AHmin =**375** mm2/m

No containment links required

Define horizontal wall steel in one face;

Provide 8 dia bars @ 125 centres; in each face

Library item - Calcs - horizontal wall bars

**Stocky wall (simple construction) - transverse bending and axial load**

**Design ultimate loading**

; Design ultimate axial load per m of wall; nw = **100** kN/m

Design ultimate transverse moment per m of wall; mi = ;**100.0**; kNm/m

**Minimum design moments**

; mmin = min(0.05  h, 20 mm)  nw = **1.5** kNm/m

**Design moments**

mdesign = if(and(\_rcwall\_Moment==1,\_rcwall\_Braced=="Unbraced",\_rcwall\_Slender==1),max(abs(getvar("mi",0 kNm/m)), mmin), max(abs(getvar("mw",0 kNm/m)), mmin )) = **100.0** kNm/m

mdesign = max(abs(mi), mmin) = ;**100.0**; kNm/m

Library item – Calcs – stocky wall simple n+m

Check of design forces - symmetrically reinforced wall section

Notes

The following calculations determine the bending capacity of the wall given that an axial load nw is present. They result in the same solution as obtained from the design charts in BS8110:Pt 3 which assume a symmetrically reinforced section.

The calculations assume a simplified concrete stress block of 0.9X deep and have a lower limit of axial load applied such that the compression steel never goes into tension.

The calculations look at 5 potential solutions with the concrete crushing and reinforcement yielding and determine which one is applicable.

Note in the following:-

; h is the wall thickness

; h' is the depth from the more highly compressed face to the "tension" steel.

Material limits

Yield strain for steel

;; y = fy / (ms ES8110) = **0.0022**

"Yield" strain for concrete

;; 0 = 2.4  10-4   (fcu/((1 N/mm2)  mc)) = **0.0012**

slimit = fy / (ms  ES8110) = **0.0022**

Determine the mode of failure: Assuming simplified stress block

Case 1 - Assume tension failure in steel

Solve for x in equation nw equals 0.45fcu 0.9 x + Asc fsc + Astfst assuming fst is fym

A1 = 0.45  fcu  0.9 = **14175.0** N/mm/m

;;; B1 = ES8110  0.0035  Asc - fy / ms  Ast - nw = **234.2** kN/m

C1 = ES8110  0.0035  Asc  (h'-h) = **-63.9** kNm/m

xtry1' = [-1  B1 +  (B12 - 4  A1  C1)] / (2  A1) = **59.4** mm

limit x to be equal to h - h' as a minumum

xtry1 = max ( h - h' , xtry1' ) = **72.5** mm

Resulting steel strains

sttry1 = 0.0035  (h' - xtry1) / xtry1 = **0.0075**

sctry1 = 0.0035  (xtry1 - (h - h')) / xtry1 = **0.0000**

Chk1 = if( sctry1>slimit ,"Not OK",if( sttry1<slimit ,"Not OK","OK")) = **"OK"**

Case 2 - Assume both tension and compression failure in steel

Solve for x in equation nw equals 0.45 fcu 0.9x + Asc fsc + Astfst assuming fst and fsc are fym

xtry2 = [ nw + fy / ms  Ast - fy / ms  Asc ] / (0.45  fcu  0.9) = **7.1** mm

Resulting steel strains

sttry2 = 0.0035  (h' - xtry2) / xtry2 = **0.1094**

sctry2 = 0.0035  (xtry2 - (h - h')) / xtry2 = **-0.0325**

Chk2 = if( sctry2<slimit ,"Not OK", if( sttry2<slimit ,"Not OK","OK")) = **"Not OK"**

Case 3 - Assume Compression steel failure (0.9x<h)

Solve for x in equation nw equals 0.45 fcu 0.9x + Asc fsc + Astfst assuming fst < fym and fsc is fym

A3 = 0.45  fcu  0.9 = **14175.0** N/mm/m

B3 = fy / ms  Asc + 0.0035  ES8110  Ast - nw = **1329.8** kN/m

C3 = -1  ES8110  0.0035  h'  Ast = **-200.7** kNm/m

xtry3 = [-1  B3 +  (B32 - 4  A3  C3)] / (2  A3) = **81.0** mm

Resulting steel strains

sttry3 = 0.0035  (h' - xtry3)/ xtry3 = **0.0063**

sctry3 = 0.0035  (xtry3 - (h - h')) / xtry3 = **0.0004**

Chk3 = if(xtry3>h/0.9,"Not OK",if( sttry3>slimit ,"Not OK",if( sctry3<slimit ,"Not OK","OK"))) = **"Not OK"**

Case 3a - Assume no tension or compression steel failure (0.9x<h)

Solve for x in equation nw equals 0.45 fcu 0.9x + Asc fsc - Astfst assuming fst < fym and fsc < fym

A3a = 0.45  fcu  0.9 = **14175.0** N/mm/m

B3a = 0.0035  ES8110  Asc + 0.0035  ES8110  Ast - nw = **1664.0** kN/m

C3a = -1  ES8110  0.0035  (Asc×(h - h’) + h'  Ast) = **-264.6** kNm/m

xtry3a = [-1  B3a +  (B3a2 - 4  A3a  C3a)] / (2  A3a) = **90.0** mm

Resulting steel strains

sttry3a = 0.0035  (h' - xtry3a)/ xtry3a = **0.0053**

sctry3a = 0.0035  (xtry3a - (h - h')) / xtry3a = **0.0007**

Chk3a = if(xtry3a>h/0.9,"Not OK",if( sttry3a>slimit ,"Not OK",if( sctry3a>=slimit ,"Not OK","OK"))) = **"Not OK"**

Case 4 - Assume Compression Steel failure (0.9x>h)

Solve for x in equation nw equals 0.45 fcu h + Asc fsc + Astfst assuming fst < fym and fsc is fym

xtry4 = 0.0035  h'  ES8110  Ast / ( h0.45fcu + Ascfy/ms + 0.0035ES8110Ast - nw) = **33.1** mm

Resulting steel strains

sttry4 = 0.0035  (h' - xtry4)/ xtry4 = **0.0205**

sctry4 = 0.0035  (xtry4 - (h - h')) / xtry4 = **-0.0042**

Chk4 = if(xtry4<h/0.9 ,"Not OK",if( sttry4>slimit ,"Not OK",if( sctry4<slimit ,"Not OK","OK"))) = **"Not OK"**

Case 5 - Assume All Steel Compression Steel failure

Limiting case occurs when

xtry5 = 0.0035  h' / ( 0.0035 - slimit ) = **600.5** mm

Resulting steel strains

sttry5 = 0.0035  (h' - xtry5) / xtry5 = **-0.0022**

sctry5 = 0.0035  (xtry5 - (h - h')) / xtry5 = **0.0031**

Determine true solution

Tension steel yields

Use the correct value of fst , fsc and x to determine mR

xcalc = if(Chk1 == "OK",xtry1 ,if(Chk2 == "OK", xtry2 ,if(Chk3 == "OK", xtry3 ,if(Chk3a == “OK”, xtry3a ,if(Chk4 == "OK",xtry4 , xtry5)))))

xcalc = **72.5** mm

st = if(Chk1 == "OK",sttry1 ,if(Chk2 == "OK", sttry2 ,if(Chk3 == "OK", sttry3 ,if(Chk3a == “OK”, sttry3a,if(Chk4 == "OK",sttry4 ,sttry5 )))))

st = **0.0075**

sc = if(Chk1 == "OK",sctry1 ,if(Chk2 == "OK", sctry2 ,if(Chk3 == "OK", sctry3 ,if(Chk3a == “OK”, sctry3a,if(Chk4 == "OK",sctry4 ,sctry5 )))))

sc = **0.0000**

Resulting steel stresses

fst = min(ES8110  st , fy / ms) = **434.8** N/mm2

fsc = min(abs(ES8110  sc ), fy / ms) = **0.0** N/mm2

**For NA depth xcalc < 0.9h**

nR1 = 0.45  fcu 0.9  xcalc + Asc  fsc - Ast  fst = **479.9** kN/m

Substitute the relevant values in equation for m (about centre of section)

mR1 = 0.45fcu0.9  xcalc (h/2-(0.9xcalc)/2) + Ascfsc(h'-h/2) + Astfst(h'-h/2) = **163.1** kNm/m

**For NA depth xcalc > 0.9h**

nR2 = 0.45fcuh + Asc  fsc - Ast  fst = **4177.2** kN/m

Substitute the relevant values in equation for m (about centre of section)

mR2 = Ascfsc(h'-h/2) + Astfst(h'-h/2) = **42.5** kNm/m

**Determine correct moment of resistance**

nR = if(xcalc<h/0.9, nR1 , nR2 ) = **479.9** kN/m

mR = if(xcalc<h/0.9, mR1 , mR2 ) = **163.1** kNm/m

Applied axial load

nw = **100.0** kN/m

Check for moment

; mdesign = **100.0** kNm/m

Moment check satisfied

;The wall vertical reinforcement defined in each face is H20 dia bars @ 250 centres

Library item - Calcs - wall design check

Check min and max areas of steel

; Overall thickness of wall; h = **300** mm

**Vertical steel**

Total area of concrete per m run of wall; Ac = h = **300000** mm2/m

Ast\_min = 0.4%  Ac = **1200** mm2/m

Ast\_max = 4 %  Ac = **12000** mm2/m

;Total vertical steel in wall; Awall = **2520** mm2/m

Area of vertical steel in wall provided OK

**Horizontal steel**

Percentage of vertical steel; pvwall = Awall / h = **0.84** %

;Diameter of horizontal steel; Dhor = **8** mm

;Minimum diameter of horizontal steel; Dmin = max(Dvert/4,6 mm) = **6** mm

Diameter of horizontal steel in wall OK

;Area of horizontal steel in one face; Ashor = **402** mm2/m

Minimum area of horizontal steel

; AHmin = If(fy>=(460 N/mm2),if(pvwall>2 %,0.13 %,0.25%),if(pvwall>2 %,0.24,0.30 %))  h/2

AHmin =**375** mm2/m

Area of horizontal steel in wall provided OK

Library item - Calcs - wall max/min steel check

Shear Resistance of Concrete Walls - (cl 3.8.4.6)

; Wall thickness; h = **300** mm

; Effective depth to steel; h' = **227** mm

Area of concrete; Aconc = h = **300000** mm2/m

; Design ultimate shear force through thickness per m of wall; vw = **100** kN/m

; Characteristic strength of concrete; fcu = **35** N/mm2

**Is a check required? (3.8.4.6)**

; Axial load per m of wall; nw = **100.0** kN/m

; Major axis moment per m of wall; mw = **100.0** kNm/m

e = mw / nw = **1000.0** mm

elimit = 0.6  h = **180.0** mm

Actual shear stress; vx = vw / h' = **0.4** N/mm2

Allowable stress; vallowable = min ((0.8 N1/2/mm)  (fcu ), 5 N/mm2 ) = **4.733** N/mm2

Shear check required

**Design shear stress to clause 3.4.5.12**

; fcu\_ratio = if (fcu > 40 N/mm2 , 40/25 , fcu/(25 N/mm2)) = **1.400**

Design concrete shear stress

;; vc = 0.79 N/mm2   min(3,100  Ast / h')1/3  max(1,(400 mm) / h')1/4 / 1.25 \* fcu\_ratio1/3

; vc = **0.669** N/mm2

;;; vc' = vc + 0.6  nw / h  min( abs(vw)  h / mw, 1.0) = **0.7** N/mm2

;vallowable = min ((0.8 N1/2/mm)  (fcu ), vc' , 5 N/mm2 ) = **0.729** N/mm2

Actual shear stress

vx = **0.4** N/mm2

Shear reinforcement not necessarily required in wall

Shear stress - OK

Check of nominal cover - (BS8110:Pt 1, Table 3.4)

; Wall thickness; h = **300** mm

; Depth to tension steel from compression face; h' = **227** mm

; Diameter of vertical reinforcement; Dvert = **20** mm

; Diameter of links; Ldia = **8** mm

Cover to tension reinforcement

cten = h - h' - Dvert / 2 = **62.5** mm

Nominal cover to links steel

cnom = cten - Ldia = **54.5** mm

Permissable minimum nominal cover to all reinforcement (Table 3.4)

; cmin = **35** mm

Cover OK

Serviceability Limit State - Cracking in walls

**(BS8110:Pt 2, Cl. 3.8 & BS8007 Cl 2.6 & Appendix B)**

**Design serviceability loading**

For a conservative assessment of crack widths, the axial compression and the compression reinforcement in the wall will be ignored.

; Serviceability transverse moment per m of wall; mSLS = **100** kNm/m

; Wall thickness; h = **300** mm

; Depth to steel; h' = **227** mm

; Characteristic strength of concrete; fcu = **35** N/mm2

; Characteristic strength of reinforcement; fy = **500** N/mm2

BS8110:Pt 1:Table 3.1

; Diameter of wall vertical reinforcement; Dvert = **20** mm

; Spacing of vertical reinforcement bars; svert = **250** mm

Area of vertical reinforcement in one face; Ast =   Dvert2 /4 / svert = **1257** mm2/m

Effective depth to tension reinforcement

h' = **227.5** mm

Cover to tension reinforcement

cten = h - h' - Dvert/2 = **63** mm

Nominal cover to tension reinforcement

cnom = cten = **62.5** mm

Tension bar centres

barcrs = svert = **250.0** mm

Modular Ratio

Modulus of elasticity for reinforcement; Es = 200 kN/mm2

BS8110:Pt 1:Cl 2.5.4

Modulus of elasticity for concrete (half the instanteneous)

Ec = ((20 kN/mm2) + 200fcu) / 2 = **14** kN/mm2

BS8110:Pt 2:Equation 17

Modular ratio; m = Es / Ec = **14.815**

Neutral Axis position

For equilibrium; Fst equates Fc

Therefore: m  Ast  [ fc(h'-x)/x ] equates to 0.5  fc  x

Solving for x gives the position of the neutral axis in the section:-

x = h'  [ -1EsAst/(Ech') + ( EsAst/(Ech')  (2+EsAst/(Ech')))] = **75.3** mm

Depth of concrete in compression

x = **75.3** mm

Concrete and Steel stresses

The serviceability limit state moment per m of wall; mSLS = **100** kNm/m

Taking moments about the centreline of the reinforcement:-

Moment of resistance of concrete is 0.5  fc  x  (h' - x/3)

Solving for concrete stress fc gives;

fc = 2  mSLS / ( x  (h' - x/3)) = **13.13** N/mm2

Allowable stress; 0.45  fcu = **15.75** N/mm2

Concrete stress OK

Taking moments about the centre of action of the concrete force:-

Moment of resistance of steel is fst  As  (h' - x/3)

Solving for steel stress fst gives;

fst = mSLS / ( Ast  (h' - x/3)) = **393.16** N/mm2

Concrete and Steel strains

Strain in the reinforcement

s = fst / Es = **1.96610-3**

Allowable steel strain; 0.8  fy / Es = **2.00010-3**

Steel strain OK

BS8007:App B.4

Strain in the concrete at the level at which crack width is required

Level of crack; a' = h = **300** mm

1 = s  (a' - x)/(h' - x) = **2.90210-3**

Strain in the concrete at the level at which crack width is required adjusted for stiffening of the concrete tension zone

; Allowable crack width; CrackAllowable = **1.0** mm

BS8007:Cl 2.2.3.3

Factor for stiffening based on limiting crack width

factor = if(CrackAllowable == (0.2 mm), (1.0 N/mm2), (1.5 N/mm2)) = **2** N/mm2

m = min( 1 ,max(0, 1 - [ factor  (h - x)  (a' - x) / (3Es Ast (h' - x))])) = **2.24210-3**

BS8007:Cl 2.2.3.3

Distance from tension bar to crack in tension face between tension bars

acr =  ( (barcrs/2)2 + (cnom + Dvert/2)2) - Dvert/2 = **134.5** mm

Design crack width

Crackdesign = 3  acr  m /(1 + 2(acr - cten)/(h - x)) = **0.551** mm

BS8007:App B.3

Max allowable crack width

CrackAllowable = **1.00** mm

BS8007:Cl 2.2.3.3

Design Crack width OK

;

;

;

BS RC stair design

RC stair design (BS8110-1:1997)

TEDDS calculation version 1.0.05

Library item - Calculation title



Library item : Show stair section sketch

Stair geometry

Number of steps; Nsteps = **10**

Waist depth for stair flight; hspan = **250** mm

Going of each step; Going = **250** mm

Rise of each step; Rise = **150** mm

Angle of stairs; Rake = atan(Rise / Going) = **30.96** deg

Library item - Stair geometry

Upper landing geometry

Support condition; **Simply supported**

Landing length; Lupper = **1000** mm

Depth of landing; hupper = **200** mm

Library item - Landing geometry

Lower landing geometry

Support condition; **Simply supported**

Landing length; Llower = **1000** mm

Depth of landing; hlower = **200** mm

Library item - Landing geometry

Material details

Characteristic strength of concrete; fcu = **40** N/mm2

Characteristic strength of reinforcement; fy = **500** N/mm2

Nominal cover to reinforcement; cnom = **25** mm

Density of concrete; conc = **24.5** kN/m3

Library item - Material details

Partial safety factors

Partial safety factor for imposed loading; fq = **1.60**

Partial safety factor for dead loading; fg = **1.40**

Library item - Partial safety factors

Loading details

Characteristic imposed loading; qk = **3.000** kN/m2

Characteristic loading from finishes; gk\_fin = **1.200** kN/m2

Average stair self weight; gk\_swt = (hspan / Cos(Rake) + Rise / 2) × conc = **8.980** kN/m2

Design load; F = (gk\_swt + gk\_fin) × fg + qk × fq = **19.053** kN/m2

Library item - Loading details

Mid span design

Library item - Design titles

Midspan moment per metre width; Mspan = 0.125  F  L2 = **43.017** kNm/m

Diameter of tension reinforcement; span = **12** mm

Depth of reinforcement; dspan = hspan - cnom - span / 2 = **219** mm

Design formula for rectangular beams (cl 3.4.4.4)

Moment redistribution ratio; b = **1.00**

Kspan = Mspan / (dspan2 × fcu) = **0.022**

K’span = **0.156**

Kspan < K'span compression reinforcement is not required

Library item - Mid span K values

Lever arm; zspan = dspan × min([0.5 + √(0.25 - Kspan / 0.9)], 0.95) = **208** mm

Area of tension reinforcement required; As\_span\_req = Mspan / (0.87 × fy × zspan) = **475** mm2/m

Minimum area of tension reinforcement; As\_span\_min = 0.13  hspan / 100 = **325** mm2/m

Tension reinforcement provided; **12 dia.bars @ 200 centres**

Area of tension reinforcement provided; As\_span\_prov = **565** mm2/m

PASS - Tension reinforcement provided exceeds tension reinforcement required

Library item - Mid span design

Basic span/effective depth ratio (cl 3.4.6.3)

From BS8110 : Part 1 : 1997 – Table 3.9

Basic span/effective depth ratio; ratiobasic = **20.0**

Library item – Basic span/depth check

Modification of span/effective depth ratio for tension reinforcement (cl 3.4.6.5)

From BS8110 : Part 1 : 1997 – Table 3.10

Design service stress; fs = 2 × fy × As\_span\_req / (3 × As\_span\_prov × b) = **280.183** N/mm2

Modification factor for tension reinforcement; factortens = 0.55 + (477 N/mm2- fs)/(120 × (0.9 N/mm2+ (Mspan / dspan2)))

factortens = **1.463**

Check span/effective depth ratio (cl 3.4.6.1)

Allowable span/effective depth ratio; ratioadm = ratiobasic × factortens = **29.255**

Actual span/effective depth ratio; ratioact = L / dspan = **19.406**

PASS - Span/effective depth ratio is adequate

Library item – Span/depth check

Upper landing support design

Library item - Design titles

Diameter of tension reinforcement; upper = **12** mm

Depth of reinforcement; dupper = hupper - cnom - upper / 2 = **169** mm

Area of tension reinforcement required; As\_upper\_req = 0.4 × As\_span\_req = **190** mm2/m

Minimum area of tension reinforcement; As\_upper\_min = **260** mm2/m

Tension reinforcement provided; **12 dia.bars @ 200 centres**

Area of tension reinforcement provided; As\_upper\_prov = **565** mm2/m

PASS - Tension reinforcement provided exceeds tension reinforcement required

Library item - Upper landing simple design

Shear stress in beam (cl 3.4.5.2)

Design shear force; Vupper = 0.500 × F × L = **40.487** kN/m

Design shear stress; vupper = Vupper / dupper = **0.240** N/mm2

Allowable design shear stress; vmax = min(0.8N/mm2 × √(fcu / 1 N/mm2), 5 N/mm2) = **5.000** N/mm2

PASS - Design shear stress does not exceed allowable shear stress

From BS 8110:Part 1:1997 - Table 3.8

vc\_upper = 0.79 N/mm2 × min(3, [100 × As\_upper\_prov / dupper]1/3) × max((400 mm / dupper)1/4, 0.67) × (min(fcu, 40 N/mm2) / 25 N/mm2)1/3 / 1.25

Design concrete shear stress; vc\_upper = **0.637** N/mm2

PASS - Design shear stress does not exceed design concrete shear stress

Library item: Check upper shear stress

Lower landing support design

Library item - Design titles

Diameter of tension reinforcement; lower = **12** mm

Depth of reinforcement; dlower = hlower - cnom - lower / 2 = **169** mm

Area of tension reinforcement required; As\_lower\_req = 0.4 × As\_span\_req = **190** mm2/m

Minimum area of tension reinforcement; As\_lower\_min = **260** mm2/m

Tension reinforcement provided; **12 dia.bars @ 200 centres**

Area of tension reinforcement provided; As\_lower\_prov = **565** mm2/m

PASS - Tension reinforcement provided exceeds tension reinforcement required

Library item - Lower landing simple design

Shear stress in beam (cl 3.4.5.2)

Design shear force; Vlower = 0.500 × F × L = **40.487** kN/m

Design shear stress; vlower = Vlower / dlower = **0.240** N/mm2

Allowable design shear stress; vmax = min(0.8N/mm2 × √(fcu / 1 N/mm2), 5 N/mm2) = **5.000** N/mm2

PASS - Design shear stress does not exceed allowable shear stress

From BS 8110:Part 1:1997 - Table 3.8

vc\_lower = 0.79 N/mm2 × min(3, [100 × As\_lower\_prov / dlower]1/3) × max((400 mm / dlower)1/4, 0.67) × (min(fcu, 40 N/mm2) / 25 N/mm2)1/3 / 1.25

Design concrete shear stress; vc\_lower = **0.637** N/mm2

PASS - Design shear stress does not exceed design concrete shear stress

Library item: Check lower shear stress

EN Floor Slab A

\_exVarsLib = “$(SysLbrDir)RC Slab-EN1992\_1-si-engb.lbr”

\_exVarsItem = ”Example variables 01”

Eval( if( GetVar(“\_exFirstCalculate”, True) == True, "EvalCalcItem(\_exVarsLib, \_exVarsItem)", "" )) = **0.000**

\_exFirstCalculate = False

RC slab design

In accordance with EN1992-1-1:2004 incorporating corrigendum January 2008 and the UK national annex

Tedds calculation version 1.0.22

Library item: Calc title

Design summary

| **Description** | **Unit** | **Provided** | **Required** | **Utilisation** | **Result** | |
| --- | --- | --- | --- | --- | --- | --- |
| **Short span** | | | | | |
| Reinf. at midspan | mm2/m | 804 | 476 | 0.592 | PASS | |
| Bar spacing at midspan | mm | 250 | 300 | 0.833 | PASS | |
| Reinf. at support | mm2/m | 804 | 476 | 0.592 | PASS | |
| Bar spacing at support | mm | 250 | 300 | 0.833 | PASS | |
| Shear at cont. supp | kN/m | 144.4 | 69.3 | 0.480 | PASS | |
| Deflection ratio |  | 14.01 | 58.96 | 0.238 | PASS | |
| **Long span** | | | | | |
| Reinf. at midspan | mm2/m | 804 | 455 | 0.566 | PASS | |
| Bar spacing at midspan | mm | 250 | 300 | 0.833 | PASS | |
| Shear at discont. supp | kN/m | 140.0 | 63.0 | 0.450 | PASS | |
| **Cover** | | | | | |
| Min cover top | mm | 35 | 21 | 0.600 | PASS | |
| Min cover bottom | mm | 35 | 21 | 0.600 | PASS | |



Library item - Show active sketch

Slab definition

Slab reference name;  **Floor Slab A**

Type of slab; **Two way spanning with restrained edges**

Overall slab depth; h = **400** mm

Shorter effective span of panel; lx = **5000** mm

Longer effective span of panel; ly = **5000** mm

Support conditions; **Two short edges discontinuous**

Top outer layer of reinforcement; **Short span direction**

Bottom outer layer of reinforcement; **Short span direction**

Loading

Characteristic permanent action; Gk = **2.0** kN/m2

Characteristic variable action; Qk = **15.0** kN/m2

Partial factor for permanent action; G = **1.35**

Partial factor for variable action; Q = **1.50**

Quasi-permanent value of variable action; 2 = **0.30**

Design ultimate load; q = G × Gk + Q × Qk = **25.2** kN/m2

Quasi-permanent load; qSLS = 1.0 × Gk + 2 × Qk = **6.5** kN/m2

Library item - Slab definition 2 way

Concrete properties

Concrete strength class; C25/30

Characteristic cylinder strength; fck = **25** N/mm2

Partial factor (Table 2.1N); C = **1.50**

Compressive strength factor (cl. 3.1.6); cc = **0.85**

Design compressive strength (cl. 3.1.6); fcd = **14.2** N/mm2

Mean axial tensile strength (Table 3.1); fctm = 0.30 N/mm2  (fck / 1 N/mm2)2/3 = **2.6** N/mm2

Maximum aggregate size; dg = **20** mm

Reinforcement properties

Characteristic yield strength; fyk = **500** N/mm2

Partial factor (Table 2.1N); S = **1.15**

Design yield strength (fig. 3.8); fyd = fyk / S = **434.8** N/mm2

Library item - Material properties simplified

Concrete cover to reinforcement

Nominal cover to outer top reinforcement; cnom\_t = **35** mm

Nominal cover to outer bottom reinforcement; cnom\_b = **35** mm

Fire resistance period to top of slab; Rtop = **60** min

Fire resistance period to bottom of slab; Rbtm = **60** min

Axia distance to top reinft (Table 5.8); afi\_t = **10** mm

Axia distance to bottom reinft (Table 5.8); afi\_b = **10** mm

Min. top cover requirement with regard to bond; cmin,b\_t = **16** mm

Min. btm cover requirement with regard to bond; cmin,b\_b = **16** mm

Reinforcement fabrication; **Subject to QA system**

Cover allowance for deviation; cdev = **5** mm

Min. required nominal cover to top reinft; cnom\_t\_min = **21.0** mm

Min. required nominal cover to bottom reinft; cnom\_b\_min = **21.0** mm

PASS - There is sufficient cover to the top reinforcement

PASS - There is sufficient cover to the bottom reinforcement

Library item - Cover 2 way T B

Reinforcement design at midspan in short span direction (cl.6.1)

Bending moment coefficient; sx\_p = **0.0340**

Design bending moment; Mx\_p = sx\_p × q × lx2 = **21.4** kNm/m

Library item - Design mt 2 way restrained

Reinforcement provided; 16 mm dia. bars at 250 mm centres

Area provided; Asx\_p = 804 mm2/m

Effective depth to tension reinforcement; dx\_p = h - cnom\_b - x\_p / 2 = **357.0** mm

K factor; K = Mx\_p / (b × dx\_p2 × fck) = **0.007**

Redistribution ratio;  = 1.0

K’ factor; K’ = 0.598   - 0.18  2 - 0.21 = **0.208**

K < K' - Compression reinforcement is not required

Library item - K factor

Lever arm; z = min(0.95 × dx\_p, dx\_p/2 × (1 + (1 - 3.53  K))) = **339.2** mm

Area of reinforcement required for bending; Asx\_p\_m = Mx\_p / (fyd × z) = **145** mm2/m

Minimum area of reinforcement required; Asx\_p\_min = max(0.26 × (fctm/fyk) × b × dx\_p, 0.0013×b×dx\_p) = **476** mm2/m

Area of reinforcement required; Asx\_p\_req = max(Asx\_p\_m, Asx\_p\_min) = **476** mm2/m

Ascx\_p\_req = 0 mm2/m

Library item - As no comp reinft

PASS - Area of reinforcement provided exceeds area required

Library item - Pass/fail bending output

Check reinforcement spacing

Reinforcement service stress; sx\_p = (fyk / S) × min((Asx\_p\_m/Asx\_p), 1.0) × qSLS / q = **20.3** N/mm2

Library item - Service stress

Maximum allowable spacing (Table 7.3N); smax\_x\_p = **300** mm

Actual bar spacing; sx\_p = **250** mm

StatusSpacing = if(smax\_x\_p>= sx\_p,”Pass”,”Fail”) = **"Pass"**

PASS - The reinforcement spacing is acceptable

Library item - Output max bar spacing

Reinforcement design at midspan in long span direction (cl.6.1)

Bending moment coefficient; sy\_p = **0.0340**

Design bending moment; My\_p = sy\_p × q × lx2 = **21.4** kNm/m

Library item - Design mt 2 way restrained

Reinforcement provided; 16 mm dia. bars at 250 mm centres

Area provided; Asy\_p = 804 mm2/m

Effective depth to tension reinforcement; dy\_p = h - cnom\_b - x\_p - y\_p / 2 = **341.0** mm

K factor; K = My\_p / (b × dy\_p2 × fck) = **0.007**

Redistribution ratio;  = 1.0

K’ factor; K’ = 0.598   - 0.18  2 - 0.21 = **0.208**

K < K' - Compression reinforcement is not required

Library item - K factor

Lever arm; z = min(0.95 × dy\_p, dy\_p/2 × (1 + (1 - 3.53  K))) = **324.0** mm

Area of reinforcement required for bending; Asy\_p\_m = My\_p / (fyd × z) = **152** mm2/m

Minimum area of reinforcement required; Asy\_p\_min = max(0.26 × (fctm/fyk) × b × dy\_p, 0.0013×b×dy\_p) = **455** mm2/m

Area of reinforcement required; Asy\_p\_req = max(Asy\_p\_m, Asy\_p\_min) = **455** mm2/m

Ascy\_p\_req = 0 mm2/m

Library item - As no comp reinft

PASS - Area of reinforcement provided exceeds area required

Library item - Pass/fail bending output

Check reinforcement spacing

Reinforcement service stress; sy\_p = (fyk / S) × min((Asy\_p\_m/Asy\_p), 1.0) × qSLS / q = **21.2** N/mm2

Library item - Service stress

Maximum allowable spacing (Table 7.3N); smax\_y\_p = **300** mm

Actual bar spacing; sy\_p = **250** mm

StatusSpacing = if(smax\_y\_p>= sy\_p,”Pass”,”Fail”) = **"Pass"**

PASS - The reinforcement spacing is acceptable

Library item - Output max bar spacing

Reinforcement design at continuous support in short span direction (cl.6.1)

Bending moment coefficient; sx\_n = **0.0460**

Design bending moment; Mx\_n = sx\_n × q × lx2 = **29.0** kNm/m

Library item - Design mt 2 way restrained

Reinforcement provided; 16 mm dia. bars at 250 mm centres

Area provided; Asx\_n = 804 mm2/m

Effective depth to tension reinforcement; dx\_n = h - cnom\_t - x\_n / 2 = **357.0** mm

K factor; K = Mx\_n / (b × dx\_n2 × fck) = **0.009**

Redistribution ratio;  = 1.0

K’ factor; K’ = 0.598   - 0.18  2 - 0.21 = **0.208**

K < K' - Compression reinforcement is not required

Library item - K factor

Lever arm; z = min(0.95 × dx\_n, dx\_n/2 × (1 + (1 - 3.53  K))) = **339.2** mm

Area of reinforcement required for bending; Asx\_n\_m = Mx\_n / (fyd × z) = **197** mm2/m

Minimum area of reinforcement required; Asx\_n\_min = max(0.26 × (fctm/fyk) × b × dx\_n, 0.0013×b×dx\_n) = **476** mm2/m

Area of reinforcement required; Asx\_n\_req = max(Asx\_n\_m, Asx\_n\_min) = **476** mm2/m

Ascx\_n\_req = 0 mm2/m

Library item - As no comp reinft

PASS - Area of reinforcement provided exceeds area required

Library item - Pass/fail bending output

Check reinforcement spacing

Reinforcement service stress; sx\_n = (fyk / S) × min((Asx\_n\_m/Asx\_n), 1.0) × qSLS / q = **27.4** N/mm2

Library item - Service stress

Maximum allowable spacing (Table 7.3N); smax\_x\_n = **300** mm

Actual bar spacing; sx\_n = **250** mm

StatusSpacing = if(smax\_x\_n>= sx\_n,”Pass”,”Fail”) = **"Pass"**

PASS - The reinforcement spacing is acceptable

Library item - Output max bar spacing

Shear capacity check at short span continuous support

Shear force; Vx\_n = 1.1  q  lx / 2 = **69.3** kN/m

Effective depth factor (cl. 6.2.2); k = min(2.0, 1 + (200 mm / dx\_n)0.5) = **1.748**

Reinforcement ratio; l = min(0.02, Asx\_n / (b × dx\_n)) = **0.0023**

Minimum shear resistance (Exp. 6.3N); VRd,c\_min = 0.035 N/mm2  k1.5  (fck / 1 N/mm2)0.5 × b × dx\_n

VRd,c\_min = **144.4** kN/m

Shear resistance constant (cl. 6.2.2); CRd,c = 0.18 N/mm2 / C = **0.12** N/mm2

Shear resistance (Exp. 6.2a);

VRd,c\_x\_n = max(VRd,c\_min, CRd,c × k × (100 × l × (fck / 1 N/mm2))0.333 × b × dx\_n) = **144.4** kN/m

Library item - Shear output

PASS - Shear capacity is adequate

Library item - Shear pass/fail output

Shear capacity check at long span discontinuous support

Vy\_d = if(\_tmp.ShearType == 1, q × lx / 2, 0.8 × q × lx / 2) = **63.0** kN/m

Shear force; ×××Vy\_d = q  lx / 2 = ;**63.0**; kN/m;

Reinforcement provided; **16 mm dia. bars at 250 mm centres**

Area provided; Asy\_d = **804** mm2/m

dy\_d = if(\_rcs.SupportReinftY==1,dy\_p, if(\_rcs.BtmLayer==”Long”, h - cnom\_b - y\_d / 2, h - cnom\_b - x\_p - y\_d / 2)) = **341.0** mm

Effective depth; dy\_d = h - cnom\_b - x\_p - y\_d / 2 = ;**341.0**; mm

Effective depth factor; k = min(2.0, 1 + (200 mm / dy\_d)0.5) = **1.766**

Reinforcement ratio; l = min(0.02, Asy\_d / (b × dy\_d)) = **0.0024**

Minimum shear resistance; VRd,c\_min = 0.035 N/mm2  k1.5  (fck / 1 N/mm2)0.5 × b × dy\_d

VRd,c\_min = **140.0** kN/m

Shear resistance constant (cl. 6.2.2); CRd,c = 0.18 N/mm2 / C = **0.12** N/mm2

Shear resistance;

VRd,c\_y\_d = max(VRd,c\_min, CRd,c × k × (100 × l × (fck/1 N/mm2))0.333 × b × dy\_d) = **140.0** kN/m

PASS - Shear capacity is adequate (0.450)

Library item - Shear discont y

Basic span-to-depth deflection ratio check (cl. 7.4.2)

Reference reinforcement ratio; 0 = (fck / 1 N/mm2)0.5 / 1000 = **0.0050**

Required tension reinforcement ratio;  = max(0.0035, Asx\_p\_req / (b × dx\_p)) = **0.0035**

Required compression reinforcement ratio; ’ = Ascx\_p\_req / (b × dx\_p) = **0.0000**

Stuctural system factor (Table 7.4N); K = **1.5**

Deflection output 1

Basic limit span-to-depth ratio (Exp. 7.16);

ratiolim\_x\_bas = K  [11 +1.5(fck/1 N/mm2)0.50/ + 3.2(fck/1 N/mm2)0.5(0/ -1)1.5] = **39.31**

Mod span-to-depth ratio limit;

ratiolim\_x = min(40  K, min(1.5, (500 N/mm2/ fyk)  (Asx\_p / Asx\_p\_m))  ratiolim\_x\_bas) = **58.96**

Actual span-to-eff. depth ratio; ratioact\_x = lx / dx\_p = **14.01**

Deflection output 2

PASS - Actual span-to-effective depth ratio is acceptable

Deflection output 3

Reinforcement summary

Midspan in short span direction; **16 mm dia. bars at 250 mm centres B1**

Midspan in long span direction; **16 mm dia. bars at 250 mm centres B2**

Library item - Reinft summary xp yp

Continuous support in short span direction; **16 mm dia. bars at 250 mm centres T1**

Library item - Reinft summary xn

Discontinuous support in long span direction; **16 mm dia. bars at 250 mm centres B2**

Library item - Reinft summary yd

Reinforcement sketch

The following sketch is indicative only. Note that additional reinforcement may be required in accordance with clauses 9.2.1.2, 9.2.1.4 and 9.2.1.5 of EN 1992-1-1:2004 to meet detailing rules.

Library item - Reinft sketch title



Library item - Show active sketch

;

EN Floor Slab B

\_exVarsLib = “$(SysLbrDir)RC Slab-EN1992\_1-si-engb.lbr”

\_exVarsItem = ”Example variables 01”

Eval( if( GetVar(“\_exFirstCalculate”, True) == True, "EvalCalcItem(\_exVarsLib, \_exVarsItem)", "" )) = **0.000**

\_exFirstCalculate = False

RC slab design

In accordance with EN1992-1-1:2004 incorporating corrigendum January 2008 and the UK national annex

Tedds calculation version 1.0.22

Library item: Calc title

Design summary

| **Description** | **Unit** | **Provided** | **Required** | **Utilisation** | **Result** | |
| --- | --- | --- | --- | --- | --- | --- |
| **Short span** | | | | | |
| Reinf. at midspan | mm2/m | 804 | 476 | 0.592 | PASS | |
| Bar spacing at midspan | mm | 250 | 300 | 0.833 | PASS | |
| Reinf. at support | mm2/m | 804 | 476 | 0.592 | PASS | |
| Bar spacing at support | mm | 250 | 300 | 0.833 | PASS | |
| Shear at cont. supp | kN/m | 144.4 | 69.3 | 0.480 | PASS | |
| Deflection ratio |  | 14.01 | 58.96 | 0.238 | PASS | |
| **Long span** | | | | | |
| Reinf. at midspan | mm2/m | 804 | 455 | 0.566 | PASS | |
| Bar spacing at midspan | mm | 250 | 300 | 0.833 | PASS | |
| Reinf. at support | mm2/m | 804 | 455 | 0.566 | PASS | |
| Bar spacing at support | mm | 250 | 300 | 0.833 | PASS | |
| Shear at cont. supp | kN/m | 140.0 | 69.3 | 0.495 | PASS | |
| Shear at discont. supp | kN/m | 140.0 | 50.4 | 0.360 | PASS | |
| **Cover** | | | | | |
| Min cover top | mm | 35 | 26 | 0.743 | PASS | |
| Min cover bottom | mm | 35 | 26 | 0.743 | PASS | |



Library item - Show active sketch

Slab definition

Slab reference name;  **Floor Slab B**

Type of slab; **Two way spanning with restrained edges**

Overall slab depth; h = **400** mm

Shorter effective span of panel; lx = **5000** mm

Longer effective span of panel; ly = **5000** mm

Support conditions; **One short edge discontinuous**

Top outer layer of reinforcement; **Short span direction**

Bottom outer layer of reinforcement; **Short span direction**

Loading

Characteristic permanent action; Gk = **2.0** kN/m2

Characteristic variable action; Qk = **15.0** kN/m2

Partial factor for permanent action; G = **1.35**

Partial factor for variable action; Q = **1.50**

Quasi-permanent value of variable action; 2 = **0.30**

Design ultimate load; q = G × Gk + Q × Qk = **25.2** kN/m2

Quasi-permanent load; qSLS = 1.0 × Gk + 2 × Qk = **6.5** kN/m2

Library item - Slab definition 2 way

Concrete properties

Concrete strength class; C25/30

Characteristic cylinder strength; fck = **25** N/mm2

Partial factor (Table 2.1N); C = **1.50**

Compressive strength factor (cl. 3.1.6); cc = **0.85**

Design compressive strength (cl. 3.1.6); fcd = **14.2** N/mm2

Mean axial tensile strength (Table 3.1); fctm = 0.30 N/mm2  (fck / 1 N/mm2)2/3 = **2.6** N/mm2

Maximum aggregate size; dg = **20** mm

Reinforcement properties

Characteristic yield strength; fyk = **500** N/mm2

Partial factor (Table 2.1N); S = **1.15**

Design yield strength (fig. 3.8); fyd = fyk / S = **434.8** N/mm2

Library item - Material properties simplified

Concrete cover to reinforcement

Nominal cover to outer top reinforcement; cnom\_t = **35** mm

Nominal cover to outer bottom reinforcement; cnom\_b = **35** mm

Fire resistance period to top of slab; Rtop = **60** min

Fire resistance period to bottom of slab; Rbtm = **60** min

Axia distance to top reinft (Table 5.8); afi\_t = **10** mm

Axia distance to bottom reinft (Table 5.8); afi\_b = **10** mm

Min. top cover requirement with regard to bond; cmin,b\_t = **16** mm

Min. btm cover requirement with regard to bond; cmin,b\_b = **16** mm

Reinforcement fabrication; **Not subject to QA system**

Cover allowance for deviation; cdev = **10** mm

Min. required nominal cover to top reinft; cnom\_t\_min = **26.0** mm

Min. required nominal cover to bottom reinft; cnom\_b\_min = **26.0** mm

PASS - There is sufficient cover to the top reinforcement

PASS - There is sufficient cover to the bottom reinforcement

Library item - Cover 2 way T B

Reinforcement design at midspan in short span direction (cl.6.1)

Bending moment coefficient; sx\_p = **0.0290**

Design bending moment; Mx\_p = sx\_p × q × lx2 = **18.3** kNm/m

Library item - Design mt 2 way restrained

Reinforcement provided; 16 mm dia. bars at 250 mm centres

Area provided; Asx\_p = 804 mm2/m

Effective depth to tension reinforcement; dx\_p = h - cnom\_b - x\_p / 2 = **357.0** mm

K factor; K = Mx\_p / (b × dx\_p2 × fck) = **0.006**

Redistribution ratio;  = 1.0

K’ factor; K’ = 0.598   - 0.18  2 - 0.21 = **0.208**

K < K' - Compression reinforcement is not required

Library item - K factor

Lever arm; z = min(0.95 × dx\_p, dx\_p/2 × (1 + (1 - 3.53  K))) = **339.2** mm

Area of reinforcement required for bending; Asx\_p\_m = Mx\_p / (fyd × z) = **124** mm2/m

Minimum area of reinforcement required; Asx\_p\_min = max(0.26 × (fctm/fyk) × b × dx\_p, 0.0013×b×dx\_p) = **476** mm2/m

Area of reinforcement required; Asx\_p\_req = max(Asx\_p\_m, Asx\_p\_min) = **476** mm2/m

Ascx\_p\_req = 0 mm2/m

Library item - As no comp reinft

PASS - Area of reinforcement provided exceeds area required

Library item - Pass/fail bending output

Check reinforcement spacing

Reinforcement service stress; sx\_p = (fyk / S) × min((Asx\_p\_m/Asx\_p), 1.0) × qSLS / q = **17.3** N/mm2

Library item - Service stress

Maximum allowable spacing (Table 7.3N); smax\_x\_p = **300** mm

Actual bar spacing; sx\_p = **250** mm

StatusSpacing = if(smax\_x\_p>= sx\_p,”Pass”,”Fail”) = **"Pass"**

PASS - The reinforcement spacing is acceptable

Library item - Output max bar spacing

Reinforcement design at midspan in long span direction (cl.6.1)

Bending moment coefficient; sy\_p = **0.0280**

Design bending moment; My\_p = sy\_p × q × lx2 = **17.6** kNm/m

Library item - Design mt 2 way restrained

Reinforcement provided; 16 mm dia. bars at 250 mm centres

Area provided; Asy\_p = 804 mm2/m

Effective depth to tension reinforcement; dy\_p = h - cnom\_b - x\_p - y\_p / 2 = **341.0** mm

K factor; K = My\_p / (b × dy\_p2 × fck) = **0.006**

Redistribution ratio;  = 1.0

K’ factor; K’ = 0.598   - 0.18  2 - 0.21 = **0.208**

K < K' - Compression reinforcement is not required

Library item - K factor

Lever arm; z = min(0.95 × dy\_p, dy\_p/2 × (1 + (1 - 3.53  K))) = **324.0** mm

Area of reinforcement required for bending; Asy\_p\_m = My\_p / (fyd × z) = **125** mm2/m

Minimum area of reinforcement required; Asy\_p\_min = max(0.26 × (fctm/fyk) × b × dy\_p, 0.0013×b×dy\_p) = **455** mm2/m

Area of reinforcement required; Asy\_p\_req = max(Asy\_p\_m, Asy\_p\_min) = **455** mm2/m

Ascy\_p\_req = 0 mm2/m

Library item - As no comp reinft

PASS - Area of reinforcement provided exceeds area required

Library item - Pass/fail bending output

Check reinforcement spacing

Reinforcement service stress; sy\_p = (fyk / S) × min((Asy\_p\_m/Asy\_p), 1.0) × qSLS / q = **17.5** N/mm2

Library item - Service stress

Maximum allowable spacing (Table 7.3N); smax\_y\_p = **300** mm

Actual bar spacing; sy\_p = **250** mm

StatusSpacing = if(smax\_y\_p>= sy\_p,”Pass”,”Fail”) = **"Pass"**

PASS - The reinforcement spacing is acceptable

Library item - Output max bar spacing

Reinforcement design at continuous support in short span direction (cl.6.1)

Bending moment coefficient; sx\_n = **0.0390**

Design bending moment; Mx\_n = sx\_n × q × lx2 = **24.6** kNm/m

Library item - Design mt 2 way restrained

Reinforcement provided; 16 mm dia. bars at 250 mm centres

Area provided; Asx\_n = 804 mm2/m

Effective depth to tension reinforcement; dx\_n = h - cnom\_t - x\_n / 2 = **357.0** mm

K factor; K = Mx\_n / (b × dx\_n2 × fck) = **0.008**

Redistribution ratio;  = 1.0

K’ factor; K’ = 0.598   - 0.18  2 - 0.21 = **0.208**

K < K' - Compression reinforcement is not required

Library item - K factor

Lever arm; z = min(0.95 × dx\_n, dx\_n/2 × (1 + (1 - 3.53  K))) = **339.2** mm

Area of reinforcement required for bending; Asx\_n\_m = Mx\_n / (fyd × z) = **167** mm2/m

Minimum area of reinforcement required; Asx\_n\_min = max(0.26 × (fctm/fyk) × b × dx\_n, 0.0013×b×dx\_n) = **476** mm2/m

Area of reinforcement required; Asx\_n\_req = max(Asx\_n\_m, Asx\_n\_min) = **476** mm2/m

Ascx\_n\_req = 0 mm2/m

Library item - As no comp reinft

PASS - Area of reinforcement provided exceeds area required

Library item - Pass/fail bending output

Check reinforcement spacing

Reinforcement service stress; sx\_n = (fyk / S) × min((Asx\_n\_m/Asx\_n), 1.0) × qSLS / q = **23.2** N/mm2

Library item - Service stress

Maximum allowable spacing (Table 7.3N); smax\_x\_n = **300** mm

Actual bar spacing; sx\_n = **250** mm

StatusSpacing = if(smax\_x\_n>= sx\_n,”Pass”,”Fail”) = **"Pass"**

PASS - The reinforcement spacing is acceptable

Library item - Output max bar spacing

Reinforcement design at continuous support in long span direction (cl.6.1)

Bending moment coefficient; sy\_n = **0.0370**

Design bending moment; My\_n = sy\_n × q × lx2 = **23.3** kNm/m

Library item - Design mt 2 way restrained

Reinforcement provided; 16 mm dia. bars at 250 mm centres

Area provided; Asy\_n = 804 mm2/m

Effective depth to tension reinforcement; dy\_n = h - cnom\_t - x\_n - y\_n / 2 = **341.0** mm

K factor; K = My\_n / (b × dy\_n2 × fck) = **0.008**

Redistribution ratio;  = 1.0

K’ factor; K’ = 0.598   - 0.18  2 - 0.21 = **0.208**

K < K' - Compression reinforcement is not required

Library item - K factor

Lever arm; z = min(0.95 × dy\_n, dy\_n/2 × (1 + (1 - 3.53  K))) = **324.0** mm

Area of reinforcement required for bending; Asy\_n\_m = My\_n / (fyd × z) = **165** mm2/m

Minimum area of reinforcement required; Asy\_n\_min = max(0.26 × (fctm/fyk) × b × dy\_n, 0.0013×b×dy\_n) = **455** mm2/m

Area of reinforcement required; Asy\_n\_req = max(Asy\_n\_m, Asy\_n\_min) = **455** mm2/m

Ascy\_n\_req = 0 mm2/m

Library item - As no comp reinft

PASS - Area of reinforcement provided exceeds area required

Library item - Pass/fail bending output

Check reinforcement spacing

Reinforcement service stress; sy\_n = (fyk / S) × min((Asy\_n\_m/Asy\_n), 1.0) × qSLS / q = **23.1** N/mm2

Library item - Service stress

Maximum allowable spacing (Table 7.3N); smax\_y\_n = **300** mm

Actual bar spacing; sy\_n = **250** mm

StatusSpacing = if(smax\_y\_n>= sy\_n,”Pass”,”Fail”) = **"Pass"**

PASS - The reinforcement spacing is acceptable

Library item - Output max bar spacing

Shear capacity check at short span continuous support

Shear force; Vx\_n = 1.1  q  lx / 2 = **69.3** kN/m

Effective depth factor (cl. 6.2.2); k = min(2.0, 1 + (200 mm / dx\_n)0.5) = **1.748**

Reinforcement ratio; l = min(0.02, Asx\_n / (b × dx\_n)) = **0.0023**

Minimum shear resistance (Exp. 6.3N); VRd,c\_min = 0.035 N/mm2  k1.5  (fck / 1 N/mm2)0.5 × b × dx\_n

VRd,c\_min = **144.4** kN/m

Shear resistance constant (cl. 6.2.2); CRd,c = 0.18 N/mm2 / C = **0.12** N/mm2

Shear resistance (Exp. 6.2a);

VRd,c\_x\_n = max(VRd,c\_min, CRd,c × k × (100 × l × (fck / 1 N/mm2))0.333 × b × dx\_n) = **144.4** kN/m

Library item - Shear output

PASS - Shear capacity is adequate

Library item - Shear pass/fail output

Shear capacity check at long span continuous support

Shear force; Vy\_n = 1.1  q  lx / 2 = **69.3** kN/m

Effective depth factor (cl. 6.2.2); k = min(2.0, 1 + (200 mm / dy\_n)0.5) = **1.766**

Reinforcement ratio; l = min(0.02, Asy\_n / (b × dy\_n)) = **0.0024**

Minimum shear resistance (Exp. 6.3N); VRd,c\_min = 0.035 N/mm2  k1.5  (fck / 1 N/mm2)0.5 × b × dy\_n

VRd,c\_min = **140.0** kN/m

Shear resistance constant (cl. 6.2.2); CRd,c = 0.18 N/mm2 / C = **0.12** N/mm2

Shear resistance (Exp. 6.2a);

VRd,c\_y\_n = max(VRd,c\_min, CRd,c × k × (100 × l × (fck / 1 N/mm2))0.333 × b × dy\_n) = **140.0** kN/m

Library item - Shear output

PASS - Shear capacity is adequate

Library item - Shear pass/fail output

Shear capacity check at long span discontinuous support

Vy\_d = if(\_tmp.ShearType == 1, q × lx / 2, 0.8 × q × lx / 2) = **50.4** kN/m

Shear force; ×××Vx\_d = 0.8  q  lx / 2 = ;**50.4**; kN/m;

Reinforcement provided; **16 mm dia. bars at 250 mm centres**

Area provided; Asy\_d = **804** mm2/m

dy\_d = if(\_rcs.SupportReinftY==1,dy\_p, if(\_rcs.BtmLayer==”Long”, h - cnom\_b - y\_d / 2, h - cnom\_b - x\_p - y\_d / 2)) = **341.0** mm

Effective depth; dy\_d = h - cnom\_b - x\_p - y\_d / 2 = ;**341.0**; mm

Effective depth factor; k = min(2.0, 1 + (200 mm / dy\_d)0.5) = **1.766**

Reinforcement ratio; l = min(0.02, Asy\_d / (b × dy\_d)) = **0.0024**

Minimum shear resistance; VRd,c\_min = 0.035 N/mm2  k1.5  (fck / 1 N/mm2)0.5 × b × dy\_d

VRd,c\_min = **140.0** kN/m

Shear resistance constant (cl. 6.2.2); CRd,c = 0.18 N/mm2 / C = **0.12** N/mm2

Shear resistance;

VRd,c\_y\_d = max(VRd,c\_min, CRd,c × k × (100 × l × (fck/1 N/mm2))0.333 × b × dy\_d) = **140.0** kN/m

PASS - Shear capacity is adequate (0.360)

Library item - Shear discont y

Basic span-to-depth deflection ratio check (cl. 7.4.2)

Reference reinforcement ratio; 0 = (fck / 1 N/mm2)0.5 / 1000 = **0.0050**

Required tension reinforcement ratio;  = max(0.0035, Asx\_p\_req / (b × dx\_p)) = **0.0035**

Required compression reinforcement ratio; ’ = Ascx\_p\_req / (b × dx\_p) = **0.0000**

Stuctural system factor (Table 7.4N); K = **1.5**

Deflection output 1

Basic limit span-to-depth ratio (Exp. 7.16);

ratiolim\_x\_bas = K  [11 +1.5(fck/1 N/mm2)0.50/ + 3.2(fck/1 N/mm2)0.5(0/ -1)1.5] = **39.31**

Mod span-to-depth ratio limit;

ratiolim\_x = min(40  K, min(1.5, (500 N/mm2/ fyk)  (Asx\_p / Asx\_p\_m))  ratiolim\_x\_bas) = **58.96**

Actual span-to-eff. depth ratio; ratioact\_x = lx / dx\_p = **14.01**

Deflection output 2

PASS - Actual span-to-effective depth ratio is acceptable

Deflection output 3

Reinforcement summary

Midspan in short span direction; **16 mm dia. bars at 250 mm centres B1**

Midspan in long span direction; **16 mm dia. bars at 250 mm centres B2**

Library item - Reinft summary xp yp

Continuous support in short span direction; **16 mm dia. bars at 250 mm centres T1**

Library item - Reinft summary xn

Continuous support in long span direction; **16 mm dia. bars at 250 mm centres T2**

Library item - Reinft summary yn

Discontinuous support in long span direction; **16 mm dia. bars at 250 mm centres B2**

Library item - Reinft summary yd

Reinforcement sketch

The following sketch is indicative only. Note that additional reinforcement may be required in accordance with clauses 9.2.1.2, 9.2.1.4 and 9.2.1.5 of EN 1992-1-1:2004 to meet detailing rules.

Library item - Reinft sketch title



Library item - Show active sketch

;

EN Floor Slab C

\_exVarsLib = “$(SysLbrDir)RC Slab-EN1992\_1-si-engb.lbr”

\_exVarsItem = ”Example variables 01”

Eval( if( GetVar(“\_exFirstCalculate”, True) == True, "EvalCalcItem(\_exVarsLib, \_exVarsItem)", "" )) = **0.000**

\_exFirstCalculate = False

RC slab design

In accordance with EN1992-1-1:2004 incorporating corrigendum January 2008 and the UK national annex

Tedds calculation version 1.0.22

Library item: Calc title

Design summary

| **Description** | **Unit** | **Provided** | **Required** | **Utilisation** | **Result** | |
| --- | --- | --- | --- | --- | --- | --- |
| **Short span** | | | | | |
| Reinf. at midspan | mm2/m | 804 | 476 | 0.592 | PASS | |
| Bar spacing at midspan | mm | 250 | 300 | 0.833 | PASS | |
| Reinf. at support | mm2/m | 804 | 476 | 0.592 | PASS | |
| Bar spacing at support | mm | 250 | 300 | 0.833 | PASS | |
| Shear at cont. supp | kN/m | 144.4 | 63.0 | 0.436 | PASS | |
| Deflection ratio |  | 14.01 | 58.96 | 0.238 | PASS | |
| **Long span** | | | | | |
| Reinf. at midspan | mm2/m | 804 | 455 | 0.566 | PASS | |
| Bar spacing at midspan | mm | 250 | 300 | 0.833 | PASS | |
| Reinf. at support | mm2/m | 804 | 455 | 0.566 | PASS | |
| Bar spacing at support | mm | 250 | 300 | 0.833 | PASS | |
| Shear at cont. supp | kN/m | 140.0 | 63.0 | 0.450 | PASS | |
| **Cover** | | | | | |
| Min cover top | mm | 35 | 26 | 0.743 | PASS | |
| Min cover bottom | mm | 35 | 26 | 0.743 | PASS | |



Library item - Show active sketch

Slab definition

Slab reference name;  **Floor Slab C**

Type of slab; **Two way spanning with restrained edges**

Overall slab depth; h = **400** mm

Shorter effective span of panel; lx = **5000** mm

Longer effective span of panel; ly = **5000** mm

Support conditions; **Four edges continuous (interior panel)**

Top outer layer of reinforcement; **Short span direction**

Bottom outer layer of reinforcement; **Short span direction**

Loading

Characteristic permanent action; Gk = **2.0** kN/m2

Characteristic variable action; Qk = **15.0** kN/m2

Partial factor for permanent action; G = **1.35**

Partial factor for variable action; Q = **1.50**

Quasi-permanent value of variable action; 2 = **0.30**

Design ultimate load; q = G × Gk + Q × Qk = **25.2** kN/m2

Quasi-permanent load; qSLS = 1.0 × Gk + 2 × Qk = **6.5** kN/m2

Library item - Slab definition 2 way

Concrete properties

Concrete strength class; C25/30

Characteristic cylinder strength; fck = **25** N/mm2

Partial factor (Table 2.1N); C = **1.50**

Compressive strength factor (cl. 3.1.6); cc = **0.85**

Design compressive strength (cl. 3.1.6); fcd = **14.2** N/mm2

Mean axial tensile strength (Table 3.1); fctm = 0.30 N/mm2  (fck / 1 N/mm2)2/3 = **2.6** N/mm2

Maximum aggregate size; dg = **20** mm

Reinforcement properties

Characteristic yield strength; fyk = **500** N/mm2

Partial factor (Table 2.1N); S = **1.15**

Design yield strength (fig. 3.8); fyd = fyk / S = **434.8** N/mm2

Library item - Material properties simplified

Concrete cover to reinforcement

Nominal cover to outer top reinforcement; cnom\_t = **35** mm

Nominal cover to outer bottom reinforcement; cnom\_b = **35** mm

Fire resistance period to top of slab; Rtop = **60** min

Fire resistance period to bottom of slab; Rbtm = **60** min

Axia distance to top reinft (Table 5.8); afi\_t = **10** mm

Axia distance to bottom reinft (Table 5.8); afi\_b = **10** mm

Min. top cover requirement with regard to bond; cmin,b\_t = **16** mm

Min. btm cover requirement with regard to bond; cmin,b\_b = **16** mm

Reinforcement fabrication; **Not subject to QA system**

Cover allowance for deviation; cdev = **10** mm

Min. required nominal cover to top reinft; cnom\_t\_min = **26.0** mm

Min. required nominal cover to bottom reinft; cnom\_b\_min = **26.0** mm

PASS - There is sufficient cover to the top reinforcement

PASS - There is sufficient cover to the bottom reinforcement

Library item - Cover 2 way T B

Reinforcement design at midspan in short span direction (cl.6.1)

Bending moment coefficient; sx\_p = **0.0240**

Design bending moment; Mx\_p = sx\_p × q × lx2 = **15.1** kNm/m

Library item - Design mt 2 way restrained

Reinforcement provided; 16 mm dia. bars at 250 mm centres

Area provided; Asx\_p = 804 mm2/m

Effective depth to tension reinforcement; dx\_p = h - cnom\_b - x\_p / 2 = **357.0** mm

K factor; K = Mx\_p / (b × dx\_p2 × fck) = **0.005**

Redistribution ratio;  = 1.0

K’ factor; K’ = 0.598   - 0.18  2 - 0.21 = **0.208**

K < K' - Compression reinforcement is not required

Library item - K factor

Lever arm; z = min(0.95 × dx\_p, dx\_p/2 × (1 + (1 - 3.53  K))) = **339.2** mm

Area of reinforcement required for bending; Asx\_p\_m = Mx\_p / (fyd × z) = **103** mm2/m

Minimum area of reinforcement required; Asx\_p\_min = max(0.26 × (fctm/fyk) × b × dx\_p, 0.0013×b×dx\_p) = **476** mm2/m

Area of reinforcement required; Asx\_p\_req = max(Asx\_p\_m, Asx\_p\_min) = **476** mm2/m

Ascx\_p\_req = 0 mm2/m

Library item - As no comp reinft

PASS - Area of reinforcement provided exceeds area required

Library item - Pass/fail bending output

Check reinforcement spacing

Reinforcement service stress; sx\_p = (fyk / S) × min((Asx\_p\_m/Asx\_p), 1.0) × qSLS / q = **14.3** N/mm2

Library item - Service stress

Maximum allowable spacing (Table 7.3N); smax\_x\_p = **300** mm

Actual bar spacing; sx\_p = **250** mm

StatusSpacing = if(smax\_x\_p>= sx\_p,”Pass”,”Fail”) = **"Pass"**

PASS - The reinforcement spacing is acceptable

Library item - Output max bar spacing

Reinforcement design at midspan in long span direction (cl.6.1)

Bending moment coefficient; sy\_p = **0.0240**

Design bending moment; My\_p = sy\_p × q × lx2 = **15.1** kNm/m

Library item - Design mt 2 way restrained

Reinforcement provided; 16 mm dia. bars at 250 mm centres

Area provided; Asy\_p = 804 mm2/m

Effective depth to tension reinforcement; dy\_p = h - cnom\_b - x\_p - y\_p / 2 = **341.0** mm

K factor; K = My\_p / (b × dy\_p2 × fck) = **0.005**

Redistribution ratio;  = 1.0

K’ factor; K’ = 0.598   - 0.18  2 - 0.21 = **0.208**

K < K' - Compression reinforcement is not required

Library item - K factor

Lever arm; z = min(0.95 × dy\_p, dy\_p/2 × (1 + (1 - 3.53  K))) = **324.0** mm

Area of reinforcement required for bending; Asy\_p\_m = My\_p / (fyd × z) = **107** mm2/m

Minimum area of reinforcement required; Asy\_p\_min = max(0.26 × (fctm/fyk) × b × dy\_p, 0.0013×b×dy\_p) = **455** mm2/m

Area of reinforcement required; Asy\_p\_req = max(Asy\_p\_m, Asy\_p\_min) = **455** mm2/m

Ascy\_p\_req = 0 mm2/m

Library item - As no comp reinft

PASS - Area of reinforcement provided exceeds area required

Library item - Pass/fail bending output

Check reinforcement spacing

Reinforcement service stress; sy\_p = (fyk / S) × min((Asy\_p\_m/Asy\_p), 1.0) × qSLS / q = **15.0** N/mm2

Library item - Service stress

Maximum allowable spacing (Table 7.3N); smax\_y\_p = **300** mm

Actual bar spacing; sy\_p = **250** mm

StatusSpacing = if(smax\_y\_p>= sy\_p,”Pass”,”Fail”) = **"Pass"**

PASS - The reinforcement spacing is acceptable

Library item - Output max bar spacing

Reinforcement design at continuous support in short span direction (cl.6.1)

Bending moment coefficient; sx\_n = **0.0310**

Design bending moment; Mx\_n = sx\_n × q × lx2 = **19.5** kNm/m

Library item - Design mt 2 way restrained

Reinforcement provided; 16 mm dia. bars at 250 mm centres

Area provided; Asx\_n = 804 mm2/m

Effective depth to tension reinforcement; dx\_n = h - cnom\_t - x\_n / 2 = **357.0** mm

K factor; K = Mx\_n / (b × dx\_n2 × fck) = **0.006**

Redistribution ratio;  = 1.0

K’ factor; K’ = 0.598   - 0.18  2 - 0.21 = **0.208**

K < K' - Compression reinforcement is not required

Library item - K factor

Lever arm; z = min(0.95 × dx\_n, dx\_n/2 × (1 + (1 - 3.53  K))) = **339.2** mm

Area of reinforcement required for bending; Asx\_n\_m = Mx\_n / (fyd × z) = **132** mm2/m

Minimum area of reinforcement required; Asx\_n\_min = max(0.26 × (fctm/fyk) × b × dx\_n, 0.0013×b×dx\_n) = **476** mm2/m

Area of reinforcement required; Asx\_n\_req = max(Asx\_n\_m, Asx\_n\_min) = **476** mm2/m

Ascx\_n\_req = 0 mm2/m

Library item - As no comp reinft

PASS - Area of reinforcement provided exceeds area required

Library item - Pass/fail bending output

Check reinforcement spacing

Reinforcement service stress; sx\_n = (fyk / S) × min((Asx\_n\_m/Asx\_n), 1.0) × qSLS / q = **18.5** N/mm2

Library item - Service stress

Maximum allowable spacing (Table 7.3N); smax\_x\_n = **300** mm

Actual bar spacing; sx\_n = **250** mm

StatusSpacing = if(smax\_x\_n>= sx\_n,”Pass”,”Fail”) = **"Pass"**

PASS - The reinforcement spacing is acceptable

Library item - Output max bar spacing

Reinforcement design at continuous support in long span direction (cl.6.1)

Bending moment coefficient; sy\_n = **0.0320**

Design bending moment; My\_n = sy\_n × q × lx2 = **20.2** kNm/m

Library item - Design mt 2 way restrained

Reinforcement provided; 16 mm dia. bars at 250 mm centres

Area provided; Asy\_n = 804 mm2/m

Effective depth to tension reinforcement; dy\_n = h - cnom\_t - x\_n - y\_n / 2 = **341.0** mm

K factor; K = My\_n / (b × dy\_n2 × fck) = **0.007**

Redistribution ratio;  = 1.0

K’ factor; K’ = 0.598   - 0.18  2 - 0.21 = **0.208**

K < K' - Compression reinforcement is not required

Library item - K factor

Lever arm; z = min(0.95 × dy\_n, dy\_n/2 × (1 + (1 - 3.53  K))) = **324.0** mm

Area of reinforcement required for bending; Asy\_n\_m = My\_n / (fyd × z) = **143** mm2/m

Minimum area of reinforcement required; Asy\_n\_min = max(0.26 × (fctm/fyk) × b × dy\_n, 0.0013×b×dy\_n) = **455** mm2/m

Area of reinforcement required; Asy\_n\_req = max(Asy\_n\_m, Asy\_n\_min) = **455** mm2/m

Ascy\_n\_req = 0 mm2/m

Library item - As no comp reinft

PASS - Area of reinforcement provided exceeds area required

Library item - Pass/fail bending output

Check reinforcement spacing

Reinforcement service stress; sy\_n = (fyk / S) × min((Asy\_n\_m/Asy\_n), 1.0) × qSLS / q = **20.0** N/mm2

Library item - Service stress

Maximum allowable spacing (Table 7.3N); smax\_y\_n = **300** mm

Actual bar spacing; sy\_n = **250** mm

StatusSpacing = if(smax\_y\_n>= sy\_n,”Pass”,”Fail”) = **"Pass"**

PASS - The reinforcement spacing is acceptable

Library item - Output max bar spacing

Shear capacity check at short span continuous support

Shear force; Vx\_n = q  lx / 2 = **63.0** kN/m

Effective depth factor (cl. 6.2.2); k = min(2.0, 1 + (200 mm / dx\_n)0.5) = **1.748**

Reinforcement ratio; l = min(0.02, Asx\_n / (b × dx\_n)) = **0.0023**

Minimum shear resistance (Exp. 6.3N); VRd,c\_min = 0.035 N/mm2  k1.5  (fck / 1 N/mm2)0.5 × b × dx\_n

VRd,c\_min = **144.4** kN/m

Shear resistance constant (cl. 6.2.2); CRd,c = 0.18 N/mm2 / C = **0.12** N/mm2

Shear resistance (Exp. 6.2a);

VRd,c\_x\_n = max(VRd,c\_min, CRd,c × k × (100 × l × (fck / 1 N/mm2))0.333 × b × dx\_n) = **144.4** kN/m

Library item - Shear output

PASS - Shear capacity is adequate

Library item - Shear pass/fail output

Shear capacity check at long span continuous support

Shear force; Vy\_n = q  lx / 2 = **63.0** kN/m

Effective depth factor (cl. 6.2.2); k = min(2.0, 1 + (200 mm / dy\_n)0.5) = **1.766**

Reinforcement ratio; l = min(0.02, Asy\_n / (b × dy\_n)) = **0.0024**

Minimum shear resistance (Exp. 6.3N); VRd,c\_min = 0.035 N/mm2  k1.5  (fck / 1 N/mm2)0.5 × b × dy\_n

VRd,c\_min = **140.0** kN/m

Shear resistance constant (cl. 6.2.2); CRd,c = 0.18 N/mm2 / C = **0.12** N/mm2

Shear resistance (Exp. 6.2a);

VRd,c\_y\_n = max(VRd,c\_min, CRd,c × k × (100 × l × (fck / 1 N/mm2))0.333 × b × dy\_n) = **140.0** kN/m

Library item - Shear output

PASS - Shear capacity is adequate

Library item - Shear pass/fail output

Basic span-to-depth deflection ratio check (cl. 7.4.2)

Reference reinforcement ratio; 0 = (fck / 1 N/mm2)0.5 / 1000 = **0.0050**

Required tension reinforcement ratio;  = max(0.0035, Asx\_p\_req / (b × dx\_p)) = **0.0035**

Required compression reinforcement ratio; ’ = Ascx\_p\_req / (b × dx\_p) = **0.0000**

Stuctural system factor (Table 7.4N); K = **1.5**

Deflection output 1

Basic limit span-to-depth ratio (Exp. 7.16);

ratiolim\_x\_bas = K  [11 +1.5(fck/1 N/mm2)0.50/ + 3.2(fck/1 N/mm2)0.5(0/ -1)1.5] = **39.31**

Mod span-to-depth ratio limit;

ratiolim\_x = min(40  K, min(1.5, (500 N/mm2/ fyk)  (Asx\_p / Asx\_p\_m))  ratiolim\_x\_bas) = **58.96**

Actual span-to-eff. depth ratio; ratioact\_x = lx / dx\_p = **14.01**

Deflection output 2

PASS - Actual span-to-effective depth ratio is acceptable

Deflection output 3

Reinforcement summary

Midspan in short span direction; **16 mm dia. bars at 250 mm centres B1**

Midspan in long span direction; **16 mm dia. bars at 250 mm centres B2**

Library item - Reinft summary xp yp

Continuous support in short span direction; **16 mm dia. bars at 250 mm centres T1**

Library item - Reinft summary xn

Continuous support in long span direction; **16 mm dia. bars at 250 mm centres T2**

Library item - Reinft summary yn

Reinforcement sketch

The following sketch is indicative only. Note that additional reinforcement may be required in accordance with clauses 9.2.1.2, 9.2.1.4 and 9.2.1.5 of EN 1992-1-1:2004 to meet detailing rules.

Library item - Reinft sketch title



Library item - Show active sketch

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EN Main Parking Beam

RC beam design

In accordance with EN1992-1-1:2004 incorporating Corrigenda January 2008 and the UK national annex

Tedds calculation version 3.3.09

Design summary

Overall design utilisation; 0.906

Overall design status; PASS

Section 1 Main RC Beam

| **Description** | **Unit** | **Provided** | **Required** | **Utilisation** | **Result** |
| --- | --- | --- | --- | --- | --- |
| Top reinforcement | mm2 | 226 | 0 | 0.000 | PASS |
| Bottom reinforcement | mm2 | 603 | 533 | 0.884 | PASS |
| Shear reinforcement | mm2/m | 335 | 304 | 0.906 | PASS |

Concrete details - Table 3.1. Strength and deformation characteristics for concrete

Concrete strength class; C40/50

Aggregate type; Quartzite

Aggregate adjustment factor - cl.3.1.3(2); AAF = **1.0**

Characteristic compressive cylinder strength; fck = **40** N/mm2

Mean value of compressive cylinder strength; fcm = fck + 8 N/mm2 = **48** N/mm2

Mean value of axial tensile strength; fctm = 0.3 N/mm2  (fck/ 1 N/mm2)2/3 = **3.5** N/mm2

Secant modulus of elasticity of concrete; Ecm = 22 kN/mm2(fcm / 10 N/mm2)0.3  AAF = **35220** N/mm2

Ultimate strain - Table 3.1; cu2 = **0.0035**

Shortening strain - Table 3.1; cu3 = **0.0035**

Effective compression zone height factor;  = **0.80**

Effective strength factor;  = **1.00**

Coefficient k1; k1 = **0.40**

Coefficient k2; k2 = 1.0  (0.6 + 0.0014 / cu2) = **1.00**

Coefficient k3; k3 = **0.40**

Coefficient k4; k4 = 1.0  (0.6 + 0.0014 / cu2) = **1.00**

Partial factor for concrete -Table 2.1N; C = **1.50**

Compressive strength coefficient - cl.3.1.6(1); cc = **0.85**

Design compressive concrete strength - exp.3.15; fcd = cc  fck / C = **22.7** N/mm2

Compressive strength coefficient - cl.3.1.6(1); ccw = **1.00**

Design compressive concrete strength - exp.3.15; fcwd = ccw  fck / C = **26.7** N/mm2

Maximum aggregate size; hagg = **20** mm

Density of reinforced concrete;  = **2500** kg/m3

Monolithic simple support moment factor; 1 = **0.25**

Reinforcement details

Characteristic yield strength of reinforcement; fyk = **500** N/mm2

Partial factor for reinforcing steel - Table 2.1N; S = **1.15**

Design yield strength of reinforcement; fyd = fyk / S = **435** N/mm2

Nominal cover to reinforcement

Nominal cover to top reinforcement; cnom\_t = **30** mm

Nominal cover to bottom reinforcement; cnom\_b = **30** mm

Nominal cover to side reinforcement; cnom\_s = **30** mm

**Fire resistance**

Standard fire resistance period; R = **60** min

Number of sides exposed to fire; 3

Minimum width of beam - EN1992-1-2 Table 5.5; bmin = **120** mm

Section 1 - Main RC Beam

Rectangular section details

Section width; b = **300** mm

Section depth; h = **500** mm

PASS - Minimum dimensions for fire resistance met



Positive moment - section 6.1

Design bending moment; M = Mpos\_s1 = **100.0** kNm

Effective depth of tension reinforcement; d = **454** mm

Redistribution ratio;  = min(pos\_s1, 1) = **1.000**

K = M / (b  d2  fck) = **0.040**

K' = (2    cc / C)  (1 -   ( - k1) / (2  k2))  (  ( - k1) / (2  k2)) = **0.207**

K' > K - No compression reinforcement is required

Lever arm; z = min(0.5  d  [1 + (1 - 2  K / (  cc / C))0.5], 0.95  d) = **431** mm

Depth of neutral axis; x = 2  (d - z) /  = **57** mm

Area of tension reinforcement required; As,req = M / (fyd  z) = **533** mm2

Tension reinforcement provided; 3  16

Area of tension reinforcement provided; As,prov = **603** mm2

Minimum area of reinforcement - exp.9.1N; As,min = max(0.26  fctm / fyk, 0.0013)  b  d = **249** mm2

Maximum area of reinforcement - cl.9.2.1.1(3); As,max = 0.04  b  h = **6000** mm2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control - Section 7.3

Maximum crack width; wk = **0.30** mm

Design value modulus of elasticity reinf – 3.2.7(4); Es = **200000** N/mm2

Mean value of concrete tensile strength; fct,eff = fctm = **3.5** N/mm2

Stress distribution coefficient; kc = **0.4**

Non-uniform self-equilibrating stress coefficient; k = min(max(1 + (300 mm - min(h, b))  0.35 / 500 mm, 0.65), 1) = **1.00**

Actual tension bar spacing; sbar = (b - (2  (cnom\_s + s1\_v) + s1\_b\_L1  Ns1\_b\_L1)) / (Ns1\_b\_L1 - 1) + s1\_b\_L1 = **104** mm

Maximum stress permitted - Table 7.3N; s = **317** N/mm2

Steel to concrete modulus of elast. ratio; cr = Es / Ecm = **5.68**

Distance of the Elastic NA from bottom of beam; y = (b  h2 / 2 + As,prov  (cr - 1)  (h - d)) / (b  h + As,prov  (cr - 1)) = **246** mm

Area of concrete in the tensile zone; Act = b  y = **73870** mm2

Minimum area of reinforcement required - exp.7.1; Asc,min = kc  k  fct,eff  Act / s = **327** mm2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent moment; MQP = Mpos\_QP\_s1 = **65.0**kNm

Permanent load ratio; RPL = MQP / M = **0.65**

Service stress in reinforcement; sr = fyd  As,req / As,prov  RPL = **250** N/mm2

Maximum bar spacing - Tables 7.3N; sbar,max = **187.7** mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Minimum bar spacing (Section 8.2)

Top bar spacing; stop = (b - (2  (cnom\_s + s1\_v) + s1\_t\_L1  Ns1\_t\_L1)) / (Ns1\_t\_L1 - 1) = **200.0** mm

Minimum allowable top bar spacing; stop,min = max(s1\_t\_L1  ks1, hagg + ks2, 20mm) = **25.0** mm

PASS - Actual bar spacing exceeds minimum allowable

Bottom bar spacing; sbot = (b - (2  (cnom\_s + s1\_v) + s1\_b\_L1  Ns1\_b\_L1)) / (Ns1\_b\_L1 - 1) = **88.0** mm

Minimum allowable bottom bar spacing; sbot,min = max(s1\_b\_L1  ks1, hagg + ks2, 20mm) = **25.0** mm

PASS - Actual bar spacing exceeds minimum allowable

Section in shear (section 6.2)

Angle of comp. shear strut for maximum shear; max = 45 deg

Strength reduction factor - cl.6.2.3(3); v1 = 0.6  (1 - fck / 250 N/mm2) = **0.504**

Compression chord coefficient - cl.6.2.3(3); cw = **1.00**

Minimum area of shear reinforcement - exp.9.5N; Asv,min = 0.08 N/mm2  b  (fck / 1 N/mm2)0.5 / fyk = **304** mm2/m

Design shear force at support ; VEd,max = VEd,max\_s1 = **50** kN

Min lever arm in shear zone; z = **431** mm

Maximum design shear resistance - exp.6.9; VRd,max = cw  b  z  v1  fcwd / (cot(max) + tan(max)) = **870** kN

PASS - Design shear force at support is less than maximum design shear resistance

Design shear force ; VEd = **50** kN

Design shear stress; vEd = VEd / (b  z) = **0.386** N/mm2

Angle of concrete compression strut - cl.6.2.3;  = min(max(0.5  Asin(min(2  vEd / (cw  fcwd  v1),1)), 21.8 deg), 45deg) = **21.8** deg

Area of shear reinforcement required - exp.6.8; Asv,des = vEd  b / (fyd  cot()) = **107** mm2/m

Area of shear reinforcement required; Asv,req = max(Asv,min, Asv,des) = **304** mm2/m

Shear reinforcement provided; 2  8 legs @ 300 c/c

Area of shear reinforcement provided; Asv,prov = **335** mm2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N; svl,max = 0.75  d = **341** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

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EN RC Parking column

RC column design (EN 1992)

In accordance with EN1992-1-1:2004 incorporating Corrigendum January 2008 and the UK national annex

Tedds calculation version 1.4.05

Design summary

| **Description** | **Unit** | **Provided** | **Required** | **Utilisation** | **Result** |
| --- | --- | --- | --- | --- | --- |
| Moment capacity (y) | kNm | 457 | 117 | 0.26 | PASS |
| Moment capacity (z) | kNm | 263 | 109 | 0.41 | PASS |
| Biaxial bending |  |  |  | 0.52 | PASS |



Library item - Show column section sketch

Column input details

Column geometry

Overall depth (perpendicular to y axis); h = **550** mm

Overall breadth (perpendicular to z axis); b = ;**350**; mm

Stability in the z direction; **Unbraced**

Stability in the y direction; **Unbraced**

Concrete details

Concrete strength class; C40/50

Partial safety factor for concrete (2.4.2.4(1)); C = **1.50**

Coefficient cc (3.1.6(1)); cc = **0.85**

Library item - Col dets no clear ht stab

Maximum aggregate size; dg = **20** mm

Reinforcement details

Nominal cover to links; cnom = **50** mm

Longitudinal bar diameter;  = **25** mm

Link diameter; v = **8** mm

Total number of longitudinal bars; N = **4**

No. of bars per face parallel to y axis; Ny = ;**2**

No. of bars per face parallel to z axis; Nz = ;**2**

Area of longitudinal reinforcement; As = N ×  × 2 / 4 = **1963** mm2

Characteristic yield strength; fyk = **500** N/mm2

Partial safety factor for reinft (2.4.2.4(1)); S = **1.15**

Es = 200000 MPa

Modulus of elasticity of reinft (3.2.7(4)); Es = **200** kN/mm2

Fire resistance details

Fire resistance period; R = **60** min

Exposure to fire; **Exposed on more than one side**

Ratio of fire design axial load to design resistance; fi = **0.70**

Library item - Reinf fire details

Axial load and bending moments from frame analysis

Design axial load; NEd = **2000.0** kN

Moment about y axis at top; Mtopy = ;**75.0**; kNm

Moment about y axis at bottom; Mbtmy = ;**75.0**; kNm

Moment about z axis at top; Mtopz = ;**50.0**; kNm

Moment about z axis at bottom; Mbtmz = ;**50.0**; kNm

Library item - Loading details

Column effective lengths

Effective length for buckling about y axis; l0y = ;**3000**; mm

Effective length for buckling about z axis; l0z = ;**3000**; mm

Library item - Eff length direct

Calculated column properties

Concrete properties

Area of concrete; Ac = h  b = **192500** mm2

Characteristic compression cylinder strength; fck = **40** N/mm2

Design compressive strength (3.1.6(1)); fcd = cc × fck / C = **22.7** N/mm2

Mean value of cylinder strength (Table 3.1); fcm = fck + 8 MPa = **48.0** N/mm2

Secant modulus of elasticity (Table 3.1); Ecm = 22000 MPa × (fcm / 10 MPa)0.3 = **35.2** kN/mm2

Library item - Design strength conc

Rectangular stress block factors

Depth factor (3.1.7(3)); sb = 0.8

Stress factor (3.1.7(3));  = 1.0

Strain limits

cu3 = 0.0035

Compression strain limit (Table 3.1); cu3 = **0.00350**

c3 = 0.00175

Pure compression strain limit (Table 3.1); c3 = **0.00175**

Library item - Stress block factors fck lte 50

Design yield strength of reinforcement

Design yield strength (3.2.7(2)); fyd = fyk / S = **434.8** N/mm2

Library item - Reinforcement properties

Check nominal cover for fire and bond requirements

Min. cover reqd for bond (to links) (4.4.1.2(3)); cmin,b = max(v,  - v) = **17** mm

Min axis distance for fire (EN1992-1-2 T 5.2a); afi = **40** mm

Allowance for deviations from min cover (4.4.1.3); cdev = **10** mm

Min allowable nominal cover; cnom\_min = max(afi -  / 2 - v, cmin,b + cdev) = **27.0** mm

PASS - the nominal cover is greater than the minimum required

Library item - Cover check

Effective depths of bars for bending about y axis

Area per bar; Abar =  × 2 / 4 = **491** mm2

Spacing of bars in faces parallel to z axis (c/c); sz = (h - 2 × (cnom + v) - ) / (Nz - 1) = **409** mm

Layer 1 (in tension face); dy1 = h - cnom - v -  / 2 = **480** mm

Layer 2; dy2 = dy1 - sz = **70** mm

Library item - Depth layer 1 2 y

Effective depth about y axis; dy = dy1 = **480** mm

Library item - Eff depth y 2 layers

Effective depths of bars for bending about z axis

Area of per bar; Abar =  × 2 / 4 = **491** mm2

Spacing of bars in faces parallel to y axis (c/c); sy = (b - 2 × (cnom + v) - ) / (Ny - 1) = **209** mm

Layer 1 (in tension face); dz1 = b - cnom - v -  / 2 = **280** mm

Layer 2; dz2 = dz1 - sy = **70** mm

Library item - Depth layer 1 2 z

Effective depth about z axis; dz = dz1 = **280** mm

Library item - Eff depth z 2 layers

Column slenderness about y axis

Radius of gyration; iy = h / (12) = **15.9** cm

Slenderness ratio (5.8.3.2(1)); y = l0y / iy = **18.9**

Library item - Column slend y

Column slenderness about z axis

Radius of gyration; iz = b / (12) = **10.1** cm

Slenderness ratio (5.8.3.2(1)); z = l0z / iz = **29.7**

Library item - Column slend z

Design bending moments

Frame analysis moments about y axis combined with moments due to imperfections (cl. 5.2 & 6.1(4))

Ecc. due to geometric imperfections (y axis); eiy = l0y /400 = **7.5** mm

Min end moment about y axis; M01y = min(abs(Mtopy), abs(Mbtmy)) + eiy × NEd = **90.0** kNm

Max end moment about y axis; M02y = max(abs(Mtopy), abs(Mbtmy)) + eiy × NEd = **90.0** kNm

**Slenderness limit for buckling about y axis (cl. 5.8.3.1)**

A = 0.7

Factor A; A = **0.7**

Mechanical reinforcement ratio;  = As × fyd / (Ac × fcd) = **0.196**

Factor B; B = √(1 + 2 × ) = **1.180**

rmy = if(and(\_rcc.BracedY == 1, max(abs(Mtopy),abs(Mbtmy))>=NEd × eiy) , if(or(and(Mtopy>=0 kNm,Mbtmy>=0 kNm), and(Mtopy<=0 kNm,Mbtmy<=0 kNm)),M01y / M02y ,-1.0 × M01y / M02y), 1.0) = **1.000**

Moment ratio; ××rmy = ;**1.000**

Factor C; Cy = 1.7 - rmy = **0.700**

Relative normal force; n = NEd / (Ac × fcd) = **0.458**

Slenderness limit; limy = 20 × A × B × Cy / √(n) = **17.1**

y>=limy - Second order effects must be considered

Library item - Slenderness limit y

Frame analysis moments about z axis combined with moments due to imperfections (cl. 5.2 & 6.1(4))

Ecc. due to geometric imperfections (z axis); eiz = l0z /400 = **7.5** mm

Min end moment about z axis; M01z = min(abs(Mtopz), abs(Mbtmz)) + eiz × NEd = **65.0** kNm

Max end moment about z axis; M02z = max(abs(Mtopz), abs(Mbtmz)) + eiz × NEd = **65.0** kNm

Slenderness limit for buckling about y axis (cl. 5.8.3.1)

A = 0.7

Factor A; A = **0.7**

Mechanical reinforcement ratio;  = As × fyd / (Ac × fcd) = **0.196**

Factor B; B = √(1 + 2 × ) = **1.180**

rmz = if(and(\_rcc.BracedZ == 1, max(abs(Mtopz),abs(Mbtmz))>=NEd × eiz) , if(or(and(Mtopz>=0 kNm,Mbtmz>=0 kNm), and(Mtopz<=0 kNm,Mbtmz<=0 kNm)),M01z / M02z ,-1.0 × M01z / M02z), 1.0) = **1.000**

Moment ratio; ××rmz = ;**1.000**

Factor C; Cz = 1.7 - rmz = **0.700**

Relative normal force; n = NEd / (Ac × fcd) = **0.458**

Slenderness limit; limz = 20 × A × B × Cz / √(n) = **17.1**

z>=limz - Second order effects must be considered

Library item - Slenderness limit z

Local second order bending moment about y axis (cl. 5.8.8.2 & 5.8.8.3)

Library item - 2nd order title y

Relative humidity of ambient environment; RH = **50** %

Column perimeter in contact with atmosphere; u = **1800** mm

Age of concrete at loading; t0 = **28** day

Parameter nu; nu = 1 +  = **1.196**

nbal = 0.4

Approx value of n at max moment of resistance; nbal = **0.4**

Axial load correction factor; Kr = min(1.0 , (nu - n) / (nu - nbal)) = **0.927**

Reinforcement design strain; yd = fyd / Es = **0.00217**

Library item - Load correction factor Kr

Basic curvature; curvebasic\_y = yd / (0.45 × dy) = **0.0000101** mm-1

Library item - Basic curve y

Notional size of column; h0 = 2 × Ac / u = **214** mm

Factor 1 (Annex B.1(1)); 1 = (35 MPa / fcm)0.7 = **0.802**

Factor 2 (Annex B.1(1)); 2 = (35 MPa / fcm)0.2 = **0.939**

Relative humidity factor (Annex B.1(1)); RH = [1 + ((1 - RH / 100%) / (0.1 mm-1/3 × (h0)1/3)) × 1] × 2 = **1.568**

Library item - Humidity factor fcm gt 35

Concrete strength factor (Annex B.1(1)); fcm = 16.8 × (1 MPa)1/2 / √(fcm) = **2.425**

Concrete age factor (Annex B.1(1)); t0 = 1 / (0.1 + (t0 / 1 day)0.2) = **0.488**

Notional creep coefficient (Annex B.1(1)); 0 = RH × fcm × t0 = **1.857**

Final creep development factor; (at t = ; c = 1.0

Final creep coefficient (Annex B.1(1));  = 0 × c = **1.857**

Library item - Final creep coeff

Ratio of SLS to ULS moments; rMy = **0.80**

Effective creep ratio; efy =  × rMy = **1.486**

Factor ; y = 0.35 + fck / 200 MPa - y / 150 = **0.424**

Creep factor; Ky = max(1.0 , 1 + y × efy) = **1.630**

Modified curvature; curvemod\_y = Kr × Ky × curvebasic\_y = **0.0000152** mm-1

c = 10

Curvature distribution factor; c = **10**

Deflection; e2y = curvemod\_y × l0y2 / c = **13.7** mm

Nominal 2nd order moment; M2y = NEd × e2y = **27.4** kNm

Library item - 2nd order mt y

Design bending moment about y axis (cl. 5.8.8.2 & 6.1(4))

M0ey = if(or(and(Mtopy>=0 kNm, Mbtmy>=0 kNm),and(Mtopy<=0 kNm, Mbtmy<=0 kNm)), max(0.6 × M02y + 0.4 × M01y, 0.4 × M02y), max(0.6 × M02y - 0.4 × M01y, 0.4 × M02y)) = **90.0** kNm

Equivalent moment from frame analysis; ××××××M0ey = max(0.6  M02y + 0.4  M01y, 0.4  M02y) = ;**90.0**; kNm

Design moment; MEdy = max(M02y, M0ey + M2y, M01y + 0.5×M2y, NEd × max(h/30, 20 mm))

MEdy = **117.4** kNm

Library item - Design mt slender y

Local second order bending moment about z axis (cl. 5.8.8.2 & 5.8.8.3)

Library item - 2nd order title z

Basic curvature; curvebasic\_z = yd / (0.45 × dz) = **0.0000173** mm-1

Library item - Basic curve z

Ratio of SLS to ULS moments; rMz = **0.80**

Effective creep ratio (5.8.4(2)); efz =  × rMz = **1.486**

Factor ; z = 0.35 + fck / 200 MPa - z / 150 = **0.352**

Creep factor; Kz = max(1.0 , 1 + z × efz) = **1.523**

Modified curvature; curvemod\_z = Kr × Kz × curvebasic\_z = **0.0000244** mm-1

c = 10

Curvature distribution factor; c = **10**

Deflection; e2z = curvemod\_z × l0z2 / c = **22.0** mm

Nominal 2nd order moment; M2z = NEd × e2z = **43.9** kNm

Library item - 2nd order mt z

Design bending moment about z axis (cl. 5.8.8.2 & 6.1(4))

M0ez = if(or(and(Mtopz>=0 kNm, Mbtmz>=0 kNm),and(Mtopz<=0 kNm, Mbtmz<=0 kNm)), max(0.6 × M02z + 0.4 × M01z, 0.4 × M02z), max(0.6 × M02z - 0.4 × M01z, 0.4 × M02z)) = **65.0** kNm

Equivalent moment from frame analysis; ××××××M0ez = max(0.6  M02z + 0.4  M01z, 0.4  M02z) = ;**65.0**; kNm

Design moment; MEdz = max(M02z, M0ez + M2z, M01z + 0.5×M2z, NEd × max(b/30, 20 mm))

MEdz = **108.9** kNm

Library item - Design mt slender z

Moment capacity about y axis with axial load (2000.0 kN)

Moment of resistance of concrete

By iteration:-

Position of neutral axis; y = **310.0** mm

Concrete compression force (3.1.7(3)); Fyc = × fcd × min(sb × y , h) × b = **1967.6** kN

Moment of resistance; MRdyc = Fyc × [h / 2 - (min(sb × y , h)) / 2] = **297.1** kNm

Moment of resistance of reinforcement

Library item - M of R concrete y

Strain in layer 1; y1 = cu3 × (1 - dy1 / y) = **-0.00191**

Library item - Reinft strain inside y

Stress in layer 1; y1 = max(-1×fyd, Es × y1) = **-382.7** N/mm2

Library item - Reinft stress y -ve strain

Force in layer 1; Fy1 = Ny × Abar × y1 = **-375.7** kN

Moment of resistance of layer 1; MRdy1 = Fy1 × (h / 2 - dy1) = **76.8** kNm

Library item - M of R reinft y

Strain in layer 2; y2 = cu3 × (1 - dy2 / y) = **0.00270**

Library item - Reinft strain inside y

Stress in layer 2; y2 = min(fyd, Es × y2) -  × fcd = **412.1** N/mm2

Library item - Reinft stress y +ve strain comp

Force in layer 2; Fy2 = Ny × Abar × y2 = **404.6** kN

Moment of resistance of layer 2; MRdy2 = Fy2 × (h / 2 - dy2) = **82.7** kNm

Library item - M of R reinft y

Resultant concrete/steel force; Fy = **1996.5** kN

PASS - This is within half of one percent of the applied axial load

Combined moment of resistance

Moment of resistance about y axis; MRdy = **456.7** kNm

PASS - The moment capacity about the y axis exceeds the design bending moment

Library item - Total M of R y

Moment capacity about z axis with axial load (2000.0 kN)

Moment of resistance of concrete

By iteration:-

Position of neutral axis; z = **191.8** mm

Concrete compression force (3.1.7(3)); Fzc = × fcd × min(sb × z , b) × h = **1913.0** kN

Moment of resistance; MRdzc = Fzc × [b / 2 - (min(sb × z , b)) / 2] = **188.0** kNm

Moment of resistance of reinforcement

Library item - M of R concrete z

Strain in layer 1; z1 = cu3 × (1 - dz1 / z) = **-0.00160**

Library item - Reinft strain inside z

Stress in layer 1; z1 = max(-1×fyd, Es × z1) = **-320.0** N/mm2

Library item - Reinft stress z -ve strain

Force in layer 1; Fz1 = Nz × Abar × z1 = **-314.2** kN

Moment of resistance of layer 1; MRdz1 = Fz1 × (b / 2 - dz1) = **32.8** kNm

Library item - M of R reinft z

Strain in layer 2; z2 = cu3 × (1 - dz2 / z) = **0.00221**

Library item - Reinft strain inside z

Stress in layer 2; z2 = min(fyd, Es × z2) -  × fcd = **412.1** N/mm2

Library item - Reinft stress z +ve strain comp

Force in layer 2; Fz2 = Nz × Abar × z2 = **404.6** kN

Moment of resistance of layer 2; MRdz2 = Fz2 × (b / 2 - dz2) = **42.3** kNm

Library item - M of R reinft z

Resultant concrete/steel force; Fz = **2003.5** kN

PASS - This is within half of one percent of the applied axial load

Combined moment of resistance

Moment of resistance about z axis; MRdz = **263.1** kNm

PASS - The moment capacity about the z axis exceeds the design bending moment

Library item - Total M of R z

Biaxial bending

Determine if a biaxial bending check is required (5.8.9(3))

Ratio of column slenderness ratios; ratio = max(y, z) / min(y, z) = **1.57**

Eccentricity in direction of y axis; ey = MEdz / NEd = **54.5** mm

Eccentricity in direction of z axis; ez = MEdy / NEd = **58.7** mm

Equivalent depth; heq = iy × √(12) = **550** mm

Equivalent width; beq = iz × √(12) = **350** mm

Relative eccentricity in direction of y axis; erel\_y = ey / beq = **0.156**

Relative eccentricity in direction of z axis; erel\_z = ez / heq = **0.107**

Ratio of relative eccentricities; ratioe = min(erel\_y, erel\_z) / max(erel\_y, erel\_z) = **0.686**

ratioe > 0.2 - Biaxial bending check is required

Library item - Biaxial bending initial check

Biaxial bending (5.8.9(4))

Design axial resistance of section; NRd = (Ac × fcd) + (As × fyd) = **5217.0** kN

Ratio of applied to resistance axial loads; ratioN = NEd / NRd = **0.383**

a = if(ratioN <= 0.1, 1.0, if(ratioN <= 0.7, 1.0 + 0.5 × (ratioN - 0.1) / 0.6, if(ratioN <= 1.0, 1.5 + 0.5 × (ratioN - 0.7) / 0.3, 2))) = **1.24**

Exponent a; a = **1.24**

Library item - Biaxial exponent a

Biaxial bending utilisation; UF = (MEdy / MRdy)a + (MEdz / MRdz)a = **0.523**

PASS - The biaxial bending capacity is adequate

Library item - Biaxial bending check

;

EN Pad Footing

\_exVarsLib = "$(SysLbrDir)Foundations-EN1997-si-engb.lbr"

\_exVarsItem = "Example variables 01"

Eval( if( GetVar("\_exFirstCalculate", True) == True, "EvalCalcItem(\_exVarsLib, \_exVarsItem)", "" )) = **0.000**

\_exFirstCalculate = False

Pad foundation example

Foundation analysis in accordance with EN1997-1:2004 + A1:2013 incorporating corrigendum February 2009 and the UK National Annex incorporating corrigendum No.1

Tedds calculation version 3.3.05

Summary table

| Description | Unit | Allowable | Actual | Utilisation | Result |
| --- | --- | --- | --- | --- | --- |
| Base pressure | kN/m2 | 766.9 | 226.1 | 0.295 | Pass |
| Description | Unit | Provided | Required | Utilisation | Result |
| Reinforcement x-direction | mm2 | 5890 | 5706 | 0.969 | Pass |
| Reinforcement y-direction | mm2 | 5890 | 5624 | 0.955 | Pass |
| Description | Unit | Allowable | Actual | Utilisation | Result |
| Punching shear | N/mm2 | 4.284 | 0.366 | 0.086 | Pass |
|  |  |  |  |  |  |
|  |  |  |  |  |  |

Pad foundation details

Length of foundation; Lx = **1800** mm

Width of foundation; Ly = **1800** mm

Foundation area; A = Lx × Ly = **3.240** m2

Depth of foundation; h = **1800** mm

Depth of soil over foundation; hsoil = **2200** mm

Level of water; hwater = **0** mm

Density of water; water = **9.8** kN/m3

Density of concrete; conc = **24.5** kN/m3

Library item: Foundation details output



Library item: Show foundation sketch

Column no.1 details

Length of column; lx1 = **300** mm

Width of column; ly1 = **300** mm

position in x-direction; x1 = **900** mm

position in y-direction; y1 = **900** mm

Library item: Column details output

Soil properties

Density of soil; soil = **20.0** kN/m3

Characteristic cohesion; c'k = **0** kN/m2

Characteristic effective shear resistance angle; 'k = **25** deg

Characteristic friction angle; k = **19.3** deg

Library item: Drained soil props output

Foundation loads

Self weight; Fswt = h  conc = **44.1** kN/m2

Soil weight; Fsoil = hsoil  soil = **44.0** kN/m2

Column no.1 loads

Permanent axial load; FGz1 = **200.0** kN

Variable axial load; FQz1 = **165.0** kN

Permanent moment in x-direction; MGx1 = **15.0** kNm

Variable moment in x-direction; MQx1 = **10.0** kNm

Design approach 1

Partial factors on actions - Combination1

Partial factor set; A1

Permanent unfavourable action - Table A.3; G = **1.35**

Permanent favourable action - Table A.3; Gf = **1.00**

Variable unfavourable action - Table A.3; Q = **1.50**

Variable favourable action - Table A.3; Qf = **0.00**

Library item; PF actions output

Partial factors for soil parameters - Combination1

Soil factor set; M1

Angle of shearing resistance - Table A.4; ' = **1.00**

Effective cohesion - Table A.4; c' = **1.00**

Weight density - Table A.4;  = **1.00**

Library item; PF soils output

Partial factors for spread foundations - Combination1

Resistance factor set; R1

Bearing - Table A.5; R.v = **1.00**

Sliding - Table A.5; R.h = **1.00**

Library item; PF spread founds output

Bearing resistance (Section 6.5.2)

Forces on foundation

Force in z-direction; Fdz = G  (A  (Fswt + Fsoil) + FGz1) + Q  FQz1 = **902.8** kN

Moments on foundation

Moment in x-direction; Mdx = G  (A  (Fswt + Fsoil)  Lx / 2 + FGz1  x1) + G  MGx1 + Q  FQz1  x1 + Q  MQx1 = **847.8** kNm

Moment in y-direction; Mdy = G  (A  (Fswt + Fsoil)  Ly / 2 + FGz1  y1) + Q  FQz1  y1 = **812.6** kNm

Eccentricity of base reaction

Eccentricity of base reaction in x-direction; ex = Mdx / Fdz - Lx / 2 = **39** mm

Eccentricity of base reaction in y-direction; ey = Mdy / Fdz - Ly / 2 = **0** mm

Effective area of base

Effective length; L'x = Lx - 2  ex = **1722** mm

Effective width; L'y = Ly - 2  ey = **1800** mm

Effective area; A' = L'x × L'y = **3.099** m2

Pad base pressure

Design base pressure; fdz = Fdz / A' = **291.3** kN/m2

Library item: Pad ULS pressure output

Ultimate bearing capacity under drained conditions (Annex D.4)

Design angle of shearing resistance; 'd = atan(tan('k) / ') = **25.000** deg

Design effective cohesion; c'd = c'k / c' = **0.000** kN/m2

Effective overburden pressure; q = (h + hsoil) × soil - hwater × water = **80.000** kN/m2

Design effective overburden pressure; q' = q /  = **80.000** kN/m2

Bearing resistance factors; Nq = Exp( × tan('d)) × (tan(45 deg + 'd / 2))2 = **10.662**

Nc = (Nq - 1) × cot('d) = **20.721**

N = 2 × (Nq - 1) × tan('d) = **9.011**

Foundation shape factors; sq = 1 + (L'x / L'y)  sin('d) = **1.404**

s = 1 - 0.3  (L'x / L'y) = **0.713**

sc = (sq  Nq - 1) / (Nq - 1) = **1.446**

Load inclination factors; H = **0.0** kN

my = [2 + (L'y / L'x)] / [1 + (L'y / L'x)] = **1.489**

mx = [2 + (L'x / L'y)] / [1 + (L'x / L'y)] = **1.511**

m = mx = **1.511**

iq = [1 - H / (Fdz + A'  c'd  cot('d))]m = **1.000**

i = [1 - H / (Fdz + A'  c'd  cot('d))]m + 1 = **1.000**

ic = iq - (1 - iq) / (Nc × tan('d)) = **1.000**

Ultimate bearing capacity; nf = c'd  Nc  sc  ic + q'  Nq  sq  iq + 0.5  soil  L'x  N  s  i = **1308.4** kN/m2

Library item: Drained bearing output

PASS - Ultimate bearing capacity exceeds design base pressure

Design approach 1

Partial factors on actions - Combination2

Partial factor set; A2

Permanent unfavourable action - Table A.3; G = **1.00**

Permanent favourable action - Table A.3; Gf = **1.00**

Variable unfavourable action - Table A.3; Q = **1.30**

Variable favourable action - Table A.3; Qf = **0.00**

Library item; PF actions output

Partial factors for soil parameters - Combination2

Soil factor set; M2

Angle of shearing resistance - Table A.4; ' = **1.25**

Effective cohesion - Table A.4; c' = **1.25**

Weight density - Table A.4;  = **1.00**

Library item; PF soils output

Partial factors for spread foundations - Combination2

Resistance factor set; R1

Bearing - Table A.5; R.v = **1.00**

Sliding - Table A.5; R.h = **1.00**

Library item; PF spread founds output

Bearing resistance (Section 6.5.2)

Forces on foundation

Force in z-direction; Fdz = G  (A  (Fswt + Fsoil) + FGz1) + Q  FQz1 = **699.9** kN

Moments on foundation

Moment in x-direction; Mdx = G  (A  (Fswt + Fsoil)  Lx / 2 + FGz1  x1) + G  MGx1 + Q  FQz1  x1 + Q  MQx1 = **657.9** kNm

Moment in y-direction; Mdy = G  (A  (Fswt + Fsoil)  Ly / 2 + FGz1  y1) + Q  FQz1  y1 = **629.9** kNm

Eccentricity of base reaction

Eccentricity of base reaction in x-direction; ex = Mdx / Fdz - Lx / 2 = **40** mm

Eccentricity of base reaction in y-direction; ey = Mdy / Fdz - Ly / 2 = **0** mm

Effective area of base

Effective length; L'x = Lx - 2  ex = **1720** mm

Effective width; L'y = Ly - 2  ey = **1800** mm

Effective area; A' = L'x × L'y = **3.096** m2

Pad base pressure

Design base pressure; fdz = Fdz / A' = **226.1** kN/m2

Library item: Pad ULS pressure output

Ultimate bearing capacity under drained conditions (Annex D.4)

Design angle of shearing resistance; 'd = atan(tan('k) / ') = **20.458** deg

Design effective cohesion; c'd = c'k / c' = **0.000** kN/m2

Effective overburden pressure; q = (h + hsoil) × soil - hwater × water = **80.000** kN/m2

Design effective overburden pressure; q' = q /  = **80.000** kN/m2

Bearing resistance factors; Nq = Exp( × tan('d)) × (tan(45 deg + 'd / 2))2 = **6.698**

Nc = (Nq - 1) × cot('d) = **15.273**

N = 2 × (Nq - 1) × tan('d) = **4.251**

Foundation shape factors; sq = 1 + (L'x / L'y)  sin('d) = **1.334**

s = 1 - 0.3  (L'x / L'y) = **0.713**

sc = (sq  Nq - 1) / (Nq - 1) = **1.393**

Load inclination factors; H = **0.0** kN

my = [2 + (L'y / L'x)] / [1 + (L'y / L'x)] = **1.489**

mx = [2 + (L'x / L'y)] / [1 + (L'x / L'y)] = **1.511**

m = mx = **1.511**

iq = [1 - H / (Fdz + A'  c'd  cot('d))]m = **1.000**

i = [1 - H / (Fdz + A'  c'd  cot('d))]m + 1 = **1.000**

ic = iq - (1 - iq) / (Nc × tan('d)) = **1.000**

Ultimate bearing capacity; nf = c'd  Nc  sc  ic + q'  Nq  sq  iq + 0.5  soil  L'x  N  s  i = **766.9** kN/m2

Library item: Drained bearing output

PASS - Ultimate bearing capacity exceeds design base pressure

Pad foundation example

Foundation design in accordance with EN1992-1-1:2004 + A1:2014 incorporating corrigenda January 2008, November 2010 and January 2014 and the UK National Annex incorporating National Amendment No.1 and No.2

Tedds calculation version 3.3.05

Concrete details (Table 3.1 - Strength and deformation characteristics for concrete)

Concrete strength class; C40/50

Characteristic compressive cylinder strength; fck = **40** N/mm2

Characteristic compressive cube strength; fck,cube = **50** N/mm2

Mean value of compressive cylinder strength; fcm = fck + 8 N/mm2 = **48** N/mm2

Mean value of axial tensile strength; fctm = 0.3 N/mm2 × (fck/ 1 N/mm2)2/3 = **3.5** N/mm2

5% fractile of axial tensile strength; fctk,0.05 = 0.7 × fctm = **2.5** N/mm2

Secant modulus of elasticity of concrete; Ecm = 22 kN/mm2 × [fcm/10 N/mm2]0.3 = **35220** N/mm2

Partial factor for concrete (Table 2.1N); C = **1.50**

Compressive strength coefficient (cl.3.1.6(1)); cc = **0.85**

Design compressive concrete strength (exp.3.15); fcd = cc × fck / C = **22.7** N/mm2

Tens.strength coeff.for plain concrete (cl.12.3.1(1)); ct,pl = **0.80**

Des.tens.strength for plain concrete (exp.12.1); fctd,pl = ct,pl × fctk,0.05 / C = **1.3** N/mm2

Maximum aggregate size; hagg = **20** mm

Ultimate strain - Table 3.1; cu2 = **0.0035**

Shortening strain - Table 3.1; cu3 = **0.0035**

Effective compression zone height factor;  = **0.80**

Effective strength factor;  = **1.00**

Bending coefficient k1; K1 = **0.40**

Bending coefficient k2; K2 = 1.00  (0.6 + 0.0014/cu2) =**1.00**

Bending coefficient k3; K3 =**0.40**

Bending coefficient k4; K4 =1.00  (0.6 + 0.0014/cu2) = **1.00**

Library item: Material details

Reinforcement details

Characteristic yield strength of reinforcement; fyk = **500** N/mm2

Modulus of elasticity of reinforcement; Es = **210000** N/mm2

Partial factor for reinforcing steel (Table 2.1N); S = **1.15**

Design yield strength of reinforcement; fyd = fyk / S = **435** N/mm2

Nominal cover to top of foundation; cnom\_t = **50** mm

Nominal cover to bottom of foundation; cnom\_b = **50** mm

Nominal cover to side of foundation; cnom\_s = **50** mm

Library item – Reinforcement details

Shear diagram, x axis (kN)



Library item: Show foundation sketch

Moment diagram, x axis (kNm)



Library item: Show foundation sketch

Rectangular section in flexure (Section 6.1)

Design bending moment; MEd.x.max = **94.1** kNm

Depth to tension reinforcement; d = h - cnom\_b - x.bot / 2 = **1738** mm

K = MEd.x.max / (Ly  d2  fck) = **0.000**

K' = (2    cc/C)(1 -   ( - K1)/(2  K2))(  ( - K1)/(2  K2))

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm; z = min(0.5 + 0.5  (1 - 2  K / (  cc / C))0.5, 0.95)  d = **1651** mm

Depth of neutral axis; x = 2.5 × (d - z) = **217** mm

Area of tension reinforcement required; Asx.bot.req = MEd.x.max / (fyd  z) = **131** mm2

Tension reinforcement provided; 12 No.25  bars bottom (150 c/c)

Area of tension reinforcement provided; Asx.bot.prov = **5890** mm2

Minimum area of reinforcement (exp.9.1N); As.min = max(0.26  fctm / fyk, 0.0013)  Ly  d = **5706** mm2

Maximum area of reinforcement (cl.9.2.1.1(3)); As.max = 0.04  Ly  d = **125100** mm2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Library item: Rectangular single output

Crack control (Section 7.3)

Limiting crack width; wmax = **0.3** mm

Variable load factor (EN1990 – Table A1.1); 2 = **0.3**

Serviceability bending moment; Msls.x.max = **45.8** kNm

Tensile stress in reinforcement; s = Msls.x.max / (Asx.bot.prov  z) = **4.7** N/mm2

Load duration factor; kt = **0.4**

Effective depth of concrete in tension; hc.ef = min(2.5  (h - d), (h - x) / 3, h / 2) = **156** mm

Effective area of concrete in tension; Ac.eff = hc.ef  Ly = **281250** mm2

Mean value of concrete tensile strength; fct.eff = fctm = **3.5** N/mm2

Reinforcement ratio; p.eff = Asx.bot.prov / Ac.eff = **0.021**

Modular ratio; e = Es / Ecm = **5.962**

Bond property coefficient; k1 = **0.8**

Strain distribution coefficient; k2 = **0.5**

k3 = 3.4 =**3.4**

k4 = **0.425**

Maximum crack spacing (exp.7.11); sr.max = k3  cnom\_b + k1  k2  k4  x.bot / p.eff = **373** mm

Maximum crack width (exp.7.8); wk = sr.max × max([s – kt × (fct.eff / p.eff) ×(1 + e × p.eff)] / Es,   
0.6 × s / Es) = **0.005** mm

PASS - Maximum crack width is less than limiting crack width

Library item: Crack width output

Shear diagram, y axis (kN)



Library item: Show foundation sketch

Moment diagram, y axis (kNm)



Library item: Show foundation sketch

Rectangular section in flexure (Section 6.1)

Design bending moment; MEd.y.max = **80.9** kNm

Depth to tension reinforcement; d = h - cnom\_b - x.bot - y.bot / 2 = **1713** mm

K = MEd.y.max / (Lx  d2  fck) = **0.000**

K' = (2    cc/C)(1 -   ( - K1)/(2  K2))(  ( - K1)/(2  K2))

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm; z = min(0.5 + 0.5  (1 - 2  K / (  cc / C))0.5, 0.95)  d = **1627** mm

Depth of neutral axis; x = 2.5 × (d - z) = **214** mm

Area of tension reinforcement required; Asy.bot.req = MEd.y.max / (fyd  z) = **114** mm2

Tension reinforcement provided; 12 No.25  bars bottom (150 c/c)

Area of tension reinforcement provided; Asy.bot.prov = **5890** mm2

Minimum area of reinforcement (exp.9.1N); As.min = max(0.26  fctm / fyk, 0.0013)  Lx  d = **5624** mm2

Maximum area of reinforcement (cl.9.2.1.1(3)); As.max = 0.04  Lx  d = **123300** mm2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Library item: Rectangular single output

Crack control (Section 7.3)

Limiting crack width; wmax = **0.3** mm

Variable load factor (EN1990 – Table A1.1); 2 = **0.3**

Serviceability bending moment; Msls.y.max = **39** kNm

Tensile stress in reinforcement; s = Msls.y.max / (Asy.bot.prov  z) = **4.1** N/mm2

Load duration factor; kt = **0.4**

Effective depth of concrete in tension; hc.ef = min(2.5  (h - d), (h - x) / 3, h / 2) = **219** mm

Effective area of concrete in tension; Ac.eff = hc.ef  Lx = **393750** mm2

Mean value of concrete tensile strength; fct.eff = fctm = **3.5** N/mm2

Reinforcement ratio; p.eff = Asy.bot.prov / Ac.eff = **0.015**

Modular ratio; e = Es / Ecm = **5.962**

Bond property coefficient; k1 = **0.8**

Strain distribution coefficient; k2 = **0.5**

k3 = 3.4 =**3.4**

k4 = **0.425**

Maximum crack spacing (exp.7.11); sr.max = k3  (cnom\_b + x.bot) + k1  k2  k4  y.bot / p.eff = **539** mm

Maximum crack width (exp.7.8); wk = sr.max × max([s – kt × (fct.eff / p.eff) ×(1 + e × p.eff)] / Es,   
0.6 × s / Es) = **0.006** mm

PASS - Maximum crack width is less than limiting crack width

Library item: Crack width output

**Punching shear (Section 6.4)**

Strength reduction factor (exp 6.6N); v = 0.6  [1 - fck / 250 N/mm2] = **0.504**

Average depth to reinforcement; d = **1713** mm

Maximum punching shear resistance (cl.6.4.5(3)); vRd.max = 0.5  v  fcd = **5.712** N/mm2

k = min(1 + √(200 mm / d), 2) = **1.342**

Longitudinal reinforcement ratio (cl.6.4.4(1)); lx = Asx.bot.prov / (Ly  d) = **0.002**

ly = Asy.bot.prov / (Lx  d) = **0.002**

l = min(√(lx × ly), 0.02) = **0.002**

CRd,c = 0.18 / C =**0.120**

vmin = 0.035 N1/2/mm  k3/2  fck0.5 = **0.344** N/mm2

Design punching shear resistance (exp.6.47); vRd.c = max(CRd.c  k  (100 N2/mm4  l  fck)1/3, vmin) = **0.344** N/mm2

Design punching shear resistance at 1d (exp. 6.50); vRd.c1 = (2  d / d)  vRd.c = **0.688** N/mm2

Library item: Punching resistance output

Column No.1 - Punching shear perimeter at column face

Punching shear perimeter; u0 = **1200** mm

Area within punching shear perimeter; A0 = **0.090** m2

Library item: Shear perimeter output

Maximum punching shear force; VEd.max = **502** kN

Punching shear stress factor (fig 6.21N);  = **1.500**

Maximum punching shear stress (exp 6.38); vEd.max =   VEd.max / (u0  d) = **0.366** N/mm2

PASS - Maximum punching shear resistance exceeds maximum punching shear stress

Library item: Punching stress beta output



Library item: Show foundation sketch

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EN Stair design

RC stair design (EN 1992)

In accordance with EN1992-1-1:2004 incorporating Corrigenda January 2008 and the UK national annex

Tedds calculation version 1.0.07

Design summary

| **Description** | **Unit** | **Provided** | **Required** | **Utilisation** | **Result** |
| --- | --- | --- | --- | --- | --- |
| Bottom long. reinf. -Mid Span | mm2 | 679 | 556(crk) | 0.82 | PASS |
| Bottom long. reinf. -Upper land. | mm2 | 471 | 310 | 0.66 | PASS |
| Bottom long. reinf. -Lower land. | mm2 | 471 | 310 | 0.66 | PASS |
|  |  | **Allowable** | **Actual** | **Utilisation** | **Result** |
| Span-to-depth ratio |  | 40.00 | 23.96 | 0.60 | PASS |
| Shear capacity -Upper supp. | kN | 106.44 | 33.04 | 0.31 | PASS |
| Shear capacity -Lower supp. | kN | 106.44 | 33.04 | 0.31 | PASS |



Stair geometry

Number of risers; Nsteps = **10**

Going; G = **250** mm

Rise; R = **150** mm

Waist; hwaist = **200** mm

Breadth; b = **1000** mm

Length of the tread span; Lmid = (Nsteps – 1) × G = **2250** mm

Overall height of stairs; hstairs = Nsteps × R = **1500** mm

Angle of stairs; stairs = atan (R / G) = **30.96** 

Upper landing - Simple end support

Length of the upper landing; Lup = **1000** mm

Depth of the upper landing; hup = **200** mm

Width of the supporting element; wup = **200** mm

Distance to the centre of the support; ds,up = **100** mm

Lower landing - Simple end support

Length of the lower landing; Llow = **1000** mm

Depth of the lower landing; hlow = **200** mm

Width of the supporting element; wlow = **200** mm

Distance to the centre of the support; ds,low = **100** mm

Effective span – 5.3.2.2

Overall length; Ltotal = Llow + Lmid + Lup = **4250** mm

Clear distance between the supports faces; Ln = Ltotal - wup -wlow = **3850** mm

Effective span variable – cl.5.3.2.2(1); aup = min(0.5  hup, 0.5  wup) = **100** mm

Effective span variable – cl.5.3.2.2(1); alow = min(0.5  hlow, 0.5  wlow) = **100** mm

Length of the effective span – exp.5.8; Lspan = Ln + alow + aup = **4050** mm

Concrete details - Table 3.1.

Concrete strength class; C40/50

Characteristic compressive cylinder strength; fck = **40** N/mm2

Mean value of compressive cylinder strength; fcm =fck + 8 N/mm2 = **48.0** N/mm2

Mean value of axial tensile strength; fctm = 0.3 N/mm2  (fck/ 1 N/mm2)2/3 = **3.5** N/mm2

Secant modulus of elasticity of concrete; Ecm = 22 kN/mm2(fcm/10 N/mm2)0.3  AAF = **35220.5** N/mm2

Partial factor for concrete – Table 2.1N; C = **1.5**

Density of concrete; conc = **24.50** kN/m3

Compression chord coefficient - cl.6.2.3(3); cw = **1**

Compressive strength coefficient – cl.3.1.6(1); αcc = **0.85**

Design compr. concrete strength - exp.3.15; fcd = cc  fck / C = **22.7** N/mm2

Effective strength factor – exp.3.21;  = **1.00**

Effect. compr. zone height factor – exp.3.19;  = **0.80**

Ultimate strain - Table 3.1; cu2 = **0.0035**

Shortening strain - Table 3.1; cu3 = **0.0035**

k1 = **0.40**

k2 = 1.0  (0.6 + 0.0014 / cu2) = **1.00**

k3 = **0.40**

k4 = 1.0  (0.6 + 0.0014 / cu2) = **1.00**

Maximum aggregate size; hagg = **20** mm

Reinforcing steel details

Characteristic yield strength of reinforcement; fyk = **500** N/mm2

Partial factor for reinforcing steel - Table 2.1N; S = **1.15**

Design yield strength of reinforcement; fyd = fyk / S= **435** N/mm2

Loading details

Self weight slab; gself,slab = hwaist / cos(stairs) × (conc) × b = **5.7** kN/m

Self weight steps; gself,steps = R / 2 × (conc) × b = **1.8** kN/m

Average self weight; gself,aver = gself,slab + gself,steps = **7.6** kN/m

Loading from finishes; gfin = **1.2** kN/m2

Imposed variable load; qk = **3.0** kN/m2

Permanent action factor; G = **1.35**

Imposed action factor; Q = **1.50**

Quasi-permanent value of variable action; 2 = **0.30**

Design load; FEd = G  (gself,aver + gfin  b) + Q  qk  b = **16.3** kN /m

Quasi-permanent load; FQP = 1.0  (gself,aver + gfin  b) + 2  qk  b = **9.7** kN /m

Midspan design

 = 0.125

Design moment; M = abs(  FEd  Lspan2) = **33.5** kNm

Nominal cover to reinforcement; cnom = **25** mm

Diameter of bar for long. direction; l =**12** mm

Depth of reinforcement; d = h - cnom - l / 2 = **169** mm

Redistribution ratio ;  = **1.00**

K coefficient; K = M / (b  d2  fck) = **0.029**

K' coefficient; K' = (2    cc / C)  (1 -   ( - k1) / (2  k2))  (  ( - k1) / (2  k2)) = **0.207**

K < K' -Compression reinforcement is not required

Lever arm; z = min(0.5  d  (1 + (1 - 2  Kz / (  cc / C))0.5), 0.95  d)

z = **161** mm

Tension area required in longitudinal direction; As,req = M / (fyd  z) = **479** mm2

Minimum area of reinforcement – exp.9.1.N; As,min = max(0.26  fctm / fyk  b  d, 0.0013  b  d)= **308** mm2

Maximum area of reinforcement - cl.9.2.1.1(3); As,max = 0.04  b  h = **8000** mm2

Tension reinforcement check

Diameter of bars for longitudinal direction; l = **12** mm

Number of bars in longitudinal direction; N = **6**

Diameter of bars for transverse direction; t = **12** mm

Bar spacing in transverse direction; s = **250** mm

Tension area provided in longitudinal direction; As,prov = N    l2 /4 = **679** mm2

PASS - Tension reinforcement area is greater than area required

Tension secondary area required in transverse direction – cl.9.3.1.1(2);

As,req,t = 0.2  max(As,prov, As,req) = **136** mm2

Tension area provided in transverse direction; As,prov,t =   t2 /4  (b / s) = **452** mm2

PASS - Tension reinforcement in transverse direction is greater than area required

Minimum bar spacing - Section 8.2

Bar spacing (clear distance) in long. direction; sbar,l = (b - (2  cnom + N  l)) / (N - 1) = **176** mm

kspc1 = **1**

kspc2 = **5** mm

Minimum allowable bar spacing – cl.8.2(2); smin,l = max(kspc1  l, hagg + kspc2, 20 mm) = **25** mm

PASS - Actual bar spacing exceeds minimum allowable

Bar spacing (clear distance) in transv. direction; sbar,t = s - t = **238** mm

Minimum allowable bar spacing – cl.8.2(2); smin,t = max(kspc1  t, hagg + kspc2, 20 mm) = **25** mm

PASS - Actual bar spacing exceeds minimum allowable

Crack control - Section 7.3

Maximum crack width; wk = **0.3** mm

Design value modulus of elasticity reinf – 3.2.7(4); Es = **200000** N/mm2

Mean value of concrete tensile strength; fct,eff = fctm = **3.5** N/mm2

Stress distribution coefficient; kc = **0.4**

Non-uniform self-equilibrating stress coefficient; k =min(max(1 + (300 mm - min(h, b))  0.35 / 500 mm, 0.65), 1) = **1.00**

Actual tension bar spacing; sbar = (b -(2  (cnom +  / 2))) / (N - 1) = **188** mm

Maximum stress permitted - Table 7.3N; s = **250** N/mm2

Steel to concrete modulus of elast. ratio; cr = Es / Ecm= **5.68**

Distance of the Elastic NA from tension face; y = (b  h2 / 2 + As,prov  (cr - 1)  (h - d)) / (b  h + As,prov  (cr - 1))

y = **99** mm

Area of concrete in the tensile zone; Act = b  y = **98922** mm2

Minimum area of reinforcement required - exp.7.1; Asc,min = kc  k  fct,eff  Act / s = **556** mm2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent moment; MQP = abs(  FQP  Lspan2) = **19.8**kNm

Permanent load ratio; RPL = MQP / M = **0.59**

Service stress in reinforcement; sr = fyd  As,req / As,prov  RPL = **182** N/mm2

Maximum bar spacing - Tables 7.3N; sbar,max = **272.9** mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Deflection control - Section 7.4

Reference reinforcement ratio; 0 = (fck / 1 N/mm2)0.5 / 1000 = **0.00632**

Required tension reinforcement ratio;  = As,req / (b  dmid) = **0.00284**

Required compression reinforcement ratio; ' = A's,req / (b  dmid) = **0.00000**

Structural system factor - Table 7.4N; Kb = **1.00**

Basic allow. span to depth ratio -**exp.7.16(a)**;

stdbasic = Kb  [11 + 1.5  (fck / 1 N/mm2)0.5  0 /  +3.2  (fck / 1 N/mm2)0.5  (0 /  - 1)3/2] = **59.783**

Reinforcement factor - exp.7.17; Ks = min(As,prov / As,req  500 N/mm2 / fyk, 1.5) = **1.416**

Long span supp. brittle partition factor-cl.7.4.2(2); F1 = **1.00**

Allowable span to depth ratio; stdallow = min(stdbasic  Ks  F1, 40  Kb) = **40.000**

Actual span to depth ratio; stdactual = Lspan / dmid = **23.964**

PASS-Span to effective depth is less than the maximum allowable

Upper landing support

Design moment at support; M = **0.0** kNm

Nominal cover to reinforcement; cnom = **25** mm

Diameter of bar for long. direction; l =**10** mm

Depth of reinforcement; d = h - cnom - l / 2 = **170** mm

Calculated reinforcement at midspan cl. 9.3.1.2(1); As,span = Mmid / (fyd  zmid) = **479** mm2

Minimum area of reinforcement – exp.9.1.N; As,min = max(0.26  fctm / fyk  b  d,0.0013  b  d, 0.5  As,span)

As,min = **310** mm2

Maximum area of reinforcement - cl.9.2.1.1(3); As,max = 0.04  b  h = **8000** mm2

Tension reinforcement check

Diameter of bars for longitudinal direction; l = **10** mm

Number of bars in longitudinal direction; N = **6**

Diameter of bars for transverse direction; t = **10** mm

Bar spacing in transverse direction; s = **300** mm

Tension area provided in longitudinal direction; As,prov = N    l2 /4 = **471** mm2

PASS - Tension reinforcement area is greater than area required

Tension secondary area required in transverse direction – cl.9.3.1.1(2);

As,req,t = 0.2  max(As,prov, As,req) = **94** mm2

Tension area provided in transverse direction; As,prov,t =   t2 /4  (b / s) = **262** mm2

PASS - Tension reinforcement in transverse direction is greater than area required

Minimum bar spacing - Section 8.2

Bar spacing (clear distance) in long. direction; sbar,l = (b - (2  cnom + N  l)) / (N - 1) = **178** mm

kspc1 = **1**

kspc2 = **5** mm

Minimum allowable bar spacing – cl.8.2(2); smin,l = max(kspc1  l, hagg + kspc2, 20 mm) = **25** mm

PASS - Actual bar spacing exceeds minimum allowable

Bar spacing (clear distance) in transv. direction; sbar,t = s - t = **290** mm

Minimum allowable bar spacing – cl.8.2(2); smin,t = max(kspc1  t, hagg + kspc2, 20 mm) = **25** mm

PASS - Actual bar spacing exceeds minimum allowable

Shear capacity check - Section 6.2

Shear coefficient;  = 0.500

Design shear force; V =   FEd  Lspan = **33.0** kN

Effective depth; d = **170** mm

Shear stress; v = V / (b  d) = **0.19** N/mm2

Design shear resist. coefficient. – cl.6.2.2(1); CRd,c = 0.18 / C = **0.120**

Tension reinforcement provided; Asl = N    l2 /4= **471** mm2

Design shear resist. size factor. – cl.6.2.2(1); Ksh = min(1 + (200 mm / d)1/2, 2) = **2.00**

Design shear resist. factor. – cl.6.2.2(1); ρ1 = Asl / (b  d) = **0.00277**

Design shear resist. factor. – exp.6.3.N; vmin = (0.035 (N)1/2 / mm)  Ksh 3 / 2  fck1/2= **0.63** N/mm2

Minimum design shear resistance – exp.6.2(b); VRd,cmin = (vmin)  b  d = **106.4** kN

Design shear resist. without reinf. – exp.6.2(a); VRd,c1 = (CRd,c  Ksh  (100 N2 / mm4  1  fck)1/3) b  d

VRd,c1 = **91.0** kN

VRd,c = max(VRd,c1, VRd,cmin) = **106.4** kN

PASS - Design force is less than maximum allowable- No shear reinforcement is required;

Lower landing support

Design moment at support; M = **0.0** kNm

Nominal cover to reinforcement; cnom = **25** mm

Diameter of bar for long. direction; l =**10** mm

Depth of reinforcement; d = h - cnom - l / 2 = **170** mm

Calculated reinforcement at midspan cl. 9.3.1.2(1); As,span = Mmid / (fyd  zmid) = **479** mm2

Minimum area of reinforcement – exp.9.1.N; As,min = max(0.26  fctm / fyk  b  d,0.0013  b  d, 0.5  As,span)

As,min = **310** mm2

Maximum area of reinforcement - cl.9.2.1.1(3); As,max = 0.04  b  h = **8000** mm2

Tension reinforcement check

Diameter of bars for longitudinal direction; l = **10** mm

Number of bars in longitudinal direction; N = **6**

Diameter of bars for transverse direction; t = **10** mm

Bar spacing in transverse direction; s = **300** mm

Tension area provided in longitudinal direction; As,prov = N    l2 /4 = **471** mm2

PASS - Tension reinforcement area is greater than area required

Tension secondary area required in transverse direction – cl.9.3.1.1(2);

As,req,t = 0.2  max(As,prov, As,req) = **94** mm2

Tension area provided in transverse direction; As,prov,t =   t2 /4  (b / s) = **262** mm2

PASS - Tension reinforcement in transverse direction is greater than area required

Minimum bar spacing - Section 8.2

Bar spacing (clear distance) in long. direction; sbar,l = (b - (2  cnom + N  l)) / (N - 1) = **178** mm

kspc1 = **1**

kspc2 = **5** mm

Minimum allowable bar spacing – cl.8.2(2); smin,l = max(kspc1  l, hagg + kspc2, 20 mm) = **25** mm

PASS - Actual bar spacing exceeds minimum allowable

Bar spacing (clear distance) in transv. direction; sbar,t = s - t = **290** mm

Minimum allowable bar spacing – cl.8.2(2); smin,t = max(kspc1  t, hagg + kspc2, 20 mm) = **25** mm

PASS - Actual bar spacing exceeds minimum allowable

Shear capacity check - Section 6.2

Shear coefficient;  = 0.500

Design shear force; V =   FEd  Lspan = **33.0** kN

Effective depth; d = **170** mm

Shear stress; v = V / (b  d) = **0.19** N/mm2

Design shear resist. coefficient. – cl.6.2.2(1); CRd,c = 0.18 / C = **0.120**

Tension reinforcement provided; Asl = N    l2 /4= **471** mm2

Design shear resist. size factor. – cl.6.2.2(1); Ksh = min(1 + (200 mm / d)1/2, 2) = **2.00**

Design shear resist. factor. – cl.6.2.2(1); ρ1 = Asl / (b  d) = **0.00277**

Design shear resist. factor. – exp.6.3.N; vmin = (0.035 (N)1/2 / mm)  Ksh 3 / 2  fck1/2= **0.63** N/mm2

Minimum design shear resistance – exp.6.2(b); VRd,cmin = (vmin)  b  d = **106.4** kN

Design shear resist. without reinf. – exp.6.2(a); VRd,c1 = (CRd,c  Ksh  (100 N2 / mm4  1  fck)1/3) b  d

VRd,c1 = **91.0** kN

VRd,c = max(VRd,c1, VRd,cmin) = **106.4** kN

PASS - Design force is less than maximum allowable- No shear reinforcement is required;



;

;

EN Wall design

RC wall design (EN 1992)

In accordance with EN1992-1-1:2004 incorporating corrigendum January 2008 and the UK national annex

Tedds calculation version 1.1.06

Design summary

|  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- |
| **Description** | **Unit** | **Allowable** | **Actua** | **Utilisation** | **Result** |
| Moment capacity | kNm/m | 167.51 | 32.63 | 0.19 | PASS |
| Crack width | mm | 1.00 | 0.08 | 0.08 | PASS |



Wall input details

Wall geometry

Thickness; h = **300** mm

Length; b = **1000** mm/m

Clear height between restraints; l = **3000** mm

Stability about minor axis; **Unbraced**

Concrete details

Concrete strength class; **C40/50**

Partial safety factor for concrete (2.4.2.4(1)); C = **1.50**

Coefficient cc (3.1.6(1)); cc = **0.85**

Maximum aggregate size; dg = **20** mm

Reinforcement details

Reinforcement in outer layer; **Vertical**

Nominal cover to outer layer; cnom = **35** mm

Vertical bar diameter; v = **16** mm

Spacing of vertical reinforcement; sv = **200** mm

Area of vertical reinforcement (per face); Asv = **1005** mm2/m

Horizontal bar diameter; h = **10** mm

Spacing of horizontal reinforcement; sh = **250** mm

Area of horizontal reinforcement (per face); Ash = **314** mm2/m

Characteristic yield strength; fyk = **500** N/mm2

Partial safety factor for reinft (2.4.2.4(1)); S = **1.15**

Es = 200000 MPa

Modulus of elasticity of reinft (3.2.7(4)); Es = **200.0** kN/mm2

Fire resistance details

Fire resistance period; R = **60** min

Exposure to fire; **Exposed on one side**

Ratio of fire design axial load to design resistance; fi = **0.70**

Axial load and bending moments from frame analysis

Design axial load; NEd = **500.0** kN/m

Moment about minor axis at top; Mtop = **20.0** kNm/m

Moment about minor axis at bottom; Mbtm = **30.0** kNm/m

Wall effective length factor

Effective length factor for buckling about minor axis; f = **0.70**

Crack width details

Axial load due to quasi-permanent SLS.; NEd\_SLS = **325.0** kN/m

Moment at top due to quasi-permanent SLS.; Mtop\_SLS = **13.0** kNm/m

Moment at btm due to quasi-permanent SLS.; Mbtm\_SLS = **19.5** kNm/m

Duration of applied loading; **Long term**

Maximum allowable crack width; wk\_max = **1.0** mm

Calculated wall properties

Concrete properties

Area of concrete; Ac = h × b = **300000** mm2/m

Characteristic compression cylinder strength; fck = **40** N/mm2

Design compressive strength (3.1.6(1)); fcd = cc × fck / C = **22.7** N/mm2

Mean value of cylinder strength (Table 3.1); fcm = fck + 8 MPa = **48.0** N/mm2

fctm = if(fck<=50 MPa,0.3 MPa×(fck/1 MPa)2/3,2.12 MPa×ln(1+(fcm/10 MPa))) = **3.51** N/mm2

Mean value of tensile strength; fctm = **3.51** N/mm2

Secant modulus of elasticity (Table 3.1); Ecm = 22000 MPa × (fcm / 10 MPa)0.3 = **35.2** kN/mm2

Rectangular stress block factors

Depth factor (3.1.7(3)); sb = 0.8

Stress factor (3.1.7(3));  = 1.0

Strain limits

cu3 = 0.0035

Compression strain limit (Table 3.1); cu3 = **0.00350**

c3 = 0.00175

Pure compression strain limit (Table 3.1); c3 = **0.00175**

Design yield strength of reinforcement

Design yield strength (3.2.7(2)); fyd = fyk / S = **434.8** MPa

Check nominal cover for fire and bond requirements

Min. cover reqd for bond (4.4.1.2(3)); Cmin,b = max(v, h - v) = **16** mm

Min axis distance for fire (EN1992-1-2 T 5.4); afi = **10** mm

Allowance for deviations from min cover (4.4.1.3); cdev = **5** mm

cnom\_min = if(\_rcc.Layer==1,max(afi - v / 2, cmin,b + cdev), max(afi - h / 2, cmin,b + cdev)) = **21.0** mm

Min allowable nominal cover; cnom\_min = max(afi - v / 2, cmin,b + cdev) = ;**21**; mm

PASS - the nominal cover is greater than the minimum required

Effective depth of vertical bars

Effective depth; d = h - cnom - v / 2 = **257** mm

Depth to compression face bars; d’ = cnom + v / 2 = **43** mm

Wall effective length

Wall effective length; l0 = f × l = **2100** mm

Column slenderness

Radius of gyration about minor axis; i = h / √(12) = **8.7** cm

Minor axis slenderness ratio (5.8.3.2(1));  = l0 / i = **24.2**

Design bending moments

Frame analysis moments combined with moments due to imperfections (cl. 5.2 & 6.1(4))

Ecc. due to geometric imperfections; ei = l0 /400 = **5.2** mm

Minimum end moment about minor axis; M01 = min(abs(Mtop), abs(Mbtm)) + ei × NEd = **22.6** kNm/m

Maximum end moment about minor axis; M02 = max(abs(Mtop), abs(Mbtm)) + ei × NEd = **32.6** kNm/m

Slenderness limit for buckling about minor axis (cl. 5.8.3.1)

A = 0.7

Factor A; A = **0.7**

Mechanical reinforcement ratio;  = 2 × Asv × fyd / (Ac × fcd) = **0.129**

Factor B; B = √(1 + 2 × ) = **1.121**

rm = if(and(\_rcc.Braced == 1, max(abs(Mtop),abs(Mbtm))>=NEd × ei) , if(or(and(Mtop>=0 kNm/m,Mbtm>=0 kNm/m), and(Mtop<=0 kNm/m,Mbtm<=0 kNm/m)),M01 / M02 ,-1.0 × M01 / M02), 1.0) = **1.000**

Moment ratio; ××rm = ;**1.000**

Factor C; C = 1.7 - rm = **0.700**

Relative normal force; n = NEd / (Ac × fcd) = **0.074**

Slenderness limit; lim = 20 × A × B × C / √(n) = **40.5**

<lim - Second order effects may be ignored

Design bending moment

Design moment about minor axis; MEd = max(M02, NEd  max(h/30, 20 mm)) = **32.6** kNm/m

Moment capacity about minor axis with axial load NEd

Moment of resistance of concrete

By iteration:-

Position of neutral axis; z = **47.7** mm

Concrete compression force (3.1.7(3)); Fc = × fcd × min(max(sb × z, 0 mm) , h) × b = **865.4** kN/m

Moment of resistance; MRdc = Fc × [h / 2 - (min(sb × z , h)) / 2] = **113.3** kNm/m

Moment of resistance of reinforcement

Strain in tension face bars;  = cu3 × (1 - d / z) = **-0.01535**

Stress in tension face bars;  = if(< 0, max(-1×fyd, Es × ), min(fyd, Es × )) = **-434.8** N/mm2

Force in tension face bars; Fs = if(d > sb × z, Asv × , Asv × ( -  × fcd)) = **-437.1** kN/m

Strain in compression face bars; ’ = cu3 × (1 - d’ / z) = **0.00035**

Stress in compression face bars; ’ = if(’< 0, max(-1×fyd, Es × ’), min(fyd, Es × ’)) = **69.3** N/mm2

Force in compression face bars; Fs’ = if(d’ > sb × z, Asv × ’, Asv × (’ -  × fcd)) = **69.7** kN/m

Resultant concrete/steel force; F = Fc + Fs + Fs’ = **498.0** kN/m

PASS - This is within half of one percent of the applied axial load therefore say OK

Moment of resistance of tension face bars; MRds = Fs × (d - h / 2) = **-46.8** kNm/m

Moment of resistance of compression face bars; MRds’ = Fs’ × (h / 2 - d’) = **7.5** kNm/m

Combined moment of resistance

Moment of resistance about minor axis; MRd = MRdc + MRds’ - MRds = **167.5** kNm/m

PASS - The moment capacity exceeds the design bending moment

Crack widths

Slenderness limit (cl. 5.8.3.1)

Min 1st order moment about minor axis; M01\_SLS=min(abs(Mtop\_SLS),abs(Mbtm\_SLS))+ei×NEd\_SLS = **14.7** kNm/m

Max 1st order moment about minor axis; M02\_SLS=max(abs(Mtop\_SLS),abs(Mbtm\_SLS))+ei×NEd\_SLS = **21.2** kNm/m

rm\_SLS = if(and(\_rcc.Braced == 1, max(abs(Mtop\_SLS),abs(Mbtm\_SLS))>=NEd\_SLS × ei) , if(or(and(Mtop\_SLS>=0 kNm/m,Mbtm\_SLS>=0 kNm/m), and(Mtop\_SLS<=0 kNm/m,Mbtm\_SLS<=0 kNm/m)),M01\_SLS / M02\_SLS ,-1.0 × M01\_SLS / M02\_SLS), 1.0) = **1.000**

Moment ratio; ××rm\_SLS = ;**1.000**

Factor C; CSLS = 1.7 - rm\_SLS = **0.700**

Relative normal force; nSLS = NEd\_SLS / (Ac × fcd) = **0.048**

Slenderness limit; lim\_SLS = 20 × A × B × CSLS / √(nSLS) = **50.3**

<lim\_SLS - Second order effects may be ignored

Design bending moment (cl. 7.3.4)

Design moment about minor axis; MEd\_SLS = M02\_SLS = **21.2** kNm/m

Cover to tension reinforcement; c = h - d - v / 2 = **35.0** mm

Ratio of steel to concrete modulii; e = Es / Ecm = **5.7**

Area of reinft in concrete units; As,eff = 2 × e × Asv = **11417** mm2/m

Combined area of steel/conc in conc units; Aeff = b × h + As,eff = **311417** mm2/m

Reinforcement ratio per face;  = Asv /(b × d) = **0.004**

Neutral axis depth with pure bending; xb = d × [-2×e× + √(4×e2×2 + 2×e××(1+d’/d))] = **48.2** mm

Second moment of area of cracked section; Ic = b×xb3/3 + e××b×d×[(xb-d’)2 + (d-xb)2] = **286364393** mm4/m

Strain in tension face steel due to bending; sb = MEd\_SLS × (xb - d) / (Ecm × Ic) = **-0.00044**

Strain in comp face steel due to bending; sb’ = MEd\_SLS × (xb - d’) / (Ecm × Ic) = **0.00001**

Strain due to axial load; axial = NEd\_SLS / (Aeff × Ecm) = **0.00003**

Resultant strain in tension face steel; s = sb + axial = **-0.00041**

Resultant strain in comp face steel; s’ = sb’ + axial = **0.00004**

Stress in tension steel; s = min(fyd, abs(Es × s)) = **81.9** MPa

Depth to neutral axis; x = [(s’ × d) - (s × d’)] / (s’ - s) = **62.3** mm

Effective depth of concrete in tension; hc,ef = min(2.5(h-d), (h-x)/3, h/2) = **79.2** mm

Effective area of concrete in tension; Ac,eff = hc,ef × b = **79232** mm2/m

kt = if(\_rcc.Duration==1,0.6,0.4) = **0.4**

Load duration factor; kt = **0.4**

Reinforcement ratio; p,eff = Asv / Ac,eff = **0.013**

Mean value of conc tensile strength; fct,eff = fctm = **3.51** MPa

Difference between reinft and concrete strains; diff = max([s-kt×fct,eff×(1+e×p,eff)/p,eff]/Es, 0.6×s/Es) = **0.00025**

Greater tensile strain; 1 = s × (h - x) / (d - x) = **-0.00050**

Lesser tensile strain; 2 = min(0, s’ × x / (x - d’)) = **0.00000**

k1cs = 0.8

Factor k1; k1cs = **0.8**

Factor k2; k2cs = (1 + 2) / (2 × 1) = **0.500**

Factor k3; k3cs = 3.40= **3.4**

Factor k4; k4cs = **0.425**

Maximum crack spacing; sr,max = k3cs × c + k1cs × k2cs × k4cs × v / p,eff = **333.4** mm

Crack width; wk = sr,max × diff = **0.082** mm

Allowable crack width; wk\_max = **1.0** mm

PASS - The maximum crack width is less than the maximum allowable

;