# BS FLOOR SLAB A

## RC SLAB DESIGN (BS8110:PART1:1997)

TEDDS calculation version 1.0.04

## TWO WAY SPANNING SLAB DEFINITION - RESTRAINED

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

Outer sagging steel

; Cover to outer tension reinforcement resisting sagging; $c_{sag}$ = **20** mm

; Trial bar diameter; $D_{tryx}$ = **16** mm

Depth to outer tension steel (resisting sagging)

$d_x = h - c_{sag} - D_{tryx}/2 = \mathbf{372}$ mm

Inner sagging steel

; Trial bar diameter; $D_{tryy}$ = **16** mm

Depth to inner tension steel (resisting sagging)

$d_y = h - c_{sag} - D_{tryx} - D_{tryy}/2 = \mathbf{356}$ mm

Outer hogging steel

; Cover to outer tension reinforcement resisting hogging; $c_{hog}$ = **20** mm

; Trial bar diameter; $D_{tryxhog}$ = **16** mm

Depth to outer tension steel (resisting hogging)

$d_{xhog} = h - c_{hog} - D_{tryxhog}/2 = \mathbf{372}$ mm

Inner hogging steel

; Trial bar diameter; $D_{tryyhog}$ = **16** mm

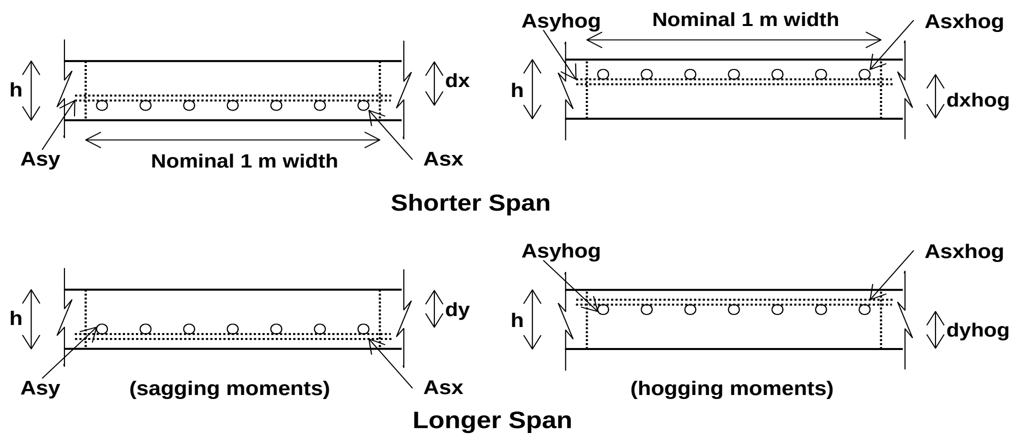
Depth to inner tension steel (resisting hogging)

$d_{yhog} = h - c_{hog} - D_{tryxhog} - D_{tryyhog}/2 = \mathbf{356}$ mm

Materials

; Characteristic strength of reinforcement; $f_y$ = **500** N/mm$^2$

; Characteristic strength of concrete; $f_{cu}$ = **40** N/mm$^2$




**Two-way spanning slab**

**(restrained)**

## RESTRAINED – 2 WAY SPANNING (CL 3.5.3)

## MAXIMUM DESIGN MOMENTS

; Length of shorter side of slab; $l_x$ = **5.000** m

; Length of longer side of slab; $l_y$ = **5.000** m

; Design ultimate load per unit area; $n_s$ = **15.0** kN/m$^2$

; Edge condition shorter side (1); Edge$_1$ = **"C"**

; Edge condition other shorter side (2); Edge$_2$ = **"D"**

; Edge condition longer side (3); Edge$_3$ = **"C"**

; Edge condition other longer side (4); Edge$_4$ = **"D"**

Number of discontinuous edges

$N_d$ = **2**

Moment coefficients

$\beta_{sy} = (24 + 2\times N_d + 1.5\times N_d^2) / 1000 =$ **0.034**

$\beta_1$ = if(Edge$_1$ == "C", $4/3\times\beta_{sy}$,0) = **0.045**

$\beta_2$ = if(Edge$_2$ == "C", $4/3\times\beta_{sy}$,0) = **0.000**

$\gamma = 2/9 \times [3 - \sqrt{(18)} \times l_x/l_y \times (\sqrt{(\beta_{sy} + \beta_1)} + \sqrt{(\beta_{sy} + \beta_2)})]$ = **0.227**

$\beta_{3x}$ = if(Edge$_3$ == "C", 4/3,0) = **1.333**

$\beta_{4x}$ = if(Edge$_4$ == "C", 4/3,0) = **0.000**

$\beta_{sx} = \gamma / [(1+\beta_{3x})^{0.5} + (1+\beta_{4x})^{0.5}]^2$ = **0.036**

$\beta_3 = \beta_{3x} \times \beta_{sx}$ = **0.047**

$\beta_4 = \beta_{4x} \times \beta_{sx}$ = **0.000**

Maximum span moments per unit width - restrained slabs

$m_{sx} = \beta_{sx} \times n_s \times l_x^2$ = **13.3** kNm/m

$m_{sy} = \beta_{sy} \times n_s \times l_x^2$ =**12.8** kNm/m

Maximum support moments per unit width - restrained slabs

$m_{sxhog} = \max(\beta_3, \beta_4) \times n_s \times l_x^2$ = **17.8** kNm/m

$m_{syhog} = \max(\beta_1, \beta_2) \times n_s \times l_x^2$ =**17.0** kNm/m

**CONCRETE SLAB DESIGN – SAGGING – OUTER LAYER OF STEEL (CL 3.5.4)**

; Design sagging moment (per m width of slab); $m_{sx}$ = **13.3** kNm/m

; Moment Redistribution Factor; $\beta_{bx}$ = **1.0**

**Area of reinforcement required**

;; $K_x = \text{abs}(m_{sx}) / (d_x^2 \times f_{cu})$ = **0.002**

$K'_x = \min(0.156, (0.402 \times (\beta_{bx} - 0.4)) - (0.18 \times (\beta_{bx} - 0.4)^2))$ = **0.156**

***Outer compression steel not required to resist sagging***

***Slab requiring outer tension steel only - bars (sagging)***

;; $z_x = \min((0.95 \times d_x),(d_x \times (0.5+\sqrt{(0.25-K_x/0.9)})))$ = **353** mm

Neutral axis depth; $x_x = (d_x - z_x) / 0.45$ = **41** mm

Area of tension steel required

;;; $A_{sx\_req} = \text{abs}(m_{sx}) / (1/\gamma_{ms} \times f_y \times z_x)$ = **87** mm²/m

**Tension steel**

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

$A_{sx\_prov} = A_{sx}$ = **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); $m_{sy}$ = **12.8** kNm/m

; Moment Redistribution Factor; $\beta_{by}$ = **1.0**

**Area of reinforcement required**

;; $K_y = \text{abs}(m_{sy}) / (d_y^2 \times f_{cu})$ = **0.003**

$K'_y = \min(0.156, (0.402 \times (\beta_{by} - 0.4)) - (0.18 \times (\beta_{by} - 0.4)^2))$ = **0.156**

***Inner compression steel not required to resist sagging***

***Slab requiring inner tension steel only - bars (sagging)***

;; $z_y = \min((0.95 \times d_y),(d_y \times (0.5+\sqrt{(0.25-K_y/0.9)})))$ = **338** mm

Neutral axis depth; $x_y = (d_y - z_y) / 0.45$ = **40** mm

Area of tension steel required

;;; $A_{sy\_req} = \text{abs}(m_{sy}) / (1/\gamma_{ms} \times f_y \times z_y)$ = **87** mm²/m

**Tension steel**

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

$A_{sy\_prov} = A_{sy} = \mathbf{1050}$ mm$^2$/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); $m_{sxhog} = \mathbf{17.8}$ kNm/m

; Moment Redistribution Factor; $\beta_{bx} = \mathbf{1.0}$

**Area of reinforcement required**

;; $K_{xhog} = abs(m_{sxhog}) / ( d_{xhog}^2 \times f_{cu} ) = \mathbf{0.003}$

$K'_x = \min (0.156 , (0.402 \times (\beta_{bx} - 0.4)) - (0.18 \times (\beta_{bx} - 0.4)^2 )) = \mathbf{0.156}$

***Outer compression steel not required to resist hogging***

***Slab requiring outer tension steel only - bars (hogging)***

;; $z_{xhog} = \min (( 0.95 \times d_{xhog}),(d_{xhog}\times(0.5+\sqrt{(0.25-K_{xhog}/0.9)}))) = \mathbf{353}$ mm

Neutral axis depth; $x_{xhog} = (d_{xhog} - z_{xhog}) / 0.45 = \mathbf{41}$ mm

Area of tension steel required

;;; $A_{sxhog\_req} = abs(m_{sxhog}) / (1/\gamma_{ms} \times f_y \times z_{xhog}) = \mathbf{116}$ mm$^2$/m

**Tension steel**

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

$A_{sxhog\_prov} = A_{sxhog} = \mathbf{1050}$ mm$^2$/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); $m_{syhog} = \mathbf{17.0}$ kNm/m

; Moment Redistribution Factor; $\beta_{by} = \mathbf{1.0}$

**Area of reinforcement required**

;; $K_{yhog} = abs(m_{syhog}) / ( d_{yhog}^2 \times f_{cu} ) = \mathbf{0.003}$

$K'_y = \min (0.156 , (0.402 \times (\beta_{by} - 0.4)) - (0.18 \times (\beta_{by} - 0.4)^2 )) = \mathbf{0.156}$

***Inner compression steel not required to resist hogging***

***Slab requiring inner tension steel only - bars (hogging)***

;; $z_{yhog} = \min (( 0.95 \times d_{yhog}),(d_{yhog}\times(0.5+\sqrt{(0.25-K_{yhog}/0.9)}))) = \mathbf{338}$ mm

Neutral axis depth; $x_{yhog} = (d_{yhog} - z_{yhog}) / 0.45 = \mathbf{40}$ mm

Area of tension steel required

;;; $A_{syhog\_req} = abs(m_{syhog}) / (1/\gamma_{ms} \times f_y \times z_{yhog}) = \mathbf{116}$ mm$^2$/m

**Tension steel**

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

$A_{syhog\_prov} = A_{syhog} = \mathbf{1050}$ mm$^2$/m

***Area of inner tension steel provided sufficient to resist hogging***

***Check min and max areas of steel resisting sagging***

;Total area of concrete; $A_c = h = \mathbf{400000}$ mm$^2$/m

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

$A_{st\_min} = k \times A_c$ = **520** $mm^2/m$

$A_{st\_max} = 4\ \% \times A_c$ = **16000** $mm^2/m$

Steel defined:

; Outer steel resisting sagging; $A_{sx\_prov}$ = **1050** $mm^2/m$

***Area of outer steel provided (sagging) OK***

; Inner steel resisting sagging; $A_{sy\_prov}$ = **1050** $mm^2/m$

***Area of inner steel provided (sagging) OK***

***Check min and max areas of steel resisting hogging***

;Total area of concrete; $A_c = h$ = **400000** $mm^2/m$

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

$A_{st\_min} = k \times A_c$ = **520** $mm^2/m$

$A_{st\_max} = 4\ \% \times A_c$ = **16000** $mm^2/m$

Steel defined:

; Outer steel resisting hogging; $A_{sxhog\_prov}$ = **1050** $mm^2/m$

***Area of outer steel provided (hogging) OK***

; Inner steel resisting hogging ; $A_{syhog\_prov}$ = **1050** $mm^2/m$

***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; $d_x$ = **372** mm

; Area of tension reinforcement provided (per m width of slab); $A_{sx\_prov}$ = **1050** $mm^2/m$

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

; Characteristic strength of concrete; $f_{cu}$ = **40** $N/mm^2$

**Applied shear stress**

$v_x = V_x / d_x$ = **0.07** $N/mm^2$

**Check shear stress to clause 3.5.5.2**

$v_{allowable} = \min((0.8\ N^{1/2}/mm) \times \sqrt{(f_{cu})}, 5\ N/mm^2)$ = **5.00** $N/mm^2$

***Shear stress - OK***

**Shear stresses to clause 3.5.5.3**

**Design shear stress**

$f_{cu\_ratio}$ = if ($f_{cu}$ > 40 $N/mm^2$ , 40/25 , $f_{cu}$/(25 $N/mm^2$)) = **1.600**

$v_{cx} = 0.79\ N/mm^2 \times \min(3{,}100 \times A_{sx\_prov} / d_x)^{1/3} \times \max(0.67,(400\ mm / d_x)^{1/4}) / 1.25 \times f_{cu\_ratio}^{1/3}$

$v_{cx}$ = **0.49** $N/mm^2$

Applied shear stress

$v_x$ = **0.07** $N/mm^2$

***No shear reinforcement required***

**SHEAR RESISTANCE OF CONCRETE SLABS (CL 3.5.5)**

**Inner tension steel resisting sagging moments**

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

; Area of tension reinforcement provided (per m width of slab); $A_{sy\_prov}$ = **1050** mm$^2$/m

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

; Characteristic strength of concrete; $f_{cu}$ = **40** N/mm$^2$

**Applied shear stress**

$v_y = V_y / d_y =$ **0.06** N/mm$^2$

**Check shear stress to clause 3.5.5.2**

$v_{allowable} = \min((0.8\ N^{1/2}/mm) \times \sqrt{(f_{cu})}, 5\ N/mm^2) =$ **5.00** N/mm$^2$

***Shear stress - OK***

**Shear stresses to clause 3.5.5.3**

**Design shear stress**

$f_{cu\_ratio} = \text{if}\ (f_{cu} > 40\ N/mm^2, 40/25, f_{cu}/(25\ N/mm^2)) =$ **1.600**

$v_{cy} = 0.79\ N/mm^2 \times \min(3,100 \times A_{sy\_prov} / d_y)^{1/3} \times \max(0.67,(400\ mm) / d_y)^{1/4} / 1.25 \times f_{cu\_ratio}^{1/3}$

$v_{cy}$ = **0.51** N/mm$^2$

Applied shear stress

$v_y$ = **0.06** N/mm$^2$

***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; $d_x$ = **372** mm

; Dimension from edge of load to shear perimeter; $l_p = k_p \times d_x$ = **558** mm; where; $k_p$ = **1.50**

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

Total length of inner perimeter at edge of loaded area; $u_{0\_gen} = 2 \times (l + w)$ = **1200** mm

Total length of outer perimeter at $l_p$ from loaded area; $u_{gen} = 2 \times (l + w) + 8 \times l_p$ = **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; $u_0$ = **1200** mm

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

; Depth to outer steel; $d_x$ = **372** mm

; Depth to inner steel; $d_y$ = **356** mm

Average depth to "tension" steel; $d_{av} = (d_x + d_y)/2$ = **364.0** mm

; Area of outer steel per m effective through the perimeter; $A_{sx\_prov}$ = **1050** mm$^2$ /m

; Area of inner steel per m effective through the perimeter; $A_{sy\_prov}$ = **1050** mm$^2$ /m

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

; Characteristic strength of concrete; $f_{cu}$ = **40** N/mm$^2$

**Applied shear stress**

Stress around loaded area; $v_{max} = V_p / (u_0 \times d_{av})$ = **0.114** N/mm$^2$

Stress around perimeter; $v = V_p / (u \times d_{av})$ = **0.024** N/mm$^2$

**Check shear stress to clause 3.7.7.2**

$v_{allowable} = \min((0.8\ N^{1/2}/mm) \times \sqrt{(f_{cu})}, 5\ N/mm^2)$ = **5.000** N/mm$^2$

***Shear stress - OK***

**Shear stresses to clause 3.7.7.4**

**Design shear stress**

$f_{cu\_ratio}$ = if ($f_{cu}$ > 40 N/mm$^2$ , 40/25 , $f_{cu}$/(25 N/mm$^2$)) = **1.600**

; Effective steel area for shear strength determination:; $A_{s\_eff}$ =**201** mm$^2$/m;

$v_c = 0.79\ N/mm^2 \times \min(3, 100\times(A_{s\_eff} / d_{av}))^{1/3} \times \max(0.67, (400\ mm / d_{av})^{1/4}) / 1.25 \times f_{cu\_ratio}^{1/3}$

$v_c$ = **0.288** N/mm$^2$

***No shear reinforcement required***

<u>**CONCRETE SLAB DEFLECTION CHECK (CL 3.5.7)**</u>

; Slab span length; $l_x$ = **5.000** m

; Design ultimate moment in shorter span per m width; $m_{sx}$ = **13** kNm/m

; Depth to outer tension steel; $d_x$ = **372** mm

**Tension steel**

; Area of outer tension reinforcement provided; $A_{sx\_prov}$ = **1050** mm$^2$/m

; Area of tension reinforcement required; $A_{sx\_req}$ = **87** mm$^2$/m

; Moment Redistribution Factor; $\beta_{bx}$ = **1.00**

**Modification Factors**

;Basic span / effective depth ratio (Table 3.9); $ratio_{span\_depth}$ = **60**

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

;$f_s = 2 \times f_y \times A_{sx\_req} / (3 \times A_{sx\_prov} \times \beta_{bx})$ = **27.6** N/mm$^2$

$factor_{tens} = \min(2, 0.55 + (477\ N/mm^2 - f_s) / (120 \times (0.9\ N/mm^2 + m_{sx} / d_x^2)))$ = **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; $l_{max} = ratio_{span\_depth} \times factor_{tens} \times d_x$ = **44.64** m

**Check the actual beam span**

Actual span/depth ratio; $l_x / d_x$ = **13.44**

Span depth limit; $ratio_{span\_depth} \times factor_{tens}$ = **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; $d_x$ = **372.0** mm

; Diameter of tension reinforcement; $D_x$ = **20** mm

; Diameter of links; $L_{diax}$ = **0** mm

Cover to outer tension reinforcement

$c_{tenx} = h - d_x - D_x / 2$ = **18.0** mm

Nominal cover to links steel

$c_{nomx} = c_{tenx} - L_{diax}$ = **18.0** mm

Permissable minimum nominal cover to all reinforcement (Table 3.4)

; $c_{min}$ = **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; $d_{xhog}$ = **372.0** mm

; Diameter of tension reinforcement; $D_{xhog}$ = **20** mm

; Diameter of links; $L_{diaxhog}$ = **0** mm

Cover to outer tension reinforcement

$c_{tenxhog} = h - d_{xhog} - D_{xhog} / 2$ = **18.0** mm

Nominal cover to links steel

$c_{nomxhog} = c_{tenxhog} - L_{diaxhog}$ = **18.0** mm

Permissable minimum nominal cover to all reinforcement (Table 3.4)

; $c_{min}$ = **15** mm

***Cover OK over steel resisting hogging***

;

;

# BS FLOOR SLAB B

## RC SLAB DESIGN (BS8110:PART1:1997)

TEDDS calculation version 1.0.04

### TWO WAY SPANNING SLAB DEFINITION - RESTRAINED

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

Outer sagging steel

; Cover to outer tension reinforcement resisting sagging; $c_{sag}$ = **20** mm

; Trial bar diameter; $D_{tryx}$ = **16** mm

Depth to outer tension steel (resisting sagging)

$d_x = h - c_{sag} - D_{tryx}/2 = \mathbf{372}$ mm

Inner sagging steel

; Trial bar diameter; $D_{tryy}$ = **16** mm

Depth to inner tension steel (resisting sagging)

$d_y = h - c_{sag} - D_{tryx} - D_{tryy}/2 = \mathbf{356}$ mm

Outer hogging steel

; Cover to outer tension reinforcement resisting hogging; $c_{hog}$ = **20** mm

; Trial bar diameter; $D_{tryxhog}$ = **16** mm

Depth to outer tension steel (resisting hogging)

$d_{xhog} = h - c_{hog} - D_{tryxhog}/2 = \mathbf{372}$ mm

Inner hogging steel

; Trial bar diameter; $D_{tryyhog}$ = **16** mm

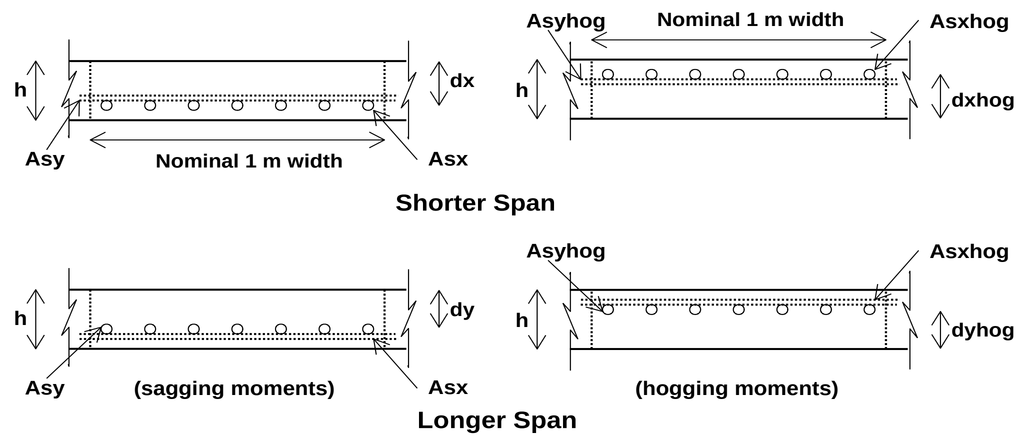
Depth to inner tension steel (resisting hogging)

$d_{yhog} = h - c_{hog} - D_{tryxhog} - D_{tryyhog}/2 = \mathbf{356}$ mm

Materials

; Characteristic strength of reinforcement; $f_y$ = **500** N/mm$^2$

; Characteristic strength of concrete; $f_{cu}$ = **40** N/mm$^2$




**Two-way spanning slab**
**(restrained)**

**<u>RESTRAINED – 2 WAY SPANNING (CL 3.5.3)</u>**

**<u>MAXIMUM DESIGN MOMENTS</u>**

; Length of shorter side of slab; $l_x$ = **5.000** m

; Length of longer side of slab; $l_y$ = **5.000** m

; Design ultimate load per unit area; $n_s$ = **15.0** kN/m$^2$

; Edge condition shorter side (1); $Edge_1$ = **"C"**

; Edge condition other shorter side (2); $Edge_2$ = **"D"**

; Edge condition longer side (3); $Edge_3$ = **"D"**

; Edge condition other longer side (4); $Edge_4$ = **"D"**

Number of discontinuous edges

$N_d = \mathbf{3}$

Moment coefficients

$\beta_{sy} = (24 + 2\times N_d + 1.5\times N_d^2) / 1000 = \mathbf{0.044}$

$\beta_1$ = if($Edge_1$ == "C", 4/3×$\beta_{sy}$,0) = **0.058**

$\beta_2$ = if($Edge_2$ == "C", 4/3×$\beta_{sy}$,0) = **0.000**

$\gamma = 2/9 \times [3 - \sqrt{(18)} \times l_x/l_y \times (\sqrt{(\beta_{sy} + \beta_1)} + \sqrt{(\beta_{sy} + \beta_2)})] = \mathbf{0.170}$

$\beta_{3x}$ = if($Edge_3$ == "C", 4/3,0) = **0.000**

$\beta_{4x}$ = if($Edge_4$ == "C", 4/3,0) = **0.000**

$\beta_{sx} = \gamma / [(1+\beta_{3x})^{0.5} + (1+\beta_{4x})^{0.5}]^2 = \mathbf{0.042}$

$\beta_3 = \beta_{3x} \times \beta_{sx} = \mathbf{0.000}$

$\beta_4 = \beta_{4x} \times \beta_{sx} = \mathbf{0.000}$

Maximum span moments per unit width - restrained slabs

$m_{sx} = \beta_{sx} \times n_s \times l_x^2$ = **15.9** kNm/m

$m_{sy} = \beta_{sy} \times n_s \times l_x^2$ =**16.3** kNm/m

Maximum support moments per unit width - restrained slabs

$m_{sxhog} = \max(\beta_3, \beta_4) \times n_s \times l_x^2$ = **0.0** kNm/m

$m_{syhog} = \max(\beta_1, \beta_2) \times n_s \times l_x^2$ =**21.7** kNm/m

**CONCRETE SLAB DESIGN – SAGGING – OUTER LAYER OF STEEL (CL 3.5.4)**

; Design sagging moment (per m width of slab); $m_{sx}$ = **15.9** kNm/m

; Moment Redistribution Factor; $\beta_{bx}$ = **1.0**

**Area of reinforcement required**

;; $K_x = abs(m_{sx}) / (d_x^2 \times f_{cu})$ = **0.003**

$K'_x = \min(0.156, (0.402 \times (\beta_{bx} - 0.4)) - (0.18 \times (\beta_{bx} - 0.4)^2))$ = **0.156**

***Outer compression steel not required to resist sagging***

***Slab requiring outer tension steel only - bars (sagging)***

;; $z_x = \min((0.95 \times d_x),(d_x \times (0.5+\sqrt{(0.25-K_x/0.9)})))$ = **353** mm

Neutral axis depth; $x_x = (d_x - z_x) / 0.45$ = **41** mm

Area of tension steel required

;;; $A_{sx\_req} = abs(m_{sx}) / (1/\gamma_{ms} \times f_y \times z_x)$ = **104** mm$^2$/m

**Tension steel**

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

$A_{sx\_prov} = A_{sx}$ = **1260** mm$^2$/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); $m_{sy}$ = **16.3** kNm/m

; Moment Redistribution Factor; $\beta_{by}$ = **1.0**

**Area of reinforcement required**

;; $K_y = abs(m_{sy}) / (d_y^2 \times f_{cu})$ = **0.003**

$K'_y = \min(0.156, (0.402 \times (\beta_{by} - 0.4)) - (0.18 \times (\beta_{by} - 0.4)^2))$ = **0.156**

***Inner compression steel not required to resist sagging***

***Slab requiring inner tension steel only - bars (sagging)***

;; $z_y = \min((0.95 \times d_y),(d_y \times (0.5+\sqrt{(0.25-K_y/0.9)})))$ = **338** mm

Neutral axis depth; $x_y = (d_y - z_y) / 0.45$ = **40** mm

Area of tension steel required

;;; $A_{sy\_req} = abs(m_{sy}) / (1/\gamma_{ms} \times f_y \times z_y)$ = **111** mm$^2$/m

**Tension steel**

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

$A_{sy\_prov} = A_{sy} =$ **1260** mm$^2$/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); $m_{sxhog}$ = **0.0** kNm/m

; Moment Redistribution Factor; $\beta_{bx}$ = **1.0**

**Area of reinforcement required**

;; $K_{xhog} = abs(m_{sxhog}) / ( d_{xhog}^2 \times f_{cu} )$ = **0.000**

$K'_x = \min (0.156 , (0.402 \times (\beta_{bx} - 0.4)) - (0.18 \times (\beta_{bx} - 0.4)^2 ))$ = **0.156**

***Outer compression steel not required to resist hogging***

***Slab requiring outer tension steel only - bars (hogging)***

;; $z_{xhog} = \min (( 0.95 \times d_{xhog}),(d_{xhog}\times(0.5+\sqrt{(0.25-K_{xhog}/0.9)})))$ = **353** mm

Neutral axis depth; $x_{xhog} = (d_{xhog} - z_{xhog}) / 0.45$ = **41** mm

Area of tension steel required

;;; $A_{sxhog\_req} = abs(m_{sxhog}) / (1/\gamma_{ms} \times f_y \times z_{xhog})$ = **0** mm$^2$/m

**Tension steel**

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

$A_{sxhog\_prov} = A_{sxhog}$ = **1260** mm$^2$/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); $m_{syhog}$ = **21.7** kNm/m

; Moment Redistribution Factor; $\beta_{by}$ = **1.0**

**Area of reinforcement required**

;; $K_{yhog} = abs(m_{syhog}) / ( d_{yhog}^2 \times f_{cu} )$ = **0.004**

$K'_y = \min (0.156 , (0.402 \times (\beta_{by} - 0.4)) - (0.18 \times (\beta_{by} - 0.4)^2 ))$ = **0.156**

***Inner compression steel not required to resist hogging***

***Slab requiring inner tension steel only - bars (hogging)***

;; $z_{yhog} = \min (( 0.95 \times d_{yhog}),(d_{yhog}\times(0.5+\sqrt{(0.25-K_{yhog}/0.9)})))$ = **338** mm

Neutral axis depth; $x_{yhog} = (d_{yhog} - z_{yhog}) / 0.45$ = **40** mm

Area of tension steel required

;;; $A_{syhog\_req} = abs(m_{syhog}) / (1/\gamma_{ms} \times f_y \times z_{yhog})$ = **148** mm$^2$/m

**Tension steel**

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

$A_{syhog\_prov} = A_{syhog}$ = **1260** mm$^2$/m

***Area of inner tension steel provided sufficient to resist hogging***

***Check min and max areas of steel resisting sagging***

;Total area of concrete; $A_c = h$ = **400000** mm$^2$/m

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

$A_{st\_min} = k \times A_c =$ **520** mm$^2$/m

$A_{st\_max} = 4\ \% \times A_c =$ **16000** mm$^2$/m

Steel defined:

; Outer steel resisting sagging; $A_{sx\_prov}$ = **1260** mm$^2$/m

***Area of outer steel provided (sagging) OK***

; Inner steel resisting sagging; $A_{sy\_prov}$ = **1260** mm$^2$/m

***Area of inner steel provided (sagging) OK***

***Check min and max areas of steel resisting hogging***

;Total area of concrete; $A_c = h =$ **400000** mm$^2$/m

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

$A_{st\_min} = k \times A_c =$ **520** mm$^2$/m

$A_{st\_max} = 4\ \% \times A_c =$ **16000** mm$^2$/m

Steel defined:

; Outer steel resisting hogging; $A_{sxhog\_prov}$ = **1260** mm$^2$/m

***Area of outer steel provided (hogging) OK***

; Inner steel resisting hogging ; $A_{syhog\_prov}$ = **1260** mm$^2$/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; $d_x$ = **372** mm

; Area of tension reinforcement provided (per m width of slab); $A_{sx\_prov}$ = **1260** mm$^2$/m

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

; Characteristic strength of concrete; $f_{cu}$ = **40** N/mm$^2$

**Applied shear stress**

$v_x = V_x / d_x =$ **0.07** N/mm$^2$

**Check shear stress to clause 3.5.5.2**

$v_{allowable} = \min((0.8\ N^{1/2}/mm) \times \sqrt{(f_{cu})}, 5\ N/mm^2) =$ **5.00** N/mm$^2$

***Shear stress - OK***

**Shear stresses to clause 3.5.5.3**

**Design shear stress**

$f_{cu\_ratio}$ = if ($f_{cu}$ > 40 N/mm$^2$ , 40/25 , $f_{cu}$/(25 N/mm$^2$)) = **1.600**

$v_{cx} = 0.79\ N/mm^2 \times \min(3, 100 \times A_{sx\_prov} / d_x)^{1/3} \times \max(0.67, (400\ mm / d_x)^{1/4}) / 1.25 \times f_{cu\_ratio}^{1/3}$

$v_{cx}$ = **0.52** N/mm$^2$

Applied shear stress

$v_x$ = **0.07** N/mm$^2$

***No shear reinforcement required***

**SHEAR RESISTANCE OF CONCRETE SLABS (CL 3.5.5)**

**Inner tension steel resisting sagging moments**

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

; Area of tension reinforcement provided (per m width of slab); $A_{sy\_prov}$ = **1260** $mm^2/m$

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

; Characteristic strength of concrete; $f_{cu}$ = **40** $N/mm^2$

**Applied shear stress**

$v_y = V_y / d_y =$ **0.06** $N/mm^2$

**Check shear stress to clause 3.5.5.2**

$v_{allowable} = \min((0.8\ N^{1/2}/mm) \times \sqrt{(f_{cu})}, 5\ N/mm^2) =$ **5.00** $N/mm^2$

***Shear stress - OK***

**Shear stresses to clause 3.5.5.3**

**Design shear stress**

$f_{cu\_ratio} = \text{if}\ (f_{cu} > 40\ N/mm^2, 40/25, f_{cu}/(25\ N/mm^2)) =$ **1.600**

$v_{cy} = 0.79\ N/mm^2 \times \min(3{,}100 \times A_{sy\_prov} / d_y)^{1/3} \times \max(0.67,(400\ mm) / d_y)^{1/4} / 1.25 \times f_{cu\_ratio}^{1/3}$

$v_{cy} =$ **0.54** $N/mm^2$

Applied shear stress

$v_y =$ **0.06** $N/mm^2$

***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; $d_x$ = **372** mm

; Dimension from edge of load to shear perimeter; $l_p = k_p \times d_x =$ **558** mm; where; $k_p$ = **1.50**

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

Total length of inner perimeter at edge of loaded area; $u_{0\_gen} = 2 \times (l + w) =$ **1200** mm

Total length of outer perimeter at $l_p$ from loaded area; $u_{gen} = 2 \times (l + w) + 8 \times l_p =$ **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; $u_0$ = **1200** mm

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

; Depth to outer steel; $d_x$ = **372** mm

; Depth to inner steel; $d_y$ = **356** mm

Average depth to "tension" steel; $d_{av} = (d_x + d_y)/2 =$ **364.0** mm

; Area of outer steel per m effective through the perimeter; $A_{sx\_prov}$ = **1260** mm$^2$ /m

; Area of inner steel per m effective through the perimeter; $A_{sy\_prov}$ = **1260** mm$^2$ /m

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

; Characteristic strength of concrete; $f_{cu}$ = **40** N/mm$^2$

**Applied shear stress**

Stress around loaded area; $v_{max} = V_p / (u_0 \times d_{av})$ = **0.114** N/mm$^2$

Stress around perimeter; $v = V_p / (u \times d_{av})$ = **0.024** N/mm$^2$

**Check shear stress to clause 3.7.7.2**

$v_{allowable} = \min((0.8\ N^{1/2}/mm) \times \sqrt{(f_{cu})}, 5\ N/mm^2)$ = **5.000** N/mm$^2$

***Shear stress - OK***

**Shear stresses to clause 3.7.7.4**

**Design shear stress**

$f_{cu\_ratio}$ = if ($f_{cu}$ > 40 N/mm$^2$ , 40/25 , $f_{cu}$/(25 N/mm$^2$)) = **1.600**

; Effective steel area for shear strength determination:; $A_{s\_eff}$ =**201** mm$^2$/m;

$v_c = 0.79\ N/mm^2 \times \min(3, 100\times(A_{s\_eff} / d_{av}))^{1/3} \times \max(0.67, (400\ mm / d_{av})^{1/4}) / 1.25 \times f_{cu\_ratio}^{1/3}$

$v_c$ = **0.288** N/mm$^2$

***No shear reinforcement required***

**CONCRETE SLAB DEFLECTION CHECK (CL 3.5.7)**

; Slab span length; $l_x$ = **5.000** m

; Design ultimate moment in shorter span per m width; $m_{sx}$ = **16** kNm/m

; Depth to outer tension steel; $d_x$ = **372** mm

**Tension steel**

; Area of outer tension reinforcement provided; $A_{sx\_prov}$ = **1260** mm$^2$/m

; Area of tension reinforcement required; $A_{sx\_req}$ = **104** mm$^2$/m

; Moment Redistribution Factor; $\beta_{bx}$ = **1.00**

**Modification Factors**

;Basic span / effective depth ratio (Table 3.9); $ratio_{span\_depth}$ = **60**

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

;$f_s = 2 \times f_y \times A_{sx\_req} / (3 \times A_{sx\_prov} \times \beta_{bx})$ = **27.4** N/mm$^2$

$factor_{tens} = \min(2, 0.55 + (477\ N/mm^2 - f_s) / (120 \times (0.9\ N/mm^2 + m_{sx} / d_x^2)))$ = **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; $l_{max} = ratio_{span\_depth} \times factor_{tens} \times d_x$ = **44.64** m

**Check the actual beam span**

Actual span/depth ratio; $l_x / d_x$ = **13.44**

Span depth limit; $ratio_{span\_depth} \times factor_{tens}$ = **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; $d_x$ = **372.0** mm

; Diameter of tension reinforcement; $D_x$ = **20** mm

; Diameter of links; $L_{diax}$ = **0** mm

Cover to outer tension reinforcement

$c_{tenx} = h - d_x - D_x / 2$ = **18.0** mm

Nominal cover to links steel

$c_{nomx} = c_{tenx} - L_{diax}$ = **18.0** mm

Permissable minimum nominal cover to all reinforcement (Table 3.4)

; $c_{min}$ = **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; $d_{xhog}$ = **372.0** mm

; Diameter of tension reinforcement; $D_{xhog}$ = **20** mm

; Diameter of links; $L_{diaxhog}$ = **0** mm

Cover to outer tension reinforcement

$c_{tenxhog} = h - d_{xhog} - D_{xhog} / 2$ = **18.0** mm

Nominal cover to links steel

$c_{nomxhog} = c_{tenxhog} - L_{diaxhog}$ = **18.0** mm

Permissable minimum nominal cover to all reinforcement (Table 3.4)

; $c_{min}$ = **15** mm

***Cover OK over steel resisting hogging***

;

;

# BS FLOOR SLAB C

## RC SLAB DESIGN (BS8110:PART1:1997)

TEDDS calculation version 1.0.04

## TWO WAY SPANNING SLAB DEFINITION - RESTRAINED

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

Outer sagging steel

; Cover to outer tension reinforcement resisting sagging; $c_{sag}$ = **20** mm

; Trial bar diameter; $D_{tryx}$ = **16** mm

Depth to outer tension steel (resisting sagging)

$d_x = h - c_{sag} - D_{tryx}/2$ = **222** mm

Inner sagging steel

; Trial bar diameter; $D_{tryy}$ = **16** mm

Depth to inner tension steel (resisting sagging)

$d_y = h - c_{sag} - D_{tryx} - D_{tryy}/2$ = **206** mm

Outer hogging steel

; Cover to outer tension reinforcement resisting hogging; $c_{hog}$ = **20** mm

; Trial bar diameter; $D_{tryxhog}$ = **16** mm

Depth to outer tension steel (resisting hogging)

$d_{xhog} = h - c_{hog} - D_{tryxhog}/2$ = **222** mm

Inner hogging steel

; Trial bar diameter; $D_{tryyhog}$ = **16** mm

Depth to inner tension steel (resisting hogging)

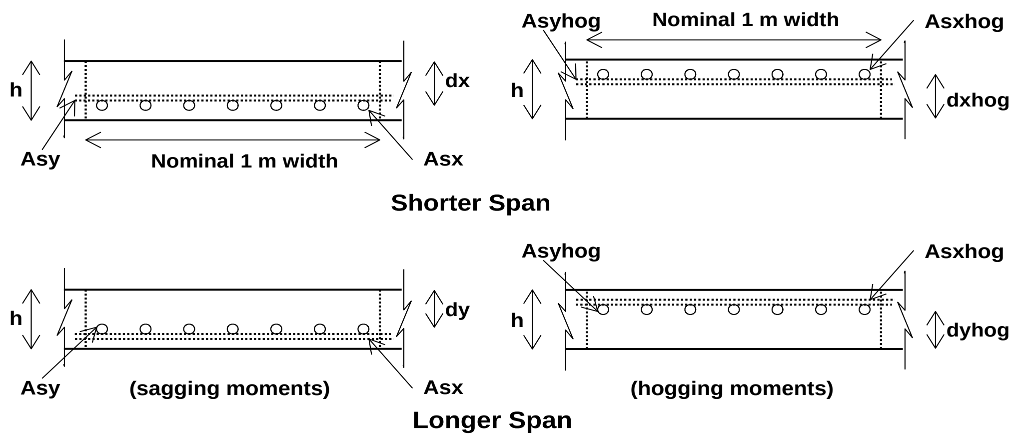
$d_{yhog} = h - c_{hog} - D_{tryxhog} - D_{tryyhog}/2$ = **206** mm

Materials

; Characteristic strength of reinforcement; $f_y$ = **500** N/mm$^2$

; Characteristic strength of concrete; $f_{cu}$ = **40** N/mm$^2$

Kelly Mweu Changilwa
+254703206460
mweukelly@gmail.com


**Two-way spanning slab**

**(restrained)**

**<u>RESTRAINED – 2 WAY SPANNING (CL 3.5.3)</u>**

**<u>MAXIMUM DESIGN MOMENTS</u>**

; Length of shorter side of slab; $l_x$ = **5.000** m

; Length of longer side of slab; $l_y$ = **5.000** m

; Design ultimate load per unit area; $n_s$ = **15.0** kN/m$^2$

; Edge condition shorter side (1); $Edge_1$ = **"C"**

; Edge condition other shorter side (2); $Edge_2$ = **"C"**

; Edge condition longer side (3); $Edge_3$ = **"C"**

; Edge condition other longer side (4); $Edge_4$ = **"C"**

Number of discontinuous edges

$N_d = \mathbf{0}$

Moment coefficients

$\beta_{sy} = (24 + 2\times N_d + 1.5\times N_d^2) / 1000 = \mathbf{0.024}$

$\beta_1$ = if($Edge_1$ == "C", 4/3×$\beta_{sy}$,0) = **0.032**

$\beta_2$ = if($Edge_2$ == "C", 4/3×$\beta_{sy}$,0) = **0.032**

$\gamma = 2/9 \times [3 - \sqrt{(18)} \times l_x/l_y \times (\sqrt{(\beta_{sy} + \beta_1)} + \sqrt{(\beta_{sy} + \beta_2)})] = \mathbf{0.220}$

$\beta_{3x}$ = if($Edge_3$ == "C", 4/3,0) = **1.333**

$\beta_{4x}$ = if($Edge_4$ == "C", 4/3,0) = **1.333**

$\beta_{sx} = \gamma / [(1+\beta_{3x})^{0.5} + (1+\beta_{4x})^{0.5}]^2 = \mathbf{0.024}$

$\beta_3 = \beta_{3x} \times \beta_{sx} = \mathbf{0.031}$

$\beta_4 = \beta_{4x} \times \beta_{sx} = \mathbf{0.031}$

Tekla Tedds

Kelly Mweu Changilwa
+254703206460
mweukelly@gmail.com



Maximum span moments per unit width - restrained slabs

$m_{sx} = \beta_{sx} \times n_s \times l_x^2$ = **8.9** kNm/m

$m_{sy} = \beta_{sy} \times n_s \times l_x^2$ =**9.0** kNm/m

Maximum support moments per unit width - restrained slabs

$m_{sxhog}$ = max( $\beta_3$ , $\beta_4$ ) $\times n_s \times l_x^2$ = **11.8** kNm/m

$m_{syhog}$ = max( $\beta_1$ , $\beta_2$ ) $\times n_s \times l_x^2$ =**12.0** kNm/m

**CONCRETE SLAB DESIGN – SAGGING – OUTER LAYER OF STEEL (CL 3.5.4)**

; Design sagging moment (per m width of slab); $m_{sx}$ = **8.9** kNm/m

; Moment Redistribution Factor; $\beta_{bx}$ = **1.0**

**Area of reinforcement required**

;; $K_x = \text{abs}(m_{sx}) / ( d_x^2 \times f_{cu} )$ = **0.004**

$K'_x = \min (0.156 , (0.402 \times (\beta_{bx} - 0.4)) - (0.18 \times (\beta_{bx} - 0.4)^2 ))$ = **0.156**

***Outer compression steel not required to resist sagging***

***Slab requiring outer tension steel only - bars (sagging)***

;; $z_x = \min (( 0.95 \times d_x),(d_x\times(0.5+\sqrt{(0.25-K_x/0.9)})))$ = **211** mm

Neutral axis depth; $x_x = (d_x - z_x) / 0.45$ = **25** mm

Area of tension steel required

;;; $A_{sx\_req} = \text{abs}(m_{sx}) / (1/\gamma_{ms} \times f_y \times z_x)$ = **97** mm$^2$/m

**Tension steel**

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

$A_{sx\_prov} = A_{sx}$ = **1050** mm$^2$/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); $m_{sy}$ = **9.0** kNm/m

; Moment Redistribution Factor; $\beta_{by}$ = **1.0**

**Area of reinforcement required**

;; $K_y = \text{abs}(m_{sy}) / ( d_y^2 \times f_{cu} )$ = **0.005**

$K'_y = \min (0.156 , (0.402 \times (\beta_{by} - 0.4)) - (0.18 \times (\beta_{by} - 0.4)^2 ))$ = **0.156**

***Inner compression steel not required to resist sagging***

***Slab requiring inner tension steel only - bars (sagging)***

;; $z_y = \min (( 0.95 \times d_y),(d_y\times(0.5+\sqrt{(0.25-K_y/0.9)})))$ = **196** mm

Neutral axis depth; $x_y = (d_y - z_y) / 0.45$ = **23** mm

Area of tension steel required

;;; $A_{sy\_req} = \text{abs}(m_{sy}) / (1/\gamma_{ms} \times f_y \times z_y)$ = **106** mm$^2$/m

**Tension steel**

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

$A_{sy\_prov} = A_{sy} =$ **1050** mm$^2$/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); $m_{sxhog}$ = **11.8** kNm/m

; Moment Redistribution Factor; $\beta_{bx}$ = **1.0**

**Area of reinforcement required**

;; $K_{xhog} = abs(m_{sxhog}) / ( d_{xhog}^2 \times f_{cu} )$ = **0.006**

$K'_x = \min (0.156 , (0.402 \times (\beta_{bx} - 0.4)) - (0.18 \times (\beta_{bx} - 0.4)^2 ))$ = **0.156**

***Outer compression steel not required to resist hogging***

***Slab requiring outer tension steel only - bars (hogging)***

;; $z_{xhog} = \min (( 0.95 \times d_{xhog}),(d_{xhog}\times(0.5+\sqrt{(0.25-K_{xhog}/0.9)})))$ = **211** mm

Neutral axis depth; $x_{xhog} = (d_{xhog} - z_{xhog}) / 0.45$ = **25** mm

Area of tension steel required

;;; $A_{sxhog\_req} = abs(m_{sxhog}) / (1/\gamma_{ms} \times f_y \times z_{xhog})$ = **129** mm$^2$/m

**Tension steel**

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

$A_{sxhog\_prov} = A_{sxhog} =$ **1050** mm$^2$/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); $m_{syhog}$ = **12.0** kNm/m

; Moment Redistribution Factor; $\beta_{by}$ = **1.0**

**Area of reinforcement required**

;; $K_{yhog} = abs(m_{syhog}) / ( d_{yhog}^2 \times f_{cu} )$ = **0.007**

$K'_y = \min (0.156 , (0.402 \times (\beta_{by} - 0.4)) - (0.18 \times (\beta_{by} - 0.4)^2 ))$ = **0.156**

***Inner compression steel not required to resist hogging***

***Slab requiring inner tension steel only - bars (hogging)***

;; $z_{yhog} = \min (( 0.95 \times d_{yhog}),(d_{yhog}\times(0.5+\sqrt{(0.25-K_{yhog}/0.9)})))$ = **196** mm

Neutral axis depth; $x_{yhog} = (d_{yhog} - z_{yhog}) / 0.45$ = **23** mm

Area of tension steel required

;;; $A_{syhog\_req} = abs(m_{syhog}) / (1/\gamma_{ms} \times f_y \times z_{yhog})$ = **141** mm$^2$/m

**Tension steel**

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

$A_{syhog\_prov} = A_{syhog} =$ **1050** mm$^2$/m

***Area of inner tension steel provided sufficient to resist hogging***

***Check min and max areas of steel resisting sagging***

;Total area of concrete; $A_c = h =$ **250000** mm$^2$/m

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

$A_{st\_min} = k \times A_c =$ **325** mm$^2$/m

$A_{st\_max} = 4\ \% \times A_c =$ **10000** mm$^2$/m

Steel defined:

; Outer steel resisting sagging; $A_{sx\_prov} =$ **1050** mm$^2$/m

***Area of outer steel provided (sagging) OK***

; Inner steel resisting sagging; $A_{sy\_prov} =$ **1050** mm$^2$/m

***Area of inner steel provided (sagging) OK***

***<u>Check min and max areas of steel resisting hogging</u>***

;Total area of concrete; $A_c = h =$ **250000** mm$^2$/m

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

$A_{st\_min} = k \times A_c =$ **325** mm$^2$/m

$A_{st\_max} = 4\ \% \times A_c =$ **10000** mm$^2$/m

Steel defined:

; Outer steel resisting hogging; $A_{sxhog\_prov} =$ **1050** mm$^2$/m

***Area of outer steel provided (hogging) OK***

; Inner steel resisting hogging ; $A_{syhog\_prov} =$ **1050** mm$^2$/m

***Area of inner steel provided (hogging) OK***

**<u>SHEAR RESISTANCE OF CONCRETE SLABS (CL 3.5.5)</u>**

**Outer tension steel resisting sagging moments**

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

; Area of tension reinforcement provided (per m width of slab); $A_{sx\_prov} =$ **1050** mm$^2$/m

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

; Characteristic strength of concrete; $f_{cu} =$ **40** N/mm$^2$

**Applied shear stress**

$v_x = V_x / d_x =$ **0.11** N/mm$^2$

**Check shear stress to clause 3.5.5.2**

$v_{allowable} = \min((0.8\ N^{1/2}/mm) \times \sqrt{(f_{cu})}, 5\ N/mm^2) =$ **5.00** N/mm$^2$

***Shear stress - OK***

**Shear stresses to clause 3.5.5.3**

**Design shear stress**

$f_{cu\_ratio} = \text{if}(f_{cu} > 40\ N/mm^2, 40/25, f_{cu}/(25\ N/mm^2)) =$ **1.600**

$v_{cx} = 0.79\ N/mm^2 \times \min(3, 100 \times A_{sx\_prov} / d_x)^{1/3} \times \max(0.67, (400\ mm / d_x)^{1/4}) / 1.25 \times f_{cu\_ratio}^{1/3}$

$v_{cx} =$ **0.67** N/mm$^2$

Applied shear stress

$v_x =$ **0.11** N/mm$^2$

***No shear reinforcement required***

**SHEAR RESISTANCE OF CONCRETE SLABS (CL 3.5.5)**

**Inner tension steel resisting sagging moments**

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

; Area of tension reinforcement provided (per m width of slab); $A_{sy\_prov}$ = **1050** mm$^2$/m

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

; Characteristic strength of concrete; $f_{cu}$ = **40** N/mm$^2$

**Applied shear stress**

$v_y = V_y / d_y$ = **0.10** N/mm$^2$

**Check shear stress to clause 3.5.5.2**

$v_{allowable} = \min ((0.8 \text{ N}^{1/2}/\text{mm}) \times \sqrt{(f_{cu})}, 5 \text{ N/mm}^2)$ = **5.00** N/mm$^2$

***Shear stress - OK***

**Shear stresses to clause 3.5.5.3**

**Design shear stress**

$f_{cu\_ratio} = \text{if } (f_{cu} > 40 \text{ N/mm}^2, 40/25, f_{cu}/(25 \text{ N/mm}^2))$ = **1.600**

$v_{cy} = 0.79 \text{ N/mm}^2 \times \min(3,100 \times A_{sy\_prov} / d_y)^{1/3} \times \max(0.67,(400 \text{ mm}) / d_y)^{1/4} / 1.25 \times f_{cu\_ratio}^{1/3}$

$v_{cy}$ = **0.70** N/mm$^2$

Applied shear stress

$v_y$ = **0.10** N/mm$^2$

***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; $d_x$ = **222** mm

; Dimension from edge of load to shear perimeter; $l_p = k_p \times d_x$ = **333** mm; where; $k_p$ = **1.50**

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

Total length of inner perimeter at edge of loaded area; $u_{0\_gen} = 2 \times (l + w)$ = **1200** mm

Total length of outer perimeter at $l_p$ from loaded area; $u_{gen} = 2 \times (l + w) + 8 \times l_p$ = **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; $u_0$ = **1200** mm

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

; Depth to outer steel; $d_x$ = **222** mm

; Depth to inner steel; $d_y$ = **206** mm

Average depth to "tension" steel; $d_{av} = (d_x + d_y)/2$ = **214.0** mm

; Area of outer steel per m effective through the perimeter; $A_{sx\_prov}$ = **1050** mm$^2$ /m

; Area of inner steel per m effective through the perimeter; $A_{sy\_prov}$ = **1050** mm$^2$ /m

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

; Characteristic strength of concrete; $f_{cu}$ = **40** N/mm$^2$

**Applied shear stress**

Stress around loaded area; $v_{max} = V_p / (u_0 \times d_{av})$ = **0.195** N/mm$^2$

Stress around perimeter; $v = V_p / (u \times d_{av})$ = **0.060** N/mm$^2$

**Check shear stress to clause 3.7.7.2**

$v_{allowable} = \min((0.8 \text{ N}^{1/2}/\text{mm}) \times \sqrt{(f_{cu})}, 5 \text{ N/mm}^2)$ = **5.000** N/mm$^2$

*Shear stress - OK*

**Shear stresses to clause 3.7.7.4**

**Design shear stress**

$f_{cu\_ratio}$ = if ($f_{cu}$ > 40 N/mm$^2$ , 40/25 , $f_{cu}$/(25 N/mm$^2$)) = **1.600**

; Effective steel area for shear strength determination:; $A_{s\_eff}$ =**201** mm$^2$/m;

$v_c = 0.79 \text{ N/mm}^2 \times \min(3, 100\times(A_{s\_eff} / d_{av}))^{1/3} \times \max(0.67, (400 \text{ mm} / d_{av})^{1/4}) / 1.25 \times f_{cu\_ratio}^{1/3}$

$v_c$ = **0.393** N/mm$^2$

***No shear reinforcement required***

**<u>CONCRETE SLAB DEFLECTION CHECK (CL 3.5.7)</u>**

; Slab span length; $l_x$ = **5.000** m

; Design ultimate moment in shorter span per m width; $m_{sx}$ = **9** kNm/m

; Depth to outer tension steel; $d_x$ = **222** mm

**Tension steel**

; Area of outer tension reinforcement provided; $A_{sx\_prov}$ = **1050** mm$^2$/m

; Area of tension reinforcement required; $A_{sx\_req}$ = **97** mm$^2$/m

; Moment Redistribution Factor; $\beta_{bx}$ = **1.00**

**Modification Factors**

;Basic span / effective depth ratio (Table 3.9); $ratio_{span\_depth}$ = **60**

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

;$f_s = 2 \times f_y \times A_{sx\_req} / (3 \times A_{sx\_prov} \times \beta_{bx})$ = **30.7** N/mm$^2$

$factor_{tens} = \min(2, 0.55 + (477 \text{ N/mm}^2 - f_s) / (120 \times (0.9 \text{ N/mm}^2 + m_{sx} / d_x^2)))$ = **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; $l_{max} = ratio_{span\_depth} \times factor_{tens} \times d_x$ = **26.64** m

**Check the actual beam span**

Actual span/depth ratio; $l_x / d_x$ = **22.52**

Span depth limit; $ratio_{span\_depth} \times factor_{tens}$ = **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; $d_x$ = **222.0** mm

; Diameter of tension reinforcement; $D_x$ = **20** mm

; Diameter of links; $L_{diax}$ = **0** mm

Cover to outer tension reinforcement

$c_{tenx} = h - d_x - D_x / 2 =$ **18.0** mm

Nominal cover to links steel

$c_{nomx} = c_{tenx} - L_{diax} =$ **18.0** mm

Permissable minimum nominal cover to all reinforcement (Table 3.4)

; $c_{min}$ = **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; $d_{xhog}$ = **222.0** mm

; Diameter of tension reinforcement; $D_{xhog}$ = **20** mm

; Diameter of links; $L_{diaxhog}$ = **0** mm

Cover to outer tension reinforcement

$c_{tenxhog} = h - d_{xhog} - D_{xhog} / 2 =$ **18.0** mm

Nominal cover to links steel

$c_{nomxhog} = c_{tenxhog} - L_{diaxhog} =$ **18.0** mm

Permissable minimum nominal cover to all reinforcement (Table 3.4)

; $c_{min}$ = **15** mm

***Cover OK over steel resisting hogging***

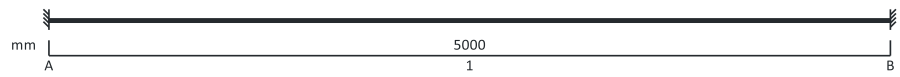
;

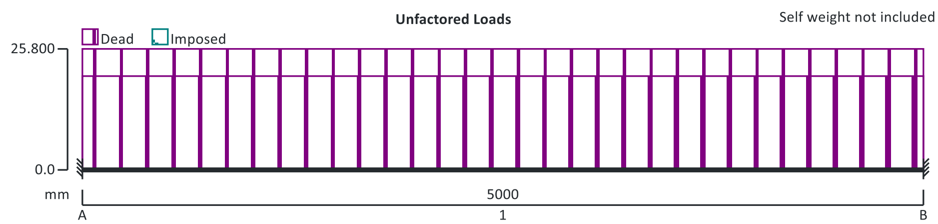
;

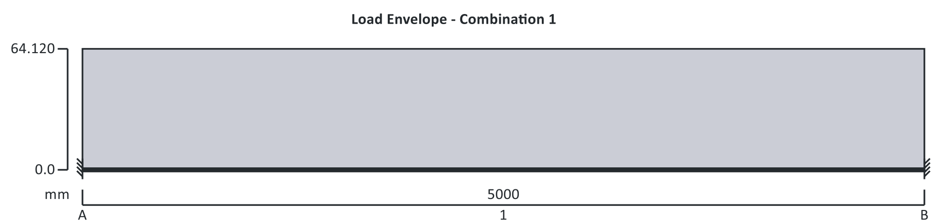
# BS MAIN PARKING BEAM

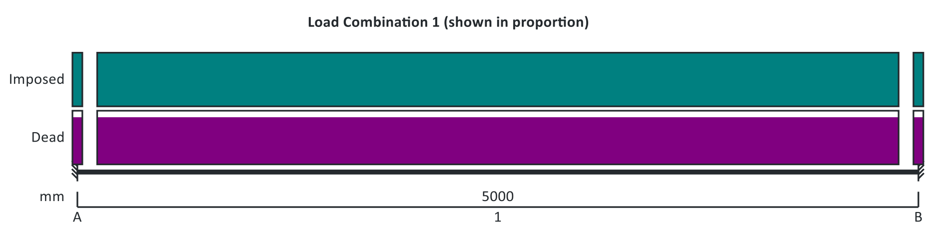
## RC BEAM ANALYSIS & DESIGN BS8110

TEDDS calculation version 2.1.14

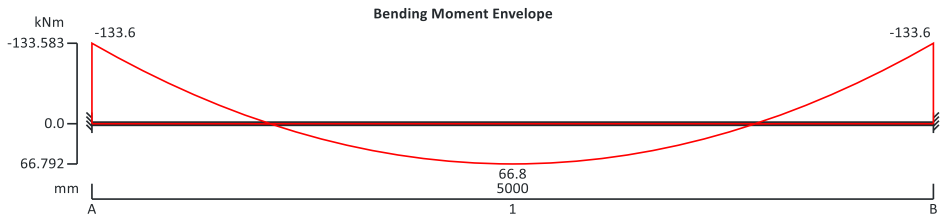


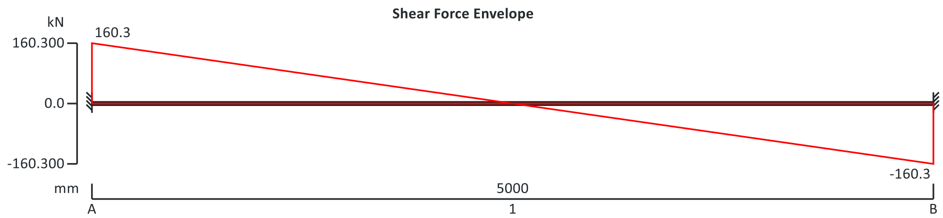



| Tekla Tedds | Project: REVIEW AND ANALYSIS OF AN UNDERGROUND | | | Job Ref.: FCE 590 | |
|---|---|---|---|---|---|
| Kelly Mweu Changilwa<br>+254703206460<br>mweukelly@gmail.com | Section: HFB STRUCTURAL ANALYSIS | | | Sheet no./rev.: 29 | |
| | Calc. by: KELLY C. | Date: 03/05/2023 | Chk'd: PROF | By: MUMENYA | |




**Support conditions**

| | | |
|---|---|---|
| Support A | Vertically restrained | |
| | Rotationally restrained | |
| Support B | Vertically restrained | |
| | Rotationally restrained | |

**Applied loading**

| | | |
|---|---|---|
| | Dead full UDL 25.8 kN/m | |
| | Dead full UDL 20 kN/m | |

**Load combinations**

| | | |
|---|---|---|
| Load combination 1 | Support A | Dead × 1.40 |
| | | Imposed × 1.60 |
| | Span 1 | Dead × 1.40 |
| | | Imposed × 1.60 |
| | Support B | Dead × 1.40 |
| | | Imposed × 1.60 |

**Analysis results**

| | | |
|---|---|---|
| Maximum moment support A; | $M_{A\_max}$ = **-134** kNm; | $M_{A\_red}$ = **-134** kNm; |
| Maximum moment span 1 at 2500 mm; | $M_{s1\_max}$ = **67** kNm; | $M_{s1\_red}$ = **67** kNm; |
| Maximum moment support B; | $M_{B\_max}$ = **-134** kNm; | $M_{B\_red}$ = **-134** kNm; |
| Maximum shear support A; | $V_{A\_max}$ = **160** kN; | $V_{A\_red}$ = **160** kN |
| Maximum shear support A span 1 at 452 mm; | $V_{A\_s1\_max}$ = **131** kN; | $V_{A\_s1\_red}$ = **131** kN |
| Maximum shear support B; | $V_{B\_max}$ = **-160** kN; | $V_{B\_red}$ = **-160** kN |
| Maximum shear support B span 1 at 4548 mm; | $V_{B\_s1\_max}$ = **-131** kN; | $V_{B\_s1\_red}$ = **-131** kN |
| Maximum reaction at support A; | $R_A$ = **160** kN | |
| Unfactored dead load reaction at support A; | $R_{A\_Dead}$ = **115** kN | |
| Maximum reaction at support B; | $R_B$ = **160** kN | |

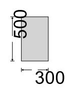
| Tekla Tedds | Project<br>REVIEW AND ANALYSIS OF AN UNDERGROUND | | | Job Ref.<br>FCE 590 | |
|---|---|---|---|---|---|
| Kelly Mweu Changilwa<br>+254703206460<br>mweukelly@gmail.com | Section<br>HFB STRUCTURAL ANALYSIS | | | Sheet no./rev.<br>30 | |
| | Calc. by<br>KELLY C. | Date<br>03/05/2023 | Chk'd<br>PROF | By<br>MUMENYA | | |

Unfactored dead load reaction at support B; $R_{B\_Dead}$ = **115** kN

**Rectangular section details**

Section width; b = **300** mm

Section depth; h = **500** mm




**Concrete details**

Concrete strength class; **C25/30**

Characteristic compressive cube strength; $f_{cu}$ = **30** N/mm$^2$

Modulus of elasticity of concrete; $E_c$ = 20kN/mm$^2$ + 200 × $f_{cu}$ = **26000** N/mm$^2$

Maximum aggregate size; $h_{agg}$ = **20** mm

**Reinforcement details**

Characteristic yield strength of reinforcement; $f_y$ = **500** N/mm$^2$

Characteristic yield strength of shear reinforcement; $f_{yv}$ = **500** N/mm$^2$

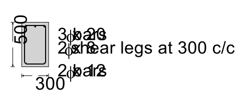
**Nominal cover to reinforcement**

Nominal cover to top reinforcement; $c_{nom\_t}$ = **30** mm

Nominal cover to bottom reinforcement; $c_{nom\_b}$ = **30** mm

Nominal cover to side reinforcement; $c_{nom\_s}$ = **30** mm

**<u>Support A</u>**




**Rectangular section in flexure (cl.3.4.4)**

Design bending moment; M = abs($M_{A\_red}$) = **134** kNm

Depth to tension reinforcement; d = h - $c_{nom\_t}$ - $\phi_v$ - $\phi_{top}$ / 2 = **452** mm

Redistribution ratio; $\beta_b$ = min(1 - $m_{rA}$, 1) = **1.000**

K = M / (b × d$^2$ × $f_{cu}$) = **0.073**

K' = 0.156

***K' > K - No compression reinforcement is required***

Lever arm; $z = \min(d \times (0.5 + (0.25 - K / 0.9)^{0.5}), 0.95 \times d) = \mathbf{412}$ mm

Depth of neutral axis; $x = (d - z) / 0.45 = \mathbf{89}$ mm

Area of tension reinforcement required; $A_{s,req} = M / (0.87 \times f_y \times z) = \mathbf{745}$ mm$^2$

Tension reinforcement provided; $3 \times 20\phi$ bars

Area of tension reinforcement provided; $A_{s,prov} = \mathbf{942}$ mm$^2$

Minimum area of reinforcement; $A_{s,min} = 0.0013 \times b \times h = \mathbf{195}$ mm$^2$

Maximum area of reinforcement; $A_{s,max} = 0.04 \times b \times h = \mathbf{6000}$ mm$^2$

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

**Rectangular section in shear**

Design shear force span 1 at 452 mm; $V = \max(V_{A\_s1\_max}, V_{A\_s1\_red}) = \mathbf{131}$ kN

Design shear stress; $v = V / (b \times d) = \mathbf{0.968}$ N/mm$^2$

Design concrete shear stress; $v_c = 0.79 \times \min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times \max(1, (400 /d)^{1/4}) \times (\min(f_{cu}, 40) / 25)^{1/3} / \gamma_m$

$v_c = \mathbf{0.595}$ N/mm$^2$

Allowable design shear stress; $v_{max} = \min(0.8\ N/mm^2 \times (f_{cu}/1\ N/mm^2)^{0.5}, 5\ N/mm^2) = \mathbf{4.382}$ N/mm$^2$

***PASS - Design shear stress is less than maximum allowable***

Value of v from Table 3.7; $0.5 \times v_c < v < (v_c + 0.4\ N/mm^2)$

Design shear resistance required; $v_s = \max(v - v_c, 0.4\ N/mm^2) = \mathbf{0.400}$ N/mm$^2$

Area of shear reinforcement required; $A_{sv,req} = v_s \times b / (0.87 \times f_{yv}) = \mathbf{276}$ mm$^2$/m

Shear reinforcement provided; $2 \times 8\phi$ legs at 300 c/c

Area of shear reinforcement provided; $A_{sv,prov} = \mathbf{335}$ mm$^2$/m

***PASS - Area of shear reinforcement provided exceeds minimum required***

Maximum longitudinal spacing; $s_{vl,max} = 0.75 \times d = \mathbf{339}$ mm

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

**Spacing of reinforcement (cl 3.12.11)**

Actual distance between bars in tension; $s = (b - 2 \times (c_{nom\_s} + \phi_v + \phi_{top}/2)) /(N_{top} - 1) - \phi_{top} = \mathbf{82}$ mm

**Minimum distance between bars in tension (cl 3.12.11.1)**

Minimum distance between bars in tension; $s_{min} = h_{agg} + 5\ mm = \mathbf{25}$ mm

***PASS - Satisfies the minimum spacing criteria***

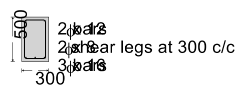
**Maximum distance between bars in tension (cl 3.12.11.2)**

Design service stress; $f_s = (2 \times f_y \times A_{s,req}) / (3 \times A_{s,prov} \times \beta_b) = \mathbf{263.6}$ N/mm$^2$

Maximum distance between bars in tension; $s_{max} = \min(47000\ N/mm / f_s, 300\ mm) = \mathbf{178}$ mm

***PASS - Satisfies the maximum spacing criteria***

**Mid span 1**




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

Design bending moment; $M = abs(M_{s1\_red}) = \mathbf{67}$ kNm

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_v - \phi_{bot} / 2 = \mathbf{454}$ mm

Redistribution ratio; $\beta_b = \min(1 - m_{rs1}, 1) = \mathbf{1.000}$

$K = M / (b \times d^2 \times f_{cu}) = \mathbf{0.036}$

$K' = 0.156$

***K' > K - No compression reinforcement is required***

Lever arm; $z = \min(d \times (0.5 + (0.25 - K / 0.9)^{0.5}), 0.95 \times d) = \mathbf{431}$ mm

Depth of neutral axis; $x = (d - z) / 0.45 = \mathbf{50}$ mm

Area of tension reinforcement required; $A_{s,req} = M / (0.87 \times f_y \times z) = \mathbf{356}$ mm$^2$

Tension reinforcement provided; $3 \times 16\phi$ bars

Area of tension reinforcement provided; $A_{s,prov} = \mathbf{603}$ mm$^2$

Minimum area of reinforcement; $A_{s,min} = 0.0013 \times b \times h = \mathbf{195}$ mm$^2$

Maximum area of reinforcement; $A_{s,max} = 0.04 \times b \times h = \mathbf{6000}$ mm$^2$

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

**Rectangular section in shear**

Shear reinforcement provided; $2 \times 8\phi$ legs at 300 c/c

Area of shear reinforcement provided; $A_{sv,prov} = \mathbf{335}$ mm$^2$/m

Minimum area of shear reinforcement (Table 3.7); $A_{sv,min} = 0.4\text{N/mm}^2 \times b / (0.87 \times f_{yv}) = \mathbf{276}$ mm$^2$/m

***PASS - Area of shear reinforcement provided exceeds minimum required***

Maximum longitudinal spacing (cl. 3.4.5.5); $s_{vl,max} = 0.75 \times d = \mathbf{340}$ mm

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

Design concrete shear stress; $v_c = 0.79\text{N/mm}^2 \times \min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times \max(1, (400\text{mm} /d)^{1/4}) \times (\min(f_{cu}, 40\text{N/mm}^2) / 25\text{N/mm}^2)^{1/3} / \gamma_m = \mathbf{0.512}$ N/mm$^2$

Design shear resistance provided; $v_{s,prov} = A_{sv,prov} \times 0.87 \times f_{yv} / b = \mathbf{0.486}$ N/mm$^2$

Design shear stress provided; $v_{prov} = v_{s,prov} + v_c = \mathbf{0.998}$ N/mm$^2$

Design shear resistance; $V_{prov} = v_{prov} \times (b \times d) = \mathbf{135.9}$ kN

***Shear links provided valid between 400 mm and 4600 mm with tension reinforcement of 603 mm$^2$***

**Spacing of reinforcement (cl 3.12.11)**

Actual distance between bars in tension; $s = (b - 2 \times (c_{nom\_s} + \phi_v + \phi_{bot}/2)) /(N_{bot} - 1) - \phi_{bot} = \mathbf{88}$ mm

**Minimum distance between bars in tension (cl 3.12.11.1)**

Minimum distance between bars in tension; $s_{min} = h_{agg} + 5\text{ mm} = \mathbf{25}$ mm

***PASS - Satisfies the minimum spacing criteria***

**Maximum distance between bars in tension (cl 3.12.11.2)**

Design service stress; $f_s = (2 \times f_y \times A_{s,req}) / (3 \times A_{s,prov} \times \beta_b) = \mathbf{196.7}$ N/mm$^2$

Maximum distance between bars in tension; $s_{max} = \min(47000 \text{ N/mm} / f_s, 300 \text{ mm}) = \mathbf{239}$ mm

***PASS - Satisfies the maximum spacing criteria***

**Span to depth ratio (cl. 3.4.6)**

Basic span to depth ratio (Table 3.9); $span\_to\_depth_{basic} = \mathbf{20.0}$

Design service stress in tension reinforcement; $f_s = (2 \times f_y \times A_{s,req})/ (3 \times A_{s,prov} \times \beta_b) = \mathbf{196.7}$ N/mm$^2$

Modification for tension reinforcement

$f_{tens} = \min(2.0, 0.55 + (477\text{N/mm}^2 - f_s) / (120 \times (0.9\text{N/mm}^2 + (M / (b \times d^2))))) = \mathbf{1.729}$

Modification for compression reinforcement

$f_{comp} = \min(1.5, 1 + (100 \times A_{s2,prov} / (b \times d)) / (3 + (100 \times A_{s2,prov} / (b \times d)))) = \mathbf{1.052}$

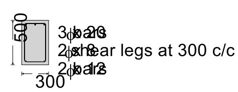
Modification for span length; $f_{long} = 1.000$

Allowable span to depth ratio; $span\_to\_depth_{allow} = span\_to\_depth_{basic} \times f_{tens} \times f_{comp} = \mathbf{36.4}$

Actual span to depth ratio; $span\_to\_depth_{actual} = L_{s1} / d = \mathbf{11.0}$

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

**<u>Support B</u>**




**Rectangular section in flexure (cl.3.4.4)**

Design bending moment; $M = \text{abs}(M_{B\_red}) = \mathbf{134}$ kNm

Depth to tension reinforcement; $d = h - c_{nom\_t} - \phi_v - \phi_{top} / 2 = \mathbf{452}$ mm

Redistribution ratio; $\beta_b = \min(1 - m_{rB}, 1) = \mathbf{1.000}$

$K = M / (b \times d^2 \times f_{cu}) = \mathbf{0.073}$

$K' = 0.156$

***K' > K - No compression reinforcement is required***

Lever arm; $z = \min(d \times (0.5 + (0.25 - K / 0.9)^{0.5}), 0.95 \times d) = \mathbf{412}$ mm

Depth of neutral axis; $x = (d - z) / 0.45 = \mathbf{89}$ mm

Area of tension reinforcement required; $A_{s,req} = M / (0.87 \times f_y \times z) = \mathbf{745}$ mm$^2$

Tension reinforcement provided; $3 \times 20\phi$ bars

Area of tension reinforcement provided; $A_{s,prov} = \mathbf{942}$ mm$^2$

Minimum area of reinforcement; $A_{s,min} = 0.0013 \times b \times h = \mathbf{195}$ mm$^2$

Maximum area of reinforcement; $A_{s,max} = 0.04 \times b \times h = \mathbf{6000}$ mm$^2$

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

**Rectangular section in shear**

Design shear force span 1 at 4548 mm; $V = abs(min(V_{B\_s1\_max}, V_{B\_s1\_red}))$ = **131** kN

Design shear stress; $v = V / (b \times d)$ = **0.968** N/mm$^2$

Design concrete shear stress; $v_c = 0.79 \times min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times max(1, (400 /d)^{1/4}) \times (min(f_{cu}, 40) / 25)^{1/3} / \gamma_m$

$v_c$ = **0.595** N/mm$^2$

Allowable design shear stress; $v_{max} = min(0.8 N/mm^2 \times (f_{cu}/1 N/mm^2)^{0.5}, 5 N/mm^2)$ = **4.382** N/mm$^2$

***PASS - Design shear stress is less than maximum allowable***

Value of v from Table 3.7; $0.5 \times v_c < v < (v_c + 0.4 N/mm^2)$

Design shear resistance required; $v_s = max(v - v_c, 0.4 N/mm^2)$ = **0.400** N/mm$^2$

Area of shear reinforcement required; $A_{sv,req} = v_s \times b / (0.87 \times f_{yv})$ = **276** mm$^2$/m

Shear reinforcement provided; $2 \times 8\phi$ legs at 300 c/c

Area of shear reinforcement provided; $A_{sv,prov}$ = **335** mm$^2$/m

***PASS - Area of shear reinforcement provided exceeds minimum required***

Maximum longitudinal spacing; $s_{vl,max} = 0.75 \times d$ = **339** mm

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

**Spacing of reinforcement (cl 3.12.11)**

Actual distance between bars in tension; $s = (b - 2 \times (c_{nom\_s} + \phi_v + \phi_{top}/2)) /(N_{top} - 1) - \phi_{top}$ = **82** mm

**Minimum distance between bars in tension (cl 3.12.11.1)**

Minimum distance between bars in tension; $s_{min} = h_{agg} + 5$ mm = **25** mm

***PASS - Satisfies the minimum spacing criteria***

**Maximum distance between bars in tension (cl 3.12.11.2)**

Design service stress; $f_s = (2 \times f_y \times A_{s,req}) / (3 \times A_{s,prov} \times \beta_b)$ = **263.6** N/mm$^2$

Maximum distance between bars in tension; $s_{max} = min(47000 N/mm / f_s, 300 mm)$ = **178** mm

***PASS - Satisfies the maximum spacing criteria***

;

# BS RC PARKING COLUMN

## RC COLUMN DESIGN (BS8110:PART1:1997)

TEDDS calculation version 2.0.08

**Column definition**

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

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

Depth to tension steel; $h' = h - c_h - L_{dia} - D_{col}/2$ = **480** mm

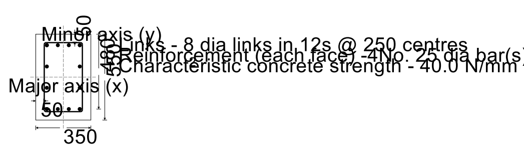
Column width (smaller column dim); $b$ = **350** mm

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

Depth to tension steel; $b' = b - c_b - L_{dia} - D_{col}/2$ = **280** mm

Characteristic strength of reinforcement; $f_y$ = **500** N/mm$^2$

Characteristic strength of concrete; $f_{cu}$ = **40** N/mm$^2$




**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; $l_o$ = **3000** mm

Slenderness limit; $l_{limit} = 60 \times b$ = **21000** mm

Slenderness limit; $l_{limit1} = 100 \times b^2 / h$ = **22273** mm

***Column slenderness limit OK***

***If column is unrestrained, slenderness limit OK***

**Short column check for unbraced columns**

Column clear height; $l_o$ = **3000** mm

Effect height factor for unbraced columns - maj axis; $\beta_x$ = **1.20**

BS8110:Table 3.20

Effective height (major axis); $l_{ex} = \beta_x \times l_o$ = **3.600** m

Slenderness check; $l_{ex}/h$ = **6.55**

***The unbraced column is short (major axis)***

Effect height factor for unbraced columns - min axis; $\beta_y$ = **1.20**

BS8110:Table 3.20

Effective height (minor axis); $l_{ey} = \beta_y \times l_o$ = **3.600** m

Slenderness check; $l_{ey}/b$ = **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; $D_{col}$ = **25** mm

Assumed no. of bars in one face (assumed sym); $L_{ncol}$ = **4**

Area of "tension" steel; $A_{st} = L_{ncol} \times \pi \times D_{col}^2 / 4$ = **1963** mm$^2$

Area of compression steel; $A_{sc} = A_{st}$ = **1963** mm$^2$

Total area of steel ; $A_{scol} = \pi \times D_{col}^2 / 4 \times 2 \times (L_{ncol} + (L_{ncol} - 2))$ = **5890.5** mm$^2$

Percentage of steel; $A_{scol} / (b \times h)$ = **3.1** %

**Design ultimate loading**

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

Initial end moment (major axis); $M_{x1}$ = **100** kNm

Initial end moment (minor axis); $M_{y1}$ = **100** kNm

Initial moment approx (major axis); $M_{ix}$ =abs( $M_{x1}$ ) = **100.0** kNm

Initial moment approx (minor axis); $M_{iy}$ = abs( $M_{y1}$ ) = **100.0** kNm

**Additional moment**

$\beta_{ay} = l_{ey}^2 / (2000 \times b^2)$ = **0.05**

Reduction factor to correct deflection for axial load

$N_{uz} = 0.45 \times f_{cu} \times h \times b + 1/\gamma_{ms} \times f_y \times A_{scol}$ = **6026.1** kN

$N_{bal} = 0.25 \times f_{cu} \times h \times b'$ = **1537.3** kN

$K = \min((N_{uz} - N)/(N_{uz} - N_{bal}), 1.0)$ = **0.90**

$a_{uy} = \beta_{ay} \times K \times b$ = **16.6** mm

Additional moment; $M_{addy} = N \times a_{uy}$ = **33.2** kNm

**Minimum design moments**

Min design moment (Major axis); $M_{xmin} = \min(0.05 \times h, 20 \text{ mm}) \times N$ = **40.0** kNm

Min design moment (Minor axis); $M_{ymin} = \min(0.05 \times b, 20 \text{ mm}) \times N$ = **35.0** kNm

**Design moments**

Design moment (major axis); $M_{xdes} = \max(M_{ix}, M_{xmin})$ = **100.0** kNm

Design moment (minor axis); $M_{ydes} = \max(M_{iy} + M_{addy}, M_{ymin})$ = **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)

$\beta = 1 - 1.165 \times \min(0.6, N/(b \times h \times f_{cu}))$ = **0.70**

Design moment;

$M_{design} = if(M_{xdes}/h' < M_{ydes}/b' , M_{ydes} + \beta \times b'/h' \times M_{xdes} , M_{xdes} + \beta \times h'/b' \times M_{ydes} )$ = **173.9** kNm

**Set up section dimensions for design:-**

Section depth; $D = if(M_{xdes}/h' < M_{ydes}/b' , b, h)$ = **350.0** mm

Depth to "tension" steel; $d = if(M_{xdes}/h' < M_{ydes}/b' , b', h')$ = **279.5** mm

Section width; $B = if(M_{xdes}/h' < M_{ydes}/b' , h, b)$ = **550.0** mm

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

**Check of design forces - symmetrically reinforced section**

**<u>NOTE</u>**

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

***Compression steel yields (0.9x<D)***

**Determine correct moment of resistance**

$N_R$ =ceiling(if($x_{calc}$<D/0.9, $N_{R1}$ , $N_{R2}$ ),0.001kN) = **2000.0** kN

$M_R$ = ceiling(if($x_{calc}$<D/0.9, $M_{R1}$ , $M_{R2}$ ) ,0.001kNm) = **301.0** kNm

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

Check for moment; $M_{design}$ = **173.9** kNm

***Moment check satisfied***

**Check min and max areas of steel**

Total area of concrete; $A_{conc} = b \times h$ = **192500** mm$^2$

Area of steel (symmetrical); $A_{scol}$ = **5890** mm$^2$

Minimum percentage of compression reinforcement; $k_c$ = **0.40** %

Minimum steel area; $A_{sc\_min} = k_c \times A_{conc}$ = **770** mm$^2$

Maximum steel area; $A_{s\_max} = 6\ \% \times A_{conc}$ = **11550** mm$^2$

***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; $A_{conc} = b \times h$ = **192500** mm$^2$

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

Characteristic strength of concrete; $f_{cu}$ = **40** N/mm$^2$

**Is a check required? (3.8.4.6)**

Axial load; N = **2000.0** kN

Major axis moment; $M_x$ = **173.9** kNm

Eccentricity; $e = M_x / N$ = **86.9** mm

Limit to eccentricity; $e_{limit} = 0.6 \times h =$ **330.0** mm

Actual shear stress; $v_x = V_x / (b \times h') =$ **0.4** N/mm$^2$

Allowable stress; $v_{allowable} = \min((0.8 \text{ N}^{1/2}\text{/mm}) \times \sqrt{(f_{cu})}, 5 \text{ N/mm}^2) =$ **5.000** N/mm$^2$

***No shear check required***

**Define Containment Links Provided**

Link spacing; $s_v$ = **250** mm; Link diameter; $L_{dia}$ = **8** mm; No of links in each group; $L_n$ = **12**

**Minimum Containment Steel (Cl 3.12.7)**

**Shear steel**

Link spacing; $s_v$ = **250** mm

Link diameter; $L_{dia}$ = **8** mm

**Column steel**

Diameter; $D_{col}$ = **25** mm

Min diameter; $L_{limit} = \max((6 \text{ mm}), D_{col}/4) =$ **6.3** mm

***Link diameter OK***

Max spacing; $s_{limit} = 12 \times D_{col} =$ **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; $A_c = h \times b =$ **192500** mm$^2$

Limit for crack check; $N_{limit} = 0.2 \times f_{cu} \times A_c =$ **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; $M_{X\_SLS}$ = **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; $f_{cu}$ = **40** N/mm$^2$

Characteristic strength of reinforcement; $f_y$ = **500** N/mm$^2$

BS8110:Pt 1:Table 3.1

Diameter of links; $L_{dia}$ = **8** mm

Diameter of tension reinforcement; $D_{col}$ = **25** mm

Number of tension reinforcement bars; $L_{ncol}$ = **4**

Area of tension reinforcement; $A_{st} = \pi \times D_{col}^2 / 4 \times L_{ncol} =$ **1963** mm$^2$

Nominal cover to reinforcement; $c_{nom} = h - h' - D_{col}/2 - L_{dia} =$ **50** mm

Cover to tension reinforcement; $c_{ten} = c_{nom} + L_{dia} =$ **58.0** mm

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

Tension bar centres; $bar_{crs} = (b - 2\times(c_{nom} + L_{dia}) - D_{col}) / (L_{ncol} - 1) =$ **69.7** mm

**Modular Ratio**

Modulus of elasticity for reinforcement; $E_s = 200$ kN/mm$^2$

BS8110:Pt 1:Cl 2.5.4

Modulus of elast for conc (half the instanteneous); $E_c = ((20 \text{ kN/mm}^2) + 200\times f_{cu}) / 2 =$ **14** kN/mm$^2$

BS8110:Pt 2:Equation 17

Modular ratio; $m = E_s / E_c =$ **14.286**

**Neutral Axis position**

For equilibrium; $F_{st}$ equates $F_c$

Therefore: $m \times A_{st} \times [\, f_c\times(h'-x)/x \,]$ equates to $0.5 \times f_c \times b \times x$

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

$x = h' \times [\, -1\times E_s\times A_{st}/(E_c\times b\times h') + \sqrt{(E_s\times A_{st}/(E_c\times b\times h') \times (2+E_s\times A_{st}/(E_c\times b\times h')))}\,] =$ **208.4** mm

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

**Concrete and Steel stresses**

The serviceability limit state moment; $M_{X\_SLS} =$ **50** kNm

Taking moments about the centreline of the reinforcement:-

Moment of resistance of concrete is $0.5 \times f_c \times b \times x \times (h' - x/3)$

Solving for concrete stress $f_c$ gives; $f_c = 2 \times M_{X\_SLS} / (b \times x \times (h' - x/3)) =$ **3.34** N/mm$^2$

Allowable stress; $0.45 \times f_{cu} =$ **18.00** N/mm$^2$

***Concrete stress OK***

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

Moment of resistance of steel is $f_{st} \times A_s \times (h' - x/3)$

Solving for steel stress $f_{st}$ gives; $f_{st} = M_{X\_SLS} / (A_{st} \times (h' - x/3)) =$ **62.11** N/mm$^2$

**Concrete and Steel strains**

Strain in the reinforcement; $\varepsilon_s = f_{st} / E_s =$ **$310.5\times10^{-6}$**

Allowable steel strain; $0.8 \times f_y / E_s =$ **$2.000\times10^{-3}$**

***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

$\varepsilon_1 = \varepsilon_s \times (a' - x)/(h' - x) =$ **$391.3\times10^{-6}$**

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

Allowable crack width; $Crack_{Allowable} =$ **2.0** mm

BS8007:Cl 2.2.3.3

Factor for stiffening based on limitiing crack width; factor = **1.0** N/mm$^2$

Breadth of tension face; $b_t = b =$ **350** mm

$\varepsilon_m = \min(\varepsilon_1, \max(0, \varepsilon_1 - [\, factor \times b_t\times (h - x) \times (a' - x) / (3\times E_s\times A_{st} \times(h' - x))])) =$ **$263.4\times10^{-6}$**

BS8007:App B.3

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

$a_{cr1} = \sqrt{((bar_{crs}/2)^2 + (c_{nom} + L_{dia} + D_{col}/2)^2)} - D_{col}/2 =$ **66.1** mm

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

$a_{cr2} = \sqrt{(2)} \times (c_{nom} + L_{dia} + D_{col}/2) - D_{col}/2 =$ **87.2** mm

Critical distance from tension bar; $a_{cr} = \max(a_{cr1}, a_{cr2}) =$ **87.2** mm

Design crack width; $Crack_{design} = 3 \times a_{cr} \times \varepsilon_m /(1 + 2\times(a_{cr} - c_{ten})/(h - x)) =$ **0.059** mm

BS8007:App B.2

Max allowable crack width; $Crack_{Allowable} =$ **2.00** mm

BS8007:Cl 2.2.3.3

**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; $M_{Y\_SLS} =$ **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; $f_{cu} =$ **40** N/mm$^2$

Characteristic strength of reinforcement; $f_y =$ **500** N/mm$^2$

BS8110:Pt 1:Table 3.1

Diameter of links; $L_{dia} =$ **8** mm

Diameter of tension reinforcement; $D_{col} =$ **25** mm

Number of tension reinforcement bars; $L_{ncol} =$ **4**

Area of tension reinforcement; $A_{st} = \pi \times D_{col}^2 /4 \times L_{ncol} =$ **1963** mm$^2$

Nominal cover to reinforcement; $c_{nom} = b - b' - D_{col}/2 - L_{dia} =$ **50** mm

Cover to tension reinforcement; $c_{ten} = c_{nom} + L_{dia} =$ **58.0** mm

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

Tension bar centres; $bar_{crs} = (h - 2\times(c_{nom} + L_{dia}) - D_{col}) / (L_{ncol} - 1) =$ **136.3** mm

**Modular Ratio**

Modulus of elasticity for reinforcement; $E_s = 200$ kN/mm$^2$

BS8110:Pt 1:Cl 2.5.4

Modulus of elasticity for conc (half instanteneous); $E_c = ((20\text{ kN/mm}^2) + 200\times f_{cu}) / 2 =$ **14** kN/mm$^2$

BS8110:Pt 2:Equation 17

Modular ratio; $m = E_s / E_c =$ **14.286**

**Neutral Axis position**

For equilibrium; $F_{st}$ equates $F_c$

Therefore: $m \times A_{st} \times [ f_c\times(b'-x)/x ]$ equates to $0.5 \times f_c \times h \times x$

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

$x = b' \times [ -1\times E_s\times A_{st}/(E_c\times h\times b') + \sqrt{( E_s\times A_{st}/(E_c\times h\times b') \times (2+E_s\times A_{st}/(E_c\times h\times b')))}] =$ **125.4** mm

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

**Concrete and Steel stresses**

The serviceability limit state moment; $M_{Y\_SLS} =$ **50** kNm

Taking moments about the centreline of the reinforcement:-

Moment of resistance of concrete is $0.5 \times f_c \times h \times x \times (b' - x/3)$

Solving for concrete stress $f_c$ gives; $f_c = 2 \times M_{Y\_SLS} / ( h \times x \times (b' - x/3)) =$ **6.10** N/mm$^2$

Allowable stress; $0.45 \times f_{cu} =$ **18.00** N/mm$^2$

***Concrete stress OK***

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

Moment of resistance of steel; $f_{st} \times A_s \times (b' - x/3)$

Solving for steel stress $f_{st}$ gives; $f_{st} = M_{Y\_SLS} / ( A_{st} \times (b' - x/3)) =$ **107.13** N/mm$^2$

**Concrete and Steel strains**

Strain in the reinforcement; $\varepsilon_s = f_{st} / E_s =$ **535.6×10$^{-6}$**

Allowable steel strain; $0.8 \times f_y / E_s =$ **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

$\varepsilon_1 = \varepsilon_s \times (a' - x)/(b' - x) =$ **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; $Crack_{Allowable} =$ **2.0** mm

BS8007:Cl 2.2.3.3

Factor for stiffening based on limitiing crack width; factor = **1.0** N/mm$^2$

Breadth of tension face; $h_t = h =$ **550** mm

$\varepsilon_m = \min( \varepsilon_1 , \max(0, \varepsilon_1 - [ factor \times h_t \times (b - x) \times (a' - x) / (3 \times E_s \times A_{st} \times (b' - x))])) =$ **627.8×10$^{-6}$**

BS8007:App B.3

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

$a_{cr1} = \sqrt{( (bar_{crs}/2)^2 + (c_{nom} + L_{dia} + D_{col}/2)^2)} - D_{col}/2 =$ **85.6** mm

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

$a_{cr2} = \sqrt{(2)} \times (c_{nom} + L_{dia} + D_{col}/2 ) - D_{col}/2 =$ **87.2** mm

Critical distance from tension bar; $a_{cr} = \max(a_{cr1}, a_{cr2} ) =$ **87.2** mm

Design crack width; $Crack_{design} = 3 \times a_{cr} \times \varepsilon_m /(1 + 2 \times (a_{cr} - c_{ten})/(b - x)) =$ **0.130** mm

BS8007:App B.2

Max allowable crack width; $Crack_{Allowable} =$ **2.00** mm

BS8007:Cl 2.2.3.3

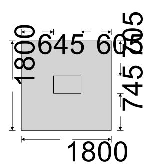
;

Kelly Mweu Changilwa
+254703206460
mweukelly@gmail.com


# BS PAD FOOTING

## PAD FOOTING ANALYSIS AND DESIGN (BS8110-1:1997)

Tedds calculation version 2.0.07




**Pad footing details**

Length of pad footing; $L = \mathbf{1800}$ mm

Width of pad footing; $B = \mathbf{1800}$ mm

Area of pad footing; $A = L \times B = \mathbf{3.240}$ $m^2$

Depth of pad footing; $h = \mathbf{1800}$ mm

Depth of soil over pad footing; $h_{soil} = \mathbf{700}$ mm

Density of concrete; $\rho_{conc} = \mathbf{24.5}$ $kN/m^3$

**Column details**

Column base length; $l_A = \mathbf{550}$ mm

Column base width; $b_A = \mathbf{350}$ mm

Column eccentricity in x; $e_{PxA} = \mathbf{20}$ mm

Column eccentricity in y; $e_{PyA} = \mathbf{20}$ mm

**Soil details**

Density of soil; $\rho_{soil} = \mathbf{20.0}$ $kN/m^3$

Design shear strength; $\phi' = \mathbf{25.0}$ deg

Design base friction; $\delta = \mathbf{19.3}$ deg

Allowable bearing pressure; $P_{bearing} = \mathbf{150}$ $kN/m^2$

**Foundation loads**

Dead surcharge load; $F_{Gsur} = \mathbf{0.000}$ $kN/m^2$

Imposed surcharge load; $F_{Qsur}$ = **0.000** kN/m$^2$

Pad footing self weight; $F_{swt} = h \times \rho_{conc}$ = **44.100** kN/m$^2$

Soil self weight; $F_{soil} = h_{soil} \times \rho_{soil}$ = **14.000** kN/m$^2$

Total foundation load; $F = A \times (F_{Gsur} + F_{Qsur} + F_{swt} + F_{soil})$ = **188.2** kN

**Calculate pad base reaction**

Total base reaction; $T = F + P_A$ = **188.2** kN

Eccentricity of base reaction in x; $e_{Tx} = (P_A \times e_{PxA} + M_{xA} + H_{xA} \times h) / T$ = **0** mm

Eccentricity of base reaction in y; $e_{Ty} = (P_A \times e_{PyA} + M_{yA} + H_{yA} \times h) / T$ = **0** mm

**Check pad base reaction eccentricity**

$abs(e_{Tx}) / L + abs(e_{Ty}) / B$ = **0.000**

***Base reaction acts within middle third of base***

**Calculate pad base pressures**

$q_1 = T / A - 6 \times T \times e_{Tx} / (L \times A) - 6 \times T \times e_{Ty} / (B \times A)$ = **58.100** kN/m$^2$

$q_2 = T / A - 6 \times T \times e_{Tx} / (L \times A) + 6 \times T \times e_{Ty} / (B \times A)$ = **58.100** kN/m$^2$

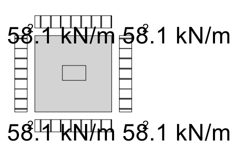
$q_3 = T / A + 6 \times T \times e_{Tx} / (L \times A) - 6 \times T \times e_{Ty} / (B \times A)$ = **58.100** kN/m$^2$

$q_4 = T / A + 6 \times T \times e_{Tx} / (L \times A) + 6 \times T \times e_{Ty} / (B \times A)$ = **58.100** kN/m$^2$

Minimum base pressure; $q_{min} = min(q_1, q_2, q_3, q_4)$ = **58.100** kN/m$^2$

Maximum base pressure; $q_{max} = max(q_1, q_2, q_3, q_4)$ = **58.100** kN/m$^2$

***PASS - Maximum base pressure is less than allowable bearing pressure***




**Partial safety factors for loads**

Partial safety factor for dead loads; $\gamma_{fG}$ = **1.40**

Partial safety factor for imposed loads; $\gamma_{fQ}$ = **1.60**

Partial safety factor for wind loads; $\gamma_{fW}$ = **0.00**

**Ultimate axial loading on column**

Ultimate axial load on column; $P_{uA} = P_{GA} \times \gamma_{fG} + P_{QA} \times \gamma_{fQ} + P_{WA} \times \gamma_{fW}$ = **0.0** kN

**Ultimate foundation loads**

Ultimate foundation load; $F_u = A \times [(F_{Gsur} + F_{swt} + F_{soil}) \times \gamma_{fG} + F_{Qsur} \times \gamma_{fQ}]$ = **263.5** kN

**Ultimate horizontal loading on column**

Ultimate horizontal load in x direction; $H_{xuA} = H_{GxA} \times \gamma_{fG} + H_{QxA} \times \gamma_{fQ} + H_{WxA} \times \gamma_{fW}$ = **0.0** kN

Ultimate horizontal load in y direction; $H_{yuA} = H_{GyA} \times \gamma_{fG} + H_{QyA} \times \gamma_{fQ} + H_{WyA} \times \gamma_{fW}$ = **0.0** kN

**Ultimate moment on column**

Ultimate moment on column in x direction; $M_{xuA} = M_{GxA} \times \gamma_{fG} + M_{QxA} \times \gamma_{fQ} + M_{WxA} \times \gamma_{fW}$ = **0.000** kNm

Ultimate moment on column in y direction; $M_{yuA} = M_{GyA} \times \gamma_{fG} + M_{QyA} \times \gamma_{fQ} + M_{WyA} \times \gamma_{fW}$ = **0.000** kNm

**Calculate ultimate pad base reaction**

Ultimate base reaction; $T_u = F_u + P_{uA}$ = **263.5** kN

Eccentricity of ultimate base reaction in x; $e_{Txu} = (P_{uA} \times e_{PxA} + M_{xuA} + H_{xuA} \times h) / T_u$ = **0** mm

Eccentricity of ultimate base reaction in y; $e_{Tyu} = (P_{uA} \times e_{PyA} + M_{yuA} + H_{yuA} \times h) / T_u$ = **0** mm

**Calculate ultimate pad base pressures**

$q_{1u} = T_u/A - 6\times T_u\times e_{Txu}/(L\times A) - 6\times T_u\times e_{Tyu}/(B\times A)$ = **81.340** kN/m$^2$

$q_{2u} = T_u/A - 6\times T_u\times e_{Txu}/(L\times A) + 6\times T_u\times e_{Tyu}/(B\times A)$ = **81.340** kN/m$^2$

$q_{3u} = T_u/A + 6\times T_u\times e_{Txu}/(L\times A) - 6\times T_u\times e_{Tyu}/(B\times A)$ = **81.340** kN/m$^2$

$q_{4u} = T_u/A + 6\times T_u\times e_{Txu}/(L\times A) + 6\times T_u\times e_{Tyu}/(B\times A)$ = **81.340** kN/m$^2$

Minimum ultimate base pressure; $q_{minu} = \min(q_{1u}, q_{2u}, q_{3u}, q_{4u})$ = **81.340** kN/m$^2$

Maximum ultimate base pressure; $q_{maxu} = \max(q_{1u}, q_{2u}, q_{3u}, q_{4u})$ = **81.340** kN/m$^2$

**Calculate rate of change of base pressure in x direction**

Left hand base reaction; $f_{uL} = (q_{1u} + q_{2u}) \times B / 2$ = **146.412** kN/m

Right hand base reaction; $f_{uR} = (q_{3u} + q_{4u}) \times B / 2$ = **146.412** kN/m

Length of base reaction; $L_x = L$ = **1800** mm

Rate of change of base pressure; $C_x = (f_{uR} - f_{uL}) / L_x$ = **0.000** kN/m/m

**Calculate pad lengths in x direction**

Left hand length; $L_L = L / 2 + e_{PxA}$ = **920** mm

Right hand length; $L_R = L / 2 - e_{PxA}$ = **880** mm

**Calculate ultimate moments in x direction**

Ultimate positive moment in x direction; $M_x = f_{uL} \times L_L^2 / 2 + C_x \times L_L^3 / 6 - F_u \times L_L^2 / (2 \times L)$ = **0.000** kNm

Position of maximum negative moment; $L_z$ = **920** mm

Ultimate negative moment in x direction; $M_{xneg} = f_{uL} \times L_L^2 / 2 + C_x \times L_L^3 / 6 - F_u \times L_L^2 / (2 \times L)$

$M_{xneg}$ = **0.000** kNm

**Calculate rate of change of base pressure in y direction**

Top edge base reaction; $f_{uT} = (q_{2u} + q_{4u}) \times L / 2$ = **146.412** kN/m

Bottom edge base reaction; $f_{uB} = (q_{1u} + q_{3u}) \times L / 2$ = **146.412** kN/m

Length of base reaction; $L_y = B$ = **1800** mm

Rate of change of base pressure; $C_y = (f_{uB} - f_{uT}) / L_y$ = **0.000** kN/m/m

**Calculate pad lengths in y direction**

Top length; $L_T = B / 2 - e_{PyA}$ = **880** mm

Bottom length; $L_B = B / 2 + e_{PyA} =$ **920** mm

**Calculate ultimate moments in y direction**

Ultimate positive moment in y direction; $M_y = f_{uT} \times L_T^2 / 2 + C_y \times L_T^3 / 6 - F_u \times L_T^2 / (2 \times B) =$ **0.000** kNm

Position of maximum negative moment; $L_z =$ **920** mm

Ultimate negative moment in y direction; $M_{yneg} = f_{uT} \times L_T^2 / 2 + C_y \times L_T^3 / 6 - F_u \times L_T^2 / (2 \times B)$

$M_{yneg} =$ **0.000** kNm

**Material details**

Characteristic strength of concrete; $f_{cu} =$ **40** N/mm$^2$

Characteristic strength of reinforcement; $f_y =$ **500** N/mm$^2$

Characteristic strength of shear reinforcement; $f_{yv} =$ **500** N/mm$^2$

Nominal cover to reinforcement; $c_{nom} =$ **50** mm

**Moment design in x direction**

Diameter of tension reinforcement; $\phi_{xB} =$ **25** mm

Depth of tension reinforcement; $d_x = h - c_{nom} - \phi_{xB} / 2 =$ **1738** mm

**Design formula for rectangular beams (cl 3.4.4.4)**

$K_x = M_x / (B \times d_x^2 \times f_{cu}) =$ **0.000**

$K_x' = 0.156$

***$K_x < K_x'$ compression reinforcement is not required***

Lever arm; $z_x = d_x \times \min([0.5 + \sqrt{(0.25 - K_x / 0.9)}], 0.95) =$ **1651** mm

Area of tension reinforcement required; $A_{s\_x\_req} = M_x / (0.87 \times f_y \times z_x) =$ **0** mm$^2$

Minimum area of tension reinforcement; $A_{s\_x\_min} = 0.0013 \times B \times h =$ **4212** mm$^2$

Tension reinforcement provided; **12 No. 25 dia. bars bottom (150 centres)**

Area of tension reinforcement provided; $A_{s\_xB\_prov} = N_{xB} \times \pi \times \phi_{xB}^2 / 4 =$ **5890** mm$^2$

***PASS - Tension reinforcement provided exceeds tension reinforcement required***

**Negative moment design in x direction**

Diameter of tension reinforcement; $\phi_{xT} =$ **12** mm

Depth of tension reinforcement; $d_x = h - c_{nom} - \phi_{xT} / 2 =$ **1744** mm

**Design formula for rectangular beams (cl 3.4.4.4)**

$K_x = -M_{xneg} / (B \times d_x^2 \times f_{cu}) =$ **0.000**

$K_x' = 0.156$

***$K_x < K_x'$ compression reinforcement is not required***

Lever arm; $z_x = d_x \times \min([0.5 + \sqrt{(0.25 - K_x / 0.9)}], 0.95) =$ **1657** mm

Area of tension reinforcement required; $A_{s\_x\_req} = -M_{xneg} / (0.87 \times f_y \times z_x) =$ **0** mm$^2$

Minimum area of tension reinforcement; $A_{s\_x\_min} = 0.0013 \times B \times h =$ **4212** mm$^2$

Tension reinforcement provided; **0 No. 12 dia. bars top**

Area of tension reinforcement provided; $A_{s\_xT\_prov} = N_{xT} \times \pi \times \phi_{xT}^2 / 4 =$ **0** mm$^2$

***FAIL - Tension reinforcement provided is less than tension reinforcement required***

**Moment design in y direction**

Diameter of tension reinforcement; $\phi_{yB} =$ **25** mm

Depth of tension reinforcement; $d_y = h - c_{nom} - \phi_{xB} - \phi_{yB} / 2 =$ **1713** mm

**Design formula for rectangular beams (cl 3.4.4.4)**

$K_y = M_y / (L \times d_y^2 \times f_{cu}) =$ **0.000**

$K_y' = 0.156$

***$K_y < K_y'$ compression reinforcement is not required***

Lever arm; $z_y = d_y \times \min([0.5 + \sqrt{(0.25 - K_y / 0.9)}], 0.95) =$ **1627** mm

Area of tension reinforcement required; $A_{s\_y\_req} = M_y / (0.87 \times f_y \times z_y) =$ **0** mm$^2$

Minimum area of tension reinforcement; $A_{s\_y\_min} = 0.0013 \times L \times h =$ **4212** mm$^2$

Tension reinforcement provided; **12 No. 25 dia. bars bottom (150 centres)**

Area of tension reinforcement provided; $A_{s\_yB\_prov} = N_{yB} \times \pi \times \phi_{yB}^2 / 4 =$ **5890** mm$^2$

***PASS - Tension reinforcement provided exceeds tension reinforcement required***

**Negative moment design in y direction**

Diameter of tension reinforcement; $\phi_{yT} =$ **12** mm

Depth of tension reinforcement; $d_y = h - c_{nom} - \phi_{xT} - \phi_{yT} / 2 =$ **1732** mm

**Design formula for rectangular beams (cl 3.4.4.4)**

$K_y = -M_{yneg} / (L \times d_y^2 \times f_{cu}) =$ **0.000**

$K_y' = 0.156$

***$K_y < K_y'$ compression reinforcement is not required***

Lever arm; $z_y = d_y \times \min([0.5 + \sqrt{(0.25 - K_y / 0.9)}], 0.95) =$ **1645** mm

Area of tension reinforcement required; $A_{s\_y\_req} = -M_{yneg} / (0.87 \times f_y \times z_y) =$ **0** mm$^2$

Minimum area of tension reinforcement; $A_{s\_y\_min} = 0.0013 \times L \times h =$ **4212** mm$^2$

Tension reinforcement provided; **0 No. 12 dia. bars top**

Area of tension reinforcement provided; $A_{s\_yT\_prov} = N_{yT} \times \pi \times \phi_{yT}^2 / 4 =$ **0** mm$^2$

***FAIL - Tension reinforcement provided is less than tension reinforcement required***

**Calculate ultimate punching shear force at face of column**

Ultimate pressure for punching shear; $q_{puA} = q_{1u}+[(L/2+e_{PxA}-l_A/2)+(l_A)/2]\times C_x/B-[(B/2+e_{PyA}-b_A/2)+(b_A)/2]\times C_y/L =$ **81.340** kN/m$^2$

Average effective depth of reinforcement; $d = (d_x + d_y) / 2 =$ **1738** mm

Area loaded for punching shear at column; $A_{pA} = (l_A)\times(b_A) =$ **0.193** m$^2$

Length of punching shear perimeter; $u_{pA} = 2\times(l_A)+2\times(b_A) =$ **1800** mm

Ultimate shear force at shear perimeter; $V_{puA} = P_{uA} + (F_u / A - q_{puA}) \times A_{pA} =$ **0.000** kN

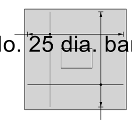
Effective shear force at shear perimeter; $V_{puAeff} = V_{puA} =$ **0.000** kN

**Punching shear stresses at face of column (cl 3.7.7.2)**

Design shear stress; $v_{puA} = V_{puAeff} / (u_{pA} \times d) =$ **0.000** N/mm$^2$

Allowable design shear stress; $v_{max} = \min(0.8\text{N/mm}^2 \times \sqrt{(f_{cu} / 1\text{ N/mm}^2)}, 5\text{ N/mm}^2) =$ **5.000** N/mm$^2$

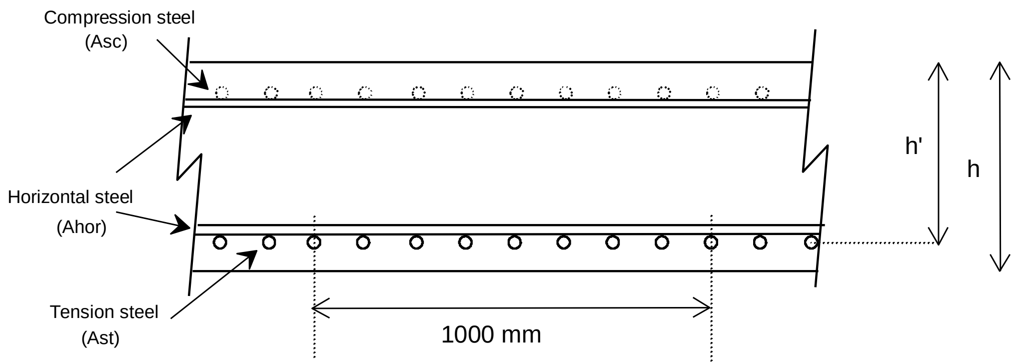
***PASS - Design shear stress is less than allowable design shear stress***



12 No. 25 dia. bars btm (150 c/c)
12 No. 25 dia. bars btm (150 c/c)


;

# BS RC WALL DESIGN




**Wall**
(assumed symmetric)

## 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; $c_w$ = **60** mm

; Trial bar diameter; $D_{try}$ = **25** mm

Depth to tension steel

$h' = h - c_w - D_{try}/2 = \mathbf{227}$ mm

Materials

; Characteristic strength of reinforcement; $f_y$ = **500** N/mm$^2$

; Characteristic strength of concrete; $f_{cu}$ = **35** N/mm$^2$

## UnBraced Wall Design to cl 3.9.3 (Simply supported construction)

**Stocky check for unbraced walls**

; Wall clear height; $l_o$ = **3000** mm

;; Effective height factor for simply supported unbraced walls (assessed for a plain wall)

$\beta = \mathbf{1.00}$

; $l_e = \beta \times l_o = \mathbf{3.000}$ m; $l_e/h = \mathbf{10.00}$

***The unbraced wall is stocky***

**Unbraced wall slenderness check**

Effective wall height; $l_e$ = **3000** mm

Slenderness limit; $l_{limit} = 30 \times h = \mathbf{9000}$ mm

***Wall slenderness limit OK***

Kelly Mweu Changilwa
+254703206460
mweukelly@gmail.com

**Define wall reinforcement**

Main reinforcement in wall

;;

***Provide 20 dia bars @ 250 centres; in each face***

Area of "tension" steel; $A_{st} = A_{svert}$ = **1260** $mm^2/m$

Area of compression steel; $A_{sc} = A_{st}$ = **1260** $mm^2/m$

Total area of steel ; $A_{wall} = A_{st} + A_{sc}$ = **2520.0** $mm^2/m$

;Percentage of steel; $(A_{st} + A_{sc}) / h$ = **0.84** %

**HORIZONTAL WALL STEEL**

; Wall thickness; h = **300** mm

;Area of vertical steel provided; $A_{wall}$ = **2520** $mm^2/m$

Percentage of vertical steel; $p_{vwall} = A_{wall} / h$ = **0.84** %

;Minimum diameter of horizontal steel; $D_{min} = max(D_{vert}/4 , 6 mm)$ = **6** mm

Minimum area of horizontal steel

; $A_{Hmin}$ = If($f_y$>=(460 $N/mm^2$),if($p_{vwall}$>2 %,0.13 %,0.25%),if($p_{vwall}$>2 %,0.24 %, 0.30 %)) × h/2

$A_{Hmin}$ =**375** $mm^2/m$

***No containment links required***

Define horizontal wall steel in one face;

***Provide 8 dia bars @ 125 centres; in each face***

**Stocky wall (simple construction) - transverse bending and axial load**

**Design ultimate loading**

; Design ultimate axial load per m of wall; $n_w$ = **100** kN/m

Design ultimate transverse moment per m of wall; $m_i$ = ;**100.0**; kNm/m

**Minimum design moments**

; $m_{min} = min(0.05 \times h, 20 mm) \times n_w$ = **1.5** kNm/m

**Design moments**

$m_{design} = max(abs(m_i), m_{min})$ = ;**100.0**; kNm/m

**CHECK OF DESIGN FORCES - SYMMETRICALLY REINFORCED WALL SECTION**

**NOTES**

; h is the wall thickness

; h' is the depth from the more highly compressed face to the "tension" steel.

***Tension steel yields***

**Determine correct moment of resistance**

$n_R = \text{if}(x_{calc}<h/0.9,\ n_{R1}\ ,\ n_{R2}\ ) =$ **479.9** kN/m

$m_R = \text{if}(x_{calc}<h/0.9,\ m_{R1}\ ,\ m_{R2}\ ) =$ **163.1** kNm/m

Applied axial load

$n_w =$ **100.0** kN/m

Check for moment

; $m_{design} =$ **100.0** kNm/m

***Moment check satisfied***

***;The wall vertical reinforcement defined in each face is H20 dia bars @ 250 centres***

**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; $A_c$ = h = **300000** mm$^2$/m

$A_{st\_min} = 0.4\% \times A_c =$ **1200** mm$^2$/m

$A_{st\_max} = 4\ \% \times A_c =$ **12000** mm$^2$/m

;Total vertical steel in wall; $A_{wall}$ = **2520** mm$^2$/m

***Area of vertical steel in wall provided OK***

**Horizontal steel**

Percentage of vertical steel; $p_{vwall} = A_{wall} / h =$ **0.84** %

;Diameter of horizontal steel; $D_{hor}$ = **8** mm

;Minimum diameter of horizontal steel; $D_{min} = \max(D_{vert}/4, 6\ \text{mm}) =$ **6** mm

***Diameter of horizontal steel in wall OK***

;Area of horizontal steel in one face; $A_{shor}$ = **402** mm$^2$/m

Minimum area of horizontal steel

; $A_{Hmin} = \text{If}(f_y>=(460\ \text{N/mm}^2),\text{if}(p_{vwall}>2\ \%,0.13\ \%,0.25\%),\text{if}(p_{vwall}>2\ \%,0.24,0.30\ \%)) \times h/2$

$A_{Hmin}$ =**375** mm$^2$/m

***Area of horizontal steel in wall provided OK***

## 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; $A_{conc}$ = h = **300000** mm$^2$/m

; Design ultimate shear force through thickness per m of wall; $v_w$ = **100** kN/m

; Characteristic strength of concrete; $f_{cu}$ = **35** N/mm$^2$

**Is a check required? (3.8.4.6)**

; Axial load per m of wall; $n_w$ = **100.0** kN/m

; Major axis moment per m of wall; $m_w$ = **100.0** kNm/m

$e = m_w / n_w$ = **1000.0** mm

$e_{limit} = 0.6 \times h$ = **180.0** mm

Actual shear stress; $v_x = v_w / h'$ = **0.4** N/mm$^2$

Allowable stress; $v_{allowable} = \min((0.8 N^{1/2}/mm) \times \sqrt{(f_{cu})}, 5 N/mm^2)$ = **4.733** N/mm$^2$

***Shear check required***

**Design shear stress to clause 3.4.5.12**

; $f_{cu\_ratio} = \text{if } (f_{cu} > 40 N/mm^2, 40/25, f_{cu}/(25 N/mm^2))$ = **1.400**

Design concrete shear stress

;; $v_c = 0.79 N/mm^2 \times \min(3, 100 \times A_{st} / h')^{1/3} \times \max(1, (400 mm) / h')^{1/4} / 1.25 * f_{cu\_ratio}^{1/3}$

; $v_c$ = **0.669** N/mm$^2$

;;; $v_c' = v_c + 0.6 \times n_w / h \times \min(\text{abs}(v_w) \times h / m_w, 1.0)$ = **0.7** N/mm$^2$

;$v_{allowable} = \min((0.8 N^{1/2}/mm) \times \sqrt{(f_{cu})}, v_c', 5 N/mm^2)$ = **0.729** N/mm$^2$

Actual shear stress

$v_x$ = **0.4** N/mm$^2$

***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; $D_{vert}$ = **20** mm

; Diameter of links; $L_{dia}$ = **8** mm

Cover to tension reinforcement

$c_{ten} = h - h' - D_{vert} / 2$ = **62.5** mm

Nominal cover to links steel

$c_{nom} = c_{ten} - L_{dia}$ = **54.5** mm

Permissable minimum nominal cover to all reinforcement (Table 3.4)

; $c_{min}$ = **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; $m_{SLS}$ = **100** kNm/m

; Wall thickness; h = **300** mm

; Depth to steel; h' = **227** mm

; Characteristic strength of concrete; $f_{cu}$ = **35** N/mm$^2$

; Characteristic strength of reinforcement; $f_y$ = **500** N/mm$^2$

BS8110:Pt 1:Table 3.1

; Diameter of wall vertical reinforcement; $D_{vert}$ = **20** mm

; Spacing of vertical reinforcement bars; $s_{vert}$ = **250** mm

Area of vertical reinforcement in one face; $A_{st} = \pi \times D_{vert}^2 /4 / s_{vert}$ = **1257** mm$^2$/m

Effective depth to tension reinforcement

h' = **227.5** mm

Cover to tension reinforcement

$c_{ten}$ = h - h' - $D_{vert}$/2 = **63** mm

Nominal cover to tension reinforcement

$c_{nom} = c_{ten}$ = **62.5** mm

Tension bar centres

$bar_{crs} = s_{vert}$ = **250.0** mm

**MODULAR RATIO**

Modulus of elasticity for reinforcement; $E_s$ = 200 kN/mm$^2$

BS8110:Pt 1:Cl 2.5.4

Modulus of elasticity for concrete (half the instanteneous)

$E_c$ = ((20 kN/mm$^2$) + 200×$f_{cu}$) / 2 = **14** kN/mm$^2$

BS8110:Pt 2:Equation 17

Modular ratio; $m = E_s / E_c$ = **14.815**

**NEUTRAL AXIS POSITION**

For equilibrium; $F_{st}$ equates $F_c$

Therefore: $m \times A_{st} \times [\, f_c \times (h'-x)/x \,]$ equates to $0.5 \times f_c \times x$

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

$x = h' \times [\, -1 \times E_s \times A_{st}/(E_c \times h') + \sqrt{(E_s \times A_{st}/(E_c \times h') \times (2+E_s \times A_{st}/(E_c \times h')))}\,]$ = **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; $m_{SLS}$ = **100** kNm/m

Taking moments about the centreline of the reinforcement:-

Moment of resistance of concrete is $0.5 \times f_c \times x \times (h' - x/3)$

Solving for concrete stress $f_c$ gives;

$f_c = 2 \times m_{SLS} / (x \times (h' - x/3))$ = **13.13** N/mm$^2$

Allowable stress; $0.45 \times f_{cu}$ = **15.75** N/mm$^2$

***Concrete stress OK***

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

Moment of resistance of steel is $f_{st} \times A_s \times (h' - x/3)$

Solving for steel stress $f_{st}$ gives;

$f_{st} = m_{SLS} / ( A_{st} \times (h' - x/3))$ = **393.16** N/mm$^2$

**CONCRETE AND STEEL STRAINS**

Strain in the reinforcement

$\varepsilon_s = f_{st} / E_s$ = **1.966×10$^{-3}$**

Allowable steel strain; $0.8 \times f_y / E_s$ = **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

$\varepsilon_1 = \varepsilon_s \times (a' - x)/(h' - x)$ = **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; $Crack_{Allowable}$ = **1.0** mm

BS8007:Cl 2.2.3.3

Factor for stiffening based on limiting crack width

factor = if($Crack_{Allowable}$ == (0.2 mm), (1.0 N/mm$^2$), (1.5 N/mm$^2$)) = **2** N/mm$^2$

$\varepsilon_m = \min( \varepsilon_1 ,\max(0, \varepsilon_1 - [ \text{factor} \times (h - x) \times (a' - x) / (3\times E_s\times A_{st} \times(h' - x))]))$ = **2.242×10$^{-3}$**

BS8007:Cl 2.2.3.3

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

$a_{cr} = \sqrt{ (bar_{crs}/2)^2 + (c_{nom} + D_{vert}/2)^2} - D_{vert}/2$ = **134.5** mm

Design crack width

$Crack_{design} = 3 \times a_{cr} \times \varepsilon_m /(1 + 2\times(a_{cr} - c_{ten})/(h - x))$ = **0.551** mm

BS8007:App B.3

Max allowable crack width

$Crack_{Allowable}$ = **1.00** mm

BS8007:Cl 2.2.3.3

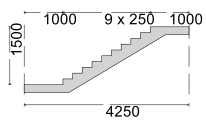
***Design Crack width OK***

;
;
;

# BS RC STAIR DESIGN

## RC STAIR DESIGN (BS8110-1:1997)

TEDDS calculation version 1.0.05




**Stair geometry**

Number of steps; $N_{steps}$ = **10**

Waist depth for stair flight; $h_{span}$ = **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

**Upper landing geometry**

Support condition; **Simply supported**

Landing length; $L_{upper}$ = **1000** mm

Depth of landing; $h_{upper}$ = **200** mm

**Lower landing geometry**

Support condition; **Simply supported**

Landing length; $L_{lower}$ = **1000** mm

Depth of landing; $h_{lower}$ = **200** mm

**Material details**

Characteristic strength of concrete; $f_{cu}$ = **40** N/mm$^2$

Characteristic strength of reinforcement; $f_y$ = **500** N/mm$^2$

Nominal cover to reinforcement; $c_{nom}$ = **25** mm

Density of concrete; $\gamma_{conc}$ = **24.5** kN/m$^3$

**Partial safety factors**

Partial safety factor for imposed loading; $\gamma_{fq}$ = **1.60**

Partial safety factor for dead loading; $\gamma_{fg}$ = **1.40**

Kelly Mweu Changilwa
+254703206460
mweukelly@gmail.com

**Loading details**

Characteristic imposed loading; $q_k$ = **3.000** kN/m$^2$

Characteristic loading from finishes; $g_{k\_fin}$ = **1.200** kN/m$^2$

Average stair self weight; $g_{k\_swt} = (h_{span} / \mathrm{Cos(Rake)} + \mathrm{Rise} / 2) \times \gamma_{conc}$ = **8.980** kN/m$^2$

Design load; $F = (g_{k\_swt} + g_{k\_fin}) \times \gamma_{fg} + q_k \times \gamma_{fq}$ = **19.053** kN/m$^2$

**Mid span design**

Midspan moment per metre width; $M_{span} = 0.125 \times F \times L^2$ = **43.017** kNm/m

Diameter of tension reinforcement; $\phi_{span}$ = **12** mm

Depth of reinforcement; $d_{span} = h_{span} - c_{nom} - \phi_{span} / 2$ = **219** mm

**Design formula for rectangular beams (cl 3.4.4.4)**

Moment redistribution ratio; $\beta_b$ = **1.00**

$K_{span} = M_{span} / (d_{span}^2 \times f_{cu})$ = **0.022**

$K'_{span}$ = **0.156**

***$K_{span} < K'_{span}$ compression reinforcement is not required***

Lever arm; $z_{span} = d_{span} \times \min([0.5 + \sqrt{(0.25 - K_{span} / 0.9)}], 0.95)$ = **208** mm

Area of tension reinforcement required; $A_{s\_span\_req} = M_{span} / (0.87 \times f_y \times z_{span})$ = **475** mm$^2$/m

Minimum area of tension reinforcement; $A_{s\_span\_min} = 0.13 \times h_{span} / 100$ = **325** mm$^2$/m

Tension reinforcement provided; **12 dia.bars @ 200 centres**

Area of tension reinforcement provided; $A_{s\_span\_prov}$ = **565** mm$^2$/m

***PASS - Tension reinforcement provided exceeds tension reinforcement required***

**Basic span/effective depth ratio (cl 3.4.6.3)**

**From BS8110 : Part 1 : 1997 – Table 3.9**

Basic span/effective depth ratio; $ratio_{basic}$ = **20.0**

**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; $f_s = 2 \times f_y \times A_{s\_span\_req} / (3 \times A_{s\_span\_prov} \times \beta_b)$ = **280.183** N/mm$^2$

Modification factor for tension reinforcement; $factor_{tens} = 0.55 + (477\ \mathrm{N/mm^2} - f_s)/(120 \times (0.9\ \mathrm{N/mm^2} + (M_{span} / d_{span}^2)))$

$factor_{tens}$ = **1.463**

**Check span/effective depth ratio (cl 3.4.6.1)**

Allowable span/effective depth ratio; $ratio_{adm} = ratio_{basic} \times factor_{tens}$ = **29.255**

Actual span/effective depth ratio; $ratio_{act} = L / d_{span}$ = **19.406**

***PASS - Span/effective depth ratio is adequate***

**Upper landing support design**

Diameter of tension reinforcement; $\phi_{upper}$ = **12** mm

Depth of reinforcement; $d_{upper} = h_{upper} - c_{nom} - \phi_{upper} / 2$ = **169** mm

Area of tension reinforcement required; $A_{s\_upper\_req} = 0.4 \times A_{s\_span\_req}$ = **190** mm$^2$/m

Minimum area of tension reinforcement; $A_{s\_upper\_min}$ = **260** mm$^2$/m

Tension reinforcement provided; **12 dia.bars @ 200 centres**

Area of tension reinforcement provided; $A_{s\_upper\_prov}$ = **565** mm$^2$/m

***PASS - Tension reinforcement provided exceeds tension reinforcement required***

Kelly Mweu Changilwa
+254703206460
mweukelly@gmail.com

**Shear stress in beam (cl 3.4.5.2)**

Design shear force; $V_{upper} = 0.500 \times F \times L =$ **40.487** kN/m

Design shear stress; $v_{upper} = V_{upper} / d_{upper} =$ **0.240** N/mm$^2$

Allowable design shear stress; $v_{max} = \min(0.8N/mm^2 \times \sqrt{(f_{cu} / 1 N/mm^2)}, 5 N/mm^2) =$ **5.000** N/mm$^2$

***PASS - Design shear stress does not exceed allowable shear stress***

**From BS 8110:Part 1:1997 - Table 3.8**

Design concrete shear stress; $v_{c\_upper} =$ **0.637** N/mm$^2$

***PASS - Design shear stress does not exceed design concrete shear stress***

**<u>Lower landing support design</u>**

Diameter of tension reinforcement; $\phi_{lower} =$ **12** mm

Depth of reinforcement; $d_{lower} = h_{lower} - c_{nom} - \phi_{lower} / 2 =$ **169** mm

Area of tension reinforcement required; $A_{s\_lower\_req} = 0.4 \times A_{s\_span\_req} =$ **190** mm$^2$/m

Minimum area of tension reinforcement; $A_{s\_lower\_min} =$ **260** mm$^2$/m

Tension reinforcement provided; **12 dia.bars @ 200 centres**

Area of tension reinforcement provided; $A_{s\_lower\_prov} =$ **565** mm$^2$/m

***PASS - Tension reinforcement provided exceeds tension reinforcement required***

**Shear stress in beam (cl 3.4.5.2)**

Design shear force; $V_{lower} = 0.500 \times F \times L =$ **40.487** kN/m

Design shear stress; $v_{lower} = V_{lower} / d_{lower} =$ **0.240** N/mm$^2$

Allowable design shear stress; $v_{max} = \min(0.8N/mm^2 \times \sqrt{(f_{cu} / 1 N/mm^2)}, 5 N/mm^2) =$ **5.000** N/mm$^2$

***PASS - Design shear stress does not exceed allowable shear stress***

**From BS 8110:Part 1:1997 - Table 3.8**

Design concrete shear stress; $v_{c\_lower} =$ **0.637** N/mm$^2$

***PASS - Design shear stress does not exceed design concrete shear stress***

# EN FLOOR SLAB A

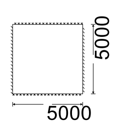
## RC SLAB DESIGN

**In accordance with EN1992-1-1:2004 incorporating corrigendum January 2008 and the UK national annex**

Tedds calculation version 1.0.22

### Design summary

| Description | Unit | Provided | Required | Utilisation | Result |
|---|---|---|---|---|---|
| **Short span** | | | | | |
| Reinf. at midspan | $mm^2/m$ | 804 | 476 | 0.592 | PASS |
| Bar spacing at midspan | mm | 250 | 300 | 0.833 | PASS |
| Reinf. at support | $mm^2/m$ | 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 | $mm^2/m$ | 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 |




**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; | $l_x$ = **5000** mm |
| Longer effective span of panel; | $l_y$ = **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; $G_k$ = **2.0** kN/m$^2$

Characteristic variable action; $Q_k$ = **15.0** kN/m$^2$

Partial factor for permanent action; $\gamma_G$ = **1.35**

Partial factor for variable action; $\gamma_Q$ = **1.50**

Quasi-permanent value of variable action; $\psi_2$ = **0.30**

Design ultimate load; $q = \gamma_G \times G_k + \gamma_Q \times Q_k$ = **25.2** kN/m$^2$

Quasi-permanent load; $q_{SLS} = 1.0 \times G_k + \psi_2 \times Q_k$ = **6.5** kN/m$^2$

**Concrete properties**

Concrete strength class; C25/30

Characteristic cylinder strength; $f_{ck}$ = **25** N/mm$^2$

Partial factor (Table 2.1N); $\gamma_C$ = **1.50**

Compressive strength factor (cl. 3.1.6); $\alpha_{cc}$ = **0.85**

Design compressive strength (cl. 3.1.6); $f_{cd}$ = **14.2** N/mm$^2$

Mean axial tensile strength (Table 3.1); $f_{ctm}$ = 0.30 N/mm$^2$ $\times$ ($f_{ck}$ / 1 N/mm$^2$)$^{2/3}$ = **2.6** N/mm$^2$

Maximum aggregate size; $d_g$ = **20** mm

**Reinforcement properties**

Characteristic yield strength; $f_{yk}$ = **500** N/mm$^2$

Partial factor (Table 2.1N); $\gamma_S$ = **1.15**

Design yield strength (fig. 3.8); $f_{yd} = f_{yk} / \gamma_S$ = **434.8** N/mm$^2$

**Concrete cover to reinforcement**

Nominal cover to outer top reinforcement; $c_{nom\_t}$ = **35** mm

Nominal cover to outer bottom reinforcement; $c_{nom\_b}$ = **35** mm

Fire resistance period to top of slab; $R_{top}$ = **60** min

Fire resistance period to bottom of slab; $R_{btm}$ = **60** min

Axia distance to top reinft (Table 5.8); $a_{fi\_t}$ = **10** mm

Axia distance to bottom reinft (Table 5.8); $a_{fi\_b}$ = **10** mm

Min. top cover requirement with regard to bond; $c_{min,b\_t}$ = **16** mm

Min. btm cover requirement with regard to bond; $c_{min,b\_b}$ = **16** mm

Reinforcement fabrication; **Subject to QA system**

Cover allowance for deviation; $\Delta c_{dev}$ = **5** mm

Min. required nominal cover to top reinft; $c_{nom\_t\_min}$ = **21.0** mm

Min. required nominal cover to bottom reinft; $c_{nom\_b\_min}$ = **21.0** mm

***PASS - There is sufficient cover to the top reinforcement***

***PASS - There is sufficient cover to the bottom reinforcement***

**Reinforcement design at midspan in short span direction (cl.6.1)**

Bending moment coefficient; $\beta_{sx\_p}$ = **0.0340**

Design bending moment; $M_{x\_p} = \beta_{sx\_p} \times q \times l_x^2$ = **21.4** kNm/m

Reinforcement provided; 16 mm dia. bars at 250 mm centres

Area provided; $A_{sx\_p}$ = 804 mm$^2$/m

Effective depth to tension reinforcement; $d_{x\_p} = h - c_{nom\_b} - \phi_{x\_p} / 2$ = **357.0** mm

Kelly Mweu Changilwa
+254703206460
mweukelly@gmail.com

K factor; $K = M_{x\_p} / (b \times d_{x\_p}^2 \times f_{ck}) =$ **0.007**

Redistribution ratio; $\delta = 1.0$

K' factor; $K' = 0.598 \times \delta - 0.18 \times \delta^2 - 0.21 =$ **0.208**

***K < K' - Compression reinforcement is not required***

Lever arm; $z = \min(0.95 \times d_{x\_p}, d_{x\_p}/2 \times (1 + \sqrt{(1 - 3.53 \times K)})) =$ **339.2** mm

Area of reinforcement required for bending; $A_{sx\_p\_m} = M_{x\_p} / (f_{yd} \times z) =$ **145** mm²/m

Minimum area of reinforcement required; $A_{sx\_p\_min} = \max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{x\_p}, 0.0013 \times b \times d_{x\_p}) =$ **476** mm²/m

Area of reinforcement required; $A_{sx\_p\_req} = \max(A_{sx\_p\_m}, A_{sx\_p\_min}) =$ **476** mm²/m

***PASS - Area of reinforcement provided exceeds area required***

**Check reinforcement spacing**

Reinforcement service stress; $\sigma_{sx\_p} = (f_{yk} / \gamma_S) \times \min((A_{sx\_p\_m}/A_{sx\_p}), 1.0) \times q_{SLS} / q =$ **20.3** N/mm²

Maximum allowable spacing (Table 7.3N); $s_{max\_x\_p} =$ **300** mm

Actual bar spacing; $s_{x\_p} =$ **250** mm

***PASS - The reinforcement spacing is acceptable***

**Reinforcement design at midspan in long span direction (cl.6.1)**

Bending moment coefficient; $\beta_{sy\_p} =$ **0.0340**

Design bending moment; $M_{y\_p} = \beta_{sy\_p} \times q \times l_x^2 =$ **21.4** kNm/m

Reinforcement provided; 16 mm dia. bars at 250 mm centres

Area provided; $A_{sy\_p} = 804$ mm²/m

Effective depth to tension reinforcement; $d_{y\_p} = h - c_{nom\_b} - \phi_{x\_p} - \phi_{y\_p} / 2 =$ **341.0** mm

K factor; $K = M_{y\_p} / (b \times d_{y\_p}^2 \times f_{ck}) =$ **0.007**

Redistribution ratio; $\delta = 1.0$

K' factor; $K' = 0.598 \times \delta - 0.18 \times \delta^2 - 0.21 =$ **0.208**

***K < K' - Compression reinforcement is not required***

Lever arm; $z = \min(0.95 \times d_{y\_p}, d_{y\_p}/2 \times (1 + \sqrt{(1 - 3.53 \times K)})) =$ **324.0** mm

Area of reinforcement required for bending; $A_{sy\_p\_m} = M_{y\_p} / (f_{yd} \times z) =$ **152** mm²/m

Minimum area of reinforcement required; $A_{sy\_p\_min} = \max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{y\_p}, 0.0013 \times b \times d_{y\_p}) =$ **455** mm²/m

Area of reinforcement required; $A_{sy\_p\_req} = \max(A_{sy\_p\_m}, A_{sy\_p\_min}) =$ **455** mm²/m

***PASS - Area of reinforcement provided exceeds area required***

**Check reinforcement spacing**

Reinforcement service stress; $\sigma_{sy\_p} = (f_{yk} / \gamma_S) \times \min((A_{sy\_p\_m}/A_{sy\_p}), 1.0) \times q_{SLS} / q =$ **21.2** N/mm²

Maximum allowable spacing (Table 7.3N); $s_{max\_y\_p} =$ **300** mm

Actual bar spacing; $s_{y\_p} =$ **250** mm

***PASS - The reinforcement spacing is acceptable***

**Reinforcement design at continuous support in short span direction (cl.6.1)**

Bending moment coefficient; $\beta_{sx\_n} =$ **0.0460**

Design bending moment; $M_{x\_n} = \beta_{sx\_n} \times q \times l_x^2 =$ **29.0** kNm/m

Reinforcement provided; 16 mm dia. bars at 250 mm centres

Area provided; $A_{sx\_n} = 804$ mm²/m

Effective depth to tension reinforcement; $d_{x\_n} = h - c_{nom\_t} - \phi_{x\_n} / 2 =$ **357.0** mm

K factor; $K = M_{x\_n} / (b \times d_{x\_n}^2 \times f_{ck}) =$ **0.009**

Redistribution ratio; $\delta = 1.0$

K' factor; $K' = 0.598 \times \delta - 0.18 \times \delta^2 - 0.21 = \mathbf{0.208}$

***K < K' - Compression reinforcement is not required***

Lever arm; $z = \min(0.95 \times d_{x\_n}, d_{x\_n}/2 \times (1 + \sqrt{(1 - 3.53 \times K)})) = \mathbf{339.2}$ mm

Area of reinforcement required for bending; $A_{sx\_n\_m} = M_{x\_n} / (f_{yd} \times z) = \mathbf{197}$ mm$^2$/m

Minimum area of reinforcement required; $A_{sx\_n\_min} = \max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{x\_n}, 0.0013 \times b \times d_{x\_n}) = \mathbf{476}$ mm$^2$/m

Area of reinforcement required; $A_{sx\_n\_req} = \max(A_{sx\_n\_m}, A_{sx\_n\_min}) = \mathbf{476}$ mm$^2$/m

***PASS - Area of reinforcement provided exceeds area required***

**Check reinforcement spacing**

Reinforcement service stress; $\sigma_{sx\_n} = (f_{yk} / \gamma_S) \times \min((A_{sx\_n\_m}/A_{sx\_n}), 1.0) \times q_{SLS} / q = \mathbf{27.4}$ N/mm$^2$

Maximum allowable spacing (Table 7.3N); $s_{max\_x\_n} = \mathbf{300}$ mm

Actual bar spacing; $s_{x\_n} = \mathbf{250}$ mm

***PASS - The reinforcement spacing is acceptable***

**Shear capacity check at short span continuous support**

Shear force; $V_{x\_n} = 1.1 \times q \times l_x / 2 = \mathbf{69.3}$ kN/m

Effective depth factor (cl. 6.2.2); $k = \min(2.0, 1 + (200 \text{ mm} / d_{x\_n})^{0.5}) = \mathbf{1.748}$

Reinforcement ratio; $\rho_l = \min(0.02, A_{sx\_n} / (b \times d_{x\_n})) = \mathbf{0.0023}$

Minimum shear resistance (Exp. 6.3N); $V_{Rd,c\_min} = 0.035 \text{ N/mm}^2 \times k^{1.5} \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} \times b \times d_{x\_n}$

$V_{Rd,c\_min} = \mathbf{144.4}$ kN/m

Shear resistance constant (cl. 6.2.2); $C_{Rd,c} = 0.18 \text{ N/mm}^2 / \gamma_C = \mathbf{0.12}$ N/mm$^2$

Shear resistance (Exp. 6.2a);

$V_{Rd,c\_x\_n} = \max(V_{Rd,c\_min}, C_{Rd,c} \times k \times (100 \times \rho_l \times (f_{ck} / 1 \text{ N/mm}^2))^{0.333} \times b \times d_{x\_n}) = \mathbf{144.4}$ kN/m

***PASS - Shear capacity is adequate***

**Shear capacity check at long span discontinuous support**

Shear force; $V_{y\_d} = q \times l_x / 2 =$ ;**63.0**; kN/m;

Reinforcement provided; **16 mm dia. bars at 250 mm centres**

Area provided; $A_{sy\_d} = \mathbf{804}$ mm$^2$/m

Effective depth; $d_{y\_d} = h - c_{nom\_b} - \phi_{x\_p} - \phi_{y\_d} / 2 =$ ;**341.0**; mm

Effective depth factor; $k = \min(2.0, 1 + (200 \text{ mm} / d_{y\_d})^{0.5}) = \mathbf{1.766}$

Reinforcement ratio; $\rho_l = \min(0.02, A_{sy\_d} / (b \times d_{y\_d})) = \mathbf{0.0024}$

Minimum shear resistance; $V_{Rd,c\_min} = 0.035 \text{ N/mm}^2 \times k^{1.5} \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} \times b \times d_{y\_d}$

$V_{Rd,c\_min} = \mathbf{140.0}$ kN/m

Shear resistance constant (cl. 6.2.2); $C_{Rd,c} = 0.18 \text{ N/mm}^2 / \gamma_C = \mathbf{0.12}$ N/mm$^2$

Shear resistance;

$V_{Rd,c\_y\_d} = \max(V_{Rd,c\_min}, C_{Rd,c} \times k \times (100 \times \rho_l \times (f_{ck}/1 \text{ N/mm}^2))^{0.333} \times b \times d_{y\_d}) = \mathbf{140.0}$ kN/m

***PASS - Shear capacity is adequate (0.450)***

**Basic span-to-depth deflection ratio check (cl. 7.4.2)**

Reference reinforcement ratio; $\rho_0 = (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / 1000 = \mathbf{0.0050}$

Required tension reinforcement ratio; $\rho = \max(0.0035, A_{sx\_p\_req} / (b \times d_{x\_p})) = \mathbf{0.0035}$

Required compression reinforcement ratio; $\rho' = A_{scx\_p\_req} / (b \times d_{x\_p}) = \mathbf{0.0000}$

Stuctural system factor (Table 7.4N); $K_\delta = \mathbf{1.5}$

Basic limit span-to-depth ratio (Exp. 7.16);

$ratio_{lim\_x\_bas} = K_\delta \times [11 + 1.5 \times (f_{ck}/1 \text{ N/mm}^2)^{0.5} \times \rho_0/\rho + 3.2 \times (f_{ck}/1 \text{ N/mm}^2)^{0.5} \times (\rho_0/\rho - 1)^{1.5}] = \mathbf{39.31}$

Mod span-to-depth ratio limit;

$$ratio_{lim\_x} = \min(40 \times K_{\delta}, \min(1.5, (500\ N/mm^2 / f_{yk}) \times (A_{sx\_p} / A_{sx\_p\_m})) \times ratio_{lim\_x\_bas}) = \mathbf{58.96}$$

Actual span-to-eff. depth ratio; $ratio_{act\_x} = l_x / d_{x\_p} = \mathbf{14.01}$

***PASS - Actual span-to-effective depth ratio is acceptable***

**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**

Continuous support in short span direction; **16 mm dia. bars at 250 mm centres T1**

Discontinuous support in long span direction; **16 mm dia. bars at 250 mm centres B2**

**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.


;

# EN FLOOR SLAB B

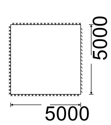
## RC SLAB DESIGN

**In accordance with EN1992-1-1:2004 incorporating corrigendum January 2008 and the UK national annex**

Tedds calculation version 1.0.22

### Design summary

| Description | Unit | Provided | Required | Utilisation | Result |
|---|---|---|---|---|---|
| **Short span** | | | | | |
| Reinf. at midspan | $mm^2/m$ | 804 | 476 | 0.592 | PASS |
| Bar spacing at midspan | mm | 250 | 300 | 0.833 | PASS |
| Reinf. at support | $mm^2/m$ | 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 | $mm^2/m$ | 804 | 455 | 0.566 | PASS |
| Bar spacing at midspan | mm | 250 | 300 | 0.833 | PASS |
| Reinf. at support | $mm^2/m$ | 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 |




**Slab definition**

Slab reference name; **Floor Slab B**

Type of slab; **Two way spanning with restrained edges**

Overall slab depth; $h = \mathbf{400}$ mm

Shorter effective span of panel; $l_x = \mathbf{5000}$ mm

Longer effective span of panel; $l_y = \mathbf{5000}$ 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; | $G_k$ = **2.0** kN/m$^2$ |
| Characteristic variable action; | $Q_k$ = **15.0** kN/m$^2$ |
| Partial factor for permanent action; | $\gamma_G$ = **1.35** |
| Partial factor for variable action; | $\gamma_Q$ = **1.50** |
| Quasi-permanent value of variable action; | $\psi_2$ = **0.30** |
| Design ultimate load; | $q = \gamma_G \times G_k + \gamma_Q \times Q_k$ = **25.2** kN/m$^2$ |
| Quasi-permanent load; | $q_{SLS} = 1.0 \times G_k + \psi_2 \times Q_k$ = **6.5** kN/m$^2$ |

**Concrete properties**

| | |
|---|---|
| Concrete strength class; | C25/30 |
| Characteristic cylinder strength; | $f_{ck}$ = **25** N/mm$^2$ |
| Partial factor (Table 2.1N); | $\gamma_C$ = **1.50** |
| Compressive strength factor (cl. 3.1.6); | $\alpha_{cc}$ = **0.85** |
| Design compressive strength (cl. 3.1.6); | $f_{cd}$ = **14.2** N/mm$^2$ |
| Mean axial tensile strength (Table 3.1); | $f_{ctm} = 0.30\ \text{N/mm}^2 \times (f_{ck} / 1\ \text{N/mm}^2)^{2/3}$ = **2.6** N/mm$^2$ |
| Maximum aggregate size; | $d_g$ = **20** mm |

**Reinforcement properties**

| | |
|---|---|
| Characteristic yield strength; | $f_{yk}$ = **500** N/mm$^2$ |
| Partial factor (Table 2.1N); | $\gamma_S$ = **1.15** |
| Design yield strength (fig. 3.8); | $f_{yd} = f_{yk} / \gamma_S$ = **434.8** N/mm$^2$ |

**Concrete cover to reinforcement**

| | |
|---|---|
| Nominal cover to outer top reinforcement; | $c_{nom\_t}$ = **35** mm |
| Nominal cover to outer bottom reinforcement; | $c_{nom\_b}$ = **35** mm |
| Fire resistance period to top of slab; | $R_{top}$ = **60** min |
| Fire resistance period to bottom of slab; | $R_{btm}$ = **60** min |
| Axia distance to top reinft (Table 5.8); | $a_{fi\_t}$ = **10** mm |
| Axia distance to bottom reinft (Table 5.8); | $a_{fi\_b}$ = **10** mm |
| Min. top cover requirement with regard to bond; | $c_{min,b\_t}$ = **16** mm |
| Min. btm cover requirement with regard to bond; | $c_{min,b\_b}$ = **16** mm |
| Reinforcement fabrication; | **Not subject to QA system** |
| Cover allowance for deviation; | $\Delta c_{dev}$ = **10** mm |
| Min. required nominal cover to top reinft; | $c_{nom\_t\_min}$ = **26.0** mm |
| Min. required nominal cover to bottom reinft; | $c_{nom\_b\_min}$ = **26.0** mm |

***PASS - There is sufficient cover to the top reinforcement***

***PASS - There is sufficient cover to the bottom reinforcement***

**Reinforcement design at midspan in short span direction (cl.6.1)**

| | |
|---|---|
| Bending moment coefficient; | $\beta_{sx\_p}$ = **0.0290** |
| Design bending moment; | $M_{x\_p} = \beta_{sx\_p} \times q \times l_x^2$ = **18.3** kNm/m |
| Reinforcement provided; | 16 mm dia. bars at 250 mm centres |

Area provided; $A_{sx\_p}$ = 804 mm$^2$/m

Effective depth to tension reinforcement; $d_{x\_p} = h - c_{nom\_b} - \phi_{x\_p} / 2 =$ **357.0** mm

K factor; $K = M_{x\_p} / (b \times d_{x\_p}^2 \times f_{ck}) =$ **0.006**

Redistribution ratio; $\delta = 1.0$

K' factor; $K' = 0.598 \times \delta - 0.18 \times \delta^2 - 0.21 =$ **0.208**

***K < K' - Compression reinforcement is not required***

Lever arm; $z = \min(0.95 \times d_{x\_p}, d_{x\_p}/2 \times (1 + \sqrt{(1 - 3.53 \times K)})) =$ **339.2** mm

Area of reinforcement required for bending; $A_{sx\_p\_m} = M_{x\_p} / (f_{yd} \times z) =$ **124** mm$^2$/m

Minimum area of reinforcement required; $A_{sx\_p\_min} = \max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{x\_p}, 0.0013 \times b \times d_{x\_p}) =$ **476** mm$^2$/m

Area of reinforcement required; $A_{sx\_p\_req} = \max(A_{sx\_p\_m}, A_{sx\_p\_min}) =$ **476** mm$^2$/m

***PASS - Area of reinforcement provided exceeds area required***

**Check reinforcement spacing**

Reinforcement service stress; $\sigma_{sx\_p} = (f_{yk} / \gamma_S) \times \min((A_{sx\_p\_m}/A_{sx\_p}), 1.0) \times q_{SLS} / q =$ **17.3** N/mm$^2$

Maximum allowable spacing (Table 7.3N); $s_{max\_x\_p} =$ **300** mm

Actual bar spacing; $s_{x\_p} =$ **250** mm

***PASS - The reinforcement spacing is acceptable***

**Reinforcement design at midspan in long span direction (cl.6.1)**

Bending moment coefficient; $\beta_{sy\_p} =$ **0.0280**

Design bending moment; $M_{y\_p} = \beta_{sy\_p} \times q \times l_x^2 =$ **17.6** kNm/m

Reinforcement provided; 16 mm dia. bars at 250 mm centres

Area provided; $A_{sy\_p}$ = 804 mm$^2$/m

Effective depth to tension reinforcement; $d_{y\_p} = h - c_{nom\_b} - \phi_{x\_p} - \phi_{y\_p} / 2 =$ **341.0** mm

K factor; $K = M_{y\_p} / (b \times d_{y\_p}^2 \times f_{ck}) =$ **0.006**

Redistribution ratio; $\delta = 1.0$

K' factor; $K' = 0.598 \times \delta - 0.18 \times \delta^2 - 0.21 =$ **0.208**

***K < K' - Compression reinforcement is not required***

Lever arm; $z = \min(0.95 \times d_{y\_p}, d_{y\_p}/2 \times (1 + \sqrt{(1 - 3.53 \times K)})) =$ **324.0** mm

Area of reinforcement required for bending; $A_{sy\_p\_m} = M_{y\_p} / (f_{yd} \times z) =$ **125** mm$^2$/m

Minimum area of reinforcement required; $A_{sy\_p\_min} = \max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{y\_p}, 0.0013 \times b \times d_{y\_p}) =$ **455** mm$^2$/m

Area of reinforcement required; $A_{sy\_p\_req} = \max(A_{sy\_p\_m}, A_{sy\_p\_min}) =$ **455** mm$^2$/m

***PASS - Area of reinforcement provided exceeds area required***

**Check reinforcement spacing**

Reinforcement service stress; $\sigma_{sy\_p} = (f_{yk} / \gamma_S) \times \min((A_{sy\_p\_m}/A_{sy\_p}), 1.0) \times q_{SLS} / q =$ **17.5** N/mm$^2$

Maximum allowable spacing (Table 7.3N); $s_{max\_y\_p} =$ **300** mm

Actual bar spacing; $s_{y\_p} =$ **250** mm

***PASS - The reinforcement spacing is acceptable***

**Reinforcement design at continuous support in short span direction (cl.6.1)**

Bending moment coefficient; $\beta_{sx\_n} =$ **0.0390**

Design bending moment; $M_{x\_n} = \beta_{sx\_n} \times q \times l_x^2 =$ **24.6** kNm/m

Reinforcement provided; 16 mm dia. bars at 250 mm centres

Area provided; $A_{sx\_n}$ = 804 mm$^2$/m

Effective depth to tension reinforcement; $d_{x\_n} = h - c_{nom\_t} - \phi_{x\_n} / 2 =$ **357.0** mm

K factor; $K = M_{x\_n} / (b \times d_{x\_n}^2 \times f_{ck}) = \mathbf{0.008}$

Redistribution ratio; $\delta = 1.0$

K' factor; $K' = 0.598 \times \delta - 0.18 \times \delta^2 - 0.21 = \mathbf{0.208}$

***K < K' - Compression reinforcement is not required***

Lever arm; $z = \min(0.95 \times d_{x\_n}, d_{x\_n}/2 \times (1 + \sqrt{(1 - 3.53 \times K)})) = \mathbf{339.2}$ mm

Area of reinforcement required for bending; $A_{sx\_n\_m} = M_{x\_n} / (f_{yd} \times z) = \mathbf{167}$ mm²/m

Minimum area of reinforcement required; $A_{sx\_n\_min} = \max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{x\_n}, 0.0013 \times b \times d_{x\_n}) = \mathbf{476}$ mm²/m

Area of reinforcement required; $A_{sx\_n\_req} = \max(A_{sx\_n\_m}, A_{sx\_n\_min}) = \mathbf{476}$ mm²/m

***PASS - Area of reinforcement provided exceeds area required***

**Check reinforcement spacing**

Reinforcement service stress; $\sigma_{sx\_n} = (f_{yk} / \gamma_S) \times \min((A_{sx\_n\_m}/A_{sx\_n}), 1.0) \times q_{SLS} / q = \mathbf{23.2}$ N/mm²

Maximum allowable spacing (Table 7.3N); $s_{max\_x\_n} = \mathbf{300}$ mm

Actual bar spacing; $s_{x\_n} = \mathbf{250}$ mm

***PASS - The reinforcement spacing is acceptable***

**Reinforcement design at continuous support in long span direction (cl.6.1)**

Bending moment coefficient; $\beta_{sy\_n} = \mathbf{0.0370}$

Design bending moment; $M_{y\_n} = \beta_{sy\_n} \times q \times l_x^2 = \mathbf{23.3}$ kNm/m

Reinforcement provided; 16 mm dia. bars at 250 mm centres

Area provided; $A_{sy\_n} = 804$ mm²/m

Effective depth to tension reinforcement; $d_{y\_n} = h - c_{nom\_t} - \phi_{x\_n} - \phi_{y\_n} / 2 = \mathbf{341.0}$ mm

K factor; $K = M_{y\_n} / (b \times d_{y\_n}^2 \times f_{ck}) = \mathbf{0.008}$

Redistribution ratio; $\delta = 1.0$

K' factor; $K' = 0.598 \times \delta - 0.18 \times \delta^2 - 0.21 = \mathbf{0.208}$

***K < K' - Compression reinforcement is not required***

Lever arm; $z = \min(0.95 \times d_{y\_n}, d_{y\_n}/2 \times (1 + \sqrt{(1 - 3.53 \times K)})) = \mathbf{324.0}$ mm

Area of reinforcement required for bending; $A_{sy\_n\_m} = M_{y\_n} / (f_{yd} \times z) = \mathbf{165}$ mm²/m

Minimum area of reinforcement required; $A_{sy\_n\_min} = \max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{y\_n}, 0.0013 \times b \times d_{y\_n}) = \mathbf{455}$ mm²/m

Area of reinforcement required; $A_{sy\_n\_req} = \max(A_{sy\_n\_m}, A_{sy\_n\_min}) = \mathbf{455}$ mm²/m

***PASS - Area of reinforcement provided exceeds area required***

**Check reinforcement spacing**

Reinforcement service stress; $\sigma_{sy\_n} = (f_{yk} / \gamma_S) \times \min((A_{sy\_n\_m}/A_{sy\_n}), 1.0) \times q_{SLS} / q = \mathbf{23.1}$ N/mm²

Maximum allowable spacing (Table 7.3N); $s_{max\_y\_n} = \mathbf{300}$ mm

Actual bar spacing; $s_{y\_n} = \mathbf{250}$ mm

***PASS - The reinforcement spacing is acceptable***

**Shear capacity check at short span continuous support**

Shear force; $V_{x\_n} = 1.1 \times q \times l_x / 2 = \mathbf{69.3}$ kN/m

Effective depth factor (cl. 6.2.2); $k = \min(2.0, 1 + (200\text{ mm} / d_{x\_n})^{0.5}) = \mathbf{1.748}$

Reinforcement ratio; $\rho_l = \min(0.02, A_{sx\_n} / (b \times d_{x\_n})) = \mathbf{0.0023}$

Minimum shear resistance (Exp. 6.3N); $V_{Rd,c\_min} = 0.035\text{ N/mm}^2 \times k^{1.5} \times (f_{ck} / 1\text{ N/mm}^2)^{0.5} \times b \times d_{x\_n}$

$V_{Rd,c\_min} = \mathbf{144.4}$ kN/m

Shear resistance constant (cl. 6.2.2); $C_{Rd,c} = 0.18\text{ N/mm}^2 / \gamma_C = \mathbf{0.12}$ N/mm²

Shear resistance (Exp. 6.2a);

$V_{Rd,c\_x\_n} = \max(V_{Rd,c\_min}, C_{Rd,c} \times k \times (100 \times \rho_l \times (f_{ck} / 1\ N/mm^2))^{0.333} \times b \times d_{x\_n}) =$ **144.4** kN/m

***PASS - Shear capacity is adequate***

**Shear capacity check at long span continuous support**

Shear force; $V_{y\_n} = 1.1 \times q \times l_x / 2 =$ **69.3** kN/m

Effective depth factor (cl. 6.2.2); $k = \min(2.0, 1 + (200\ mm / d_{y\_n})^{0.5}) =$ **1.766**

Reinforcement ratio; $\rho_l = \min(0.02, A_{sy\_n} / (b \times d_{y\_n})) =$ **0.0024**

Minimum shear resistance (Exp. 6.3N); $V_{Rd,c\_min} = 0.035\ N/mm^2 \times k^{1.5} \times (f_{ck} / 1\ N/mm^2)^{0.5} \times b \times d_{y\_n}$

$V_{Rd,c\_min} =$ **140.0** kN/m

Shear resistance constant (cl. 6.2.2); $C_{Rd,c} = 0.18\ N/mm^2 / \gamma_C =$ **0.12** $N/mm^2$

Shear resistance (Exp. 6.2a);

$V_{Rd,c\_y\_n} = \max(V_{Rd,c\_min}, C_{Rd,c} \times k \times (100 \times \rho_l \times (f_{ck} / 1\ N/mm^2))^{0.333} \times b \times d_{y\_n}) =$ **140.0** kN/m

***PASS - Shear capacity is adequate***

**Shear capacity check at long span discontinuous support**

Shear force; $V_{x\_d} = 0.8 \times q \times l_x / 2 =$ ;**50.4**; kN/m;

Reinforcement provided; **16 mm dia. bars at 250 mm centres**

Area provided; $A_{sy\_d} =$ **804** $mm^2/m$

Effective depth; $d_{y\_d} = h - c_{nom\_b} - \phi_{x\_p} - \phi_{y\_d} / 2 =$ ;**341.0**; mm

Effective depth factor; $k = \min(2.0, 1 + (200\ mm / d_{y\_d})^{0.5}) =$ **1.766**

Reinforcement ratio; $\rho_l = \min(0.02, A_{sy\_d} / (b \times d_{y\_d})) =$ **0.0024**

Minimum shear resistance; $V_{Rd,c\_min} = 0.035\ N/mm^2 \times k^{1.5} \times (f_{ck} / 1\ N/mm^2)^{0.5} \times b \times d_{y\_d}$

$V_{Rd,c\_min} =$ **140.0** kN/m

Shear resistance constant (cl. 6.2.2); $C_{Rd,c} = 0.18\ N/mm^2 / \gamma_C =$ **0.12** $N/mm^2$

Shear resistance;

$V_{Rd,c\_y\_d} = \max(V_{Rd,c\_min}, C_{Rd,c} \times k \times (100 \times \rho_l \times (f_{ck}/1\ N/mm^2))^{0.333} \times b \times d_{y\_d}) =$ **140.0** kN/m

***PASS - Shear capacity is adequate (0.360)***

**Basic span-to-depth deflection ratio check (cl. 7.4.2)**

Reference reinforcement ratio; $\rho_0 = (f_{ck} / 1\ N/mm^2)^{0.5} / 1000 =$ **0.0050**

Required tension reinforcement ratio; $\rho = \max(0.0035, A_{sx\_p\_req} / (b \times d_{x\_p})) =$ **0.0035**

Required compression reinforcement ratio; $\rho' = A_{scx\_p\_req} / (b \times d_{x\_p}) =$ **0.0000**

Stuctural system factor (Table 7.4N); $K_\delta =$ **1.5**

Basic limit span-to-depth ratio (Exp. 7.16);

$ratio_{lim\_x\_bas} = K_\delta \times [11 + 1.5 \times (f_{ck}/1\ N/mm^2)^{0.5} \times \rho_0/\rho + 3.2 \times (f_{ck}/1\ N/mm^2)^{0.5} \times (\rho_0/\rho - 1)^{1.5}] =$ **39.31**

Mod span-to-depth ratio limit;

$ratio_{lim\_x} = \min(40 \times K_\delta, \min(1.5, (500\ N/mm^2 / f_{yk}) \times (A_{sx\_p} / A_{sx\_p\_m})) \times ratio_{lim\_x\_bas}) =$ **58.96**

Actual span-to-eff. depth ratio; $ratio_{act\_x} = l_x / d_{x\_p} =$ **14.01**

***PASS - Actual span-to-effective depth ratio is acceptable***

**<u>Reinforcement summary</u>**

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**

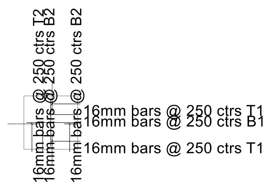
Continuous support in short span direction; **16 mm dia. bars at 250 mm centres T1**

Continuous support in long span direction; **16 mm dia. bars at 250 mm centres T2**

Discontinuous support in long span direction; **16 mm dia. bars at 250 mm centres B2**

**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.




;

Kelly Mweu Changilwa
+254703206460
mweukelly@gmail.com


# EN FLOOR SLAB C

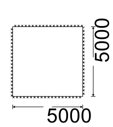
## RC SLAB DESIGN

**In accordance with EN1992-1-1:2004 incorporating corrigendum January 2008 and the UK national annex**

Tedds calculation version 1.0.22

### Design summary

| Description | Unit | Provided | Required | Utilisation | Result |
|---|---|---|---|---|---|
| **Short span** | | | | | |
| Reinf. at midspan | $mm^2/m$ | 804 | 476 | 0.592 | PASS |
| Bar spacing at midspan | mm | 250 | 300 | 0.833 | PASS |
| Reinf. at support | $mm^2/m$ | 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 | $mm^2/m$ | 804 | 455 | 0.566 | PASS |
| Bar spacing at midspan | mm | 250 | 300 | 0.833 | PASS |
| Reinf. at support | $mm^2/m$ | 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 |




**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; $l_x$ = **5000** mm

Longer effective span of panel; $l_y$ = **5000** mm

Support conditions; **Four edges continuous (interior panel)**

Kelly Mweu Changilwa
+254703206460
mweukelly@gmail.com

Top outer layer of reinforcement; **Short span direction**
Bottom outer layer of reinforcement; **Short span direction**

**Loading**

Characteristic permanent action; $G_k = \mathbf{2.0}$ kN/m$^2$
Characteristic variable action; $Q_k = \mathbf{15.0}$ kN/m$^2$
Partial factor for permanent action; $\gamma_G = \mathbf{1.35}$
Partial factor for variable action; $\gamma_Q = \mathbf{1.50}$
Quasi-permanent value of variable action; $\psi_2 = \mathbf{0.30}$
Design ultimate load; $q = \gamma_G \times G_k + \gamma_Q \times Q_k = \mathbf{25.2}$ kN/m$^2$
Quasi-permanent load; $q_{SLS} = 1.0 \times G_k + \psi_2 \times Q_k = \mathbf{6.5}$ kN/m$^2$

**Concrete properties**

Concrete strength class; C25/30
Characteristic cylinder strength; $f_{ck} = \mathbf{25}$ N/mm$^2$
Partial factor (Table 2.1N); $\gamma_C = \mathbf{1.50}$
Compressive strength factor (cl. 3.1.6); $\alpha_{cc} = \mathbf{0.85}$
Design compressive strength (cl. 3.1.6); $f_{cd} = \mathbf{14.2}$ N/mm$^2$
Mean axial tensile strength (Table 3.1); $f_{ctm} = 0.30\text{ N/mm}^2 \times (f_{ck} / 1\text{ N/mm}^2)^{2/3} = \mathbf{2.6}$ N/mm$^2$
Maximum aggregate size; $d_g = \mathbf{20}$ mm

**Reinforcement properties**

Characteristic yield strength; $f_{yk} = \mathbf{500}$ N/mm$^2$
Partial factor (Table 2.1N); $\gamma_S = \mathbf{1.15}$
Design yield strength (fig. 3.8); $f_{yd} = f_{yk} / \gamma_S = \mathbf{434.8}$ N/mm$^2$

**Concrete cover to reinforcement**

Nominal cover to outer top reinforcement; $c_{nom\_t} = \mathbf{35}$ mm
Nominal cover to outer bottom reinforcement; $c_{nom\_b} = \mathbf{35}$ mm
Fire resistance period to top of slab; $R_{top} = \mathbf{60}$ min
Fire resistance period to bottom of slab; $R_{btm} = \mathbf{60}$ min
Axia distance to top reinft (Table 5.8); $a_{fi\_t} = \mathbf{10}$ mm
Axia distance to bottom reinft (Table 5.8); $a_{fi\_b} = \mathbf{10}$ mm
Min. top cover requirement with regard to bond; $c_{min,b\_t} = \mathbf{16}$ mm
Min. btm cover requirement with regard to bond; $c_{min,b\_b} = \mathbf{16}$ mm
Reinforcement fabrication; **Not subject to QA system**
Cover allowance for deviation; $\Delta c_{dev} = \mathbf{10}$ mm
Min. required nominal cover to top reinft; $c_{nom\_t\_min} = \mathbf{26.0}$ mm
Min. required nominal cover to bottom reinft; $c_{nom\_b\_min} = \mathbf{26.0}$ mm

***PASS - There is sufficient cover to the top reinforcement***
***PASS - There is sufficient cover to the bottom reinforcement***

**Reinforcement design at midspan in short span direction (cl.6.1)**

Bending moment coefficient; $\beta_{sx\_p} = \mathbf{0.0240}$
Design bending moment; $M_{x\_p} = \beta_{sx\_p} \times q \times l_x^2 = \mathbf{15.1}$ kNm/m
Reinforcement provided; 16 mm dia. bars at 250 mm centres
Area provided; $A_{sx\_p} = 804$ mm$^2$/m

Effective depth to tension reinforcement; $d_{x\_p} = h - c_{nom\_b} - \phi_{x\_p} / 2 =$ **357.0** mm

K factor; $K = M_{x\_p} / (b \times d_{x\_p}^2 \times f_{ck}) =$ **0.005**

Redistribution ratio; $\delta = 1.0$

K' factor; $K' = 0.598 \times \delta - 0.18 \times \delta^2 - 0.21 =$ **0.208**

***K < K' - Compression reinforcement is not required***

Lever arm; $z = \min(0.95 \times d_{x\_p}, d_{x\_p}/2 \times (1 + \sqrt{(1 - 3.53 \times K)})) =$ **339.2** mm

Area of reinforcement required for bending; $A_{sx\_p\_m} = M_{x\_p} / (f_{yd} \times z) =$ **103** mm²/m

Minimum area of reinforcement required; $A_{sx\_p\_min} = \max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{x\_p}, 0.0013 \times b \times d_{x\_p}) =$ **476** mm²/m

Area of reinforcement required; $A_{sx\_p\_req} = \max(A_{sx\_p\_m}, A_{sx\_p\_min}) =$ **476** mm²/m

***PASS - Area of reinforcement provided exceeds area required***

**Check reinforcement spacing**

Reinforcement service stress; $\sigma_{sx\_p} = (f_{yk} / \gamma_S) \times \min((A_{sx\_p\_m}/A_{sx\_p}), 1.0) \times q_{SLS} / q =$ **14.3** N/mm²

Maximum allowable spacing (Table 7.3N); $s_{max\_x\_p} =$ **300** mm

Actual bar spacing; $s_{x\_p} =$ **250** mm

***PASS - The reinforcement spacing is acceptable***

**Reinforcement design at midspan in long span direction (cl.6.1)**

Bending moment coefficient; $\beta_{sy\_p} =$ **0.0240**

Design bending moment; $M_{y\_p} = \beta_{sy\_p} \times q \times l_x^2 =$ **15.1** kNm/m

Reinforcement provided; 16 mm dia. bars at 250 mm centres

Area provided; $A_{sy\_p} = 804$ mm²/m

Effective depth to tension reinforcement; $d_{y\_p} = h - c_{nom\_b} - \phi_{x\_p} - \phi_{y\_p} / 2 =$ **341.0** mm

K factor; $K = M_{y\_p} / (b \times d_{y\_p}^2 \times f_{ck}) =$ **0.005**

Redistribution ratio; $\delta = 1.0$

K' factor; $K' = 0.598 \times \delta - 0.18 \times \delta^2 - 0.21 =$ **0.208**

***K < K' - Compression reinforcement is not required***

Lever arm; $z = \min(0.95 \times d_{y\_p}, d_{y\_p}/2 \times (1 + \sqrt{(1 - 3.53 \times K)})) =$ **324.0** mm

Area of reinforcement required for bending; $A_{sy\_p\_m} = M_{y\_p} / (f_{yd} \times z) =$ **107** mm²/m

Minimum area of reinforcement required; $A_{sy\_p\_min} = \max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{y\_p}, 0.0013 \times b \times d_{y\_p}) =$ **455** mm²/m

Area of reinforcement required; $A_{sy\_p\_req} = \max(A_{sy\_p\_m}, A_{sy\_p\_min}) =$ **455** mm²/m

***PASS - Area of reinforcement provided exceeds area required***

**Check reinforcement spacing**

Reinforcement service stress; $\sigma_{sy\_p} = (f_{yk} / \gamma_S) \times \min((A_{sy\_p\_m}/A_{sy\_p}), 1.0) \times q_{SLS} / q =$ **15.0** N/mm²

Maximum allowable spacing (Table 7.3N); $s_{max\_y\_p} =$ **300** mm

Actual bar spacing; $s_{y\_p} =$ **250** mm

***PASS - The reinforcement spacing is acceptable***

**Reinforcement design at continuous support in short span direction (cl.6.1)**

Bending moment coefficient; $\beta_{sx\_n} =$ **0.0310**

Design bending moment; $M_{x\_n} = \beta_{sx\_n} \times q \times l_x^2 =$ **19.5** kNm/m

Reinforcement provided; 16 mm dia. bars at 250 mm centres

Area provided; $A_{sx\_n} = 804$ mm²/m

Effective depth to tension reinforcement; $d_{x\_n} = h - c_{nom\_t} - \phi_{x\_n} / 2 =$ **357.0** mm

K factor; $K = M_{x\_n} / (b \times d_{x\_n}^2 \times f_{ck}) =$ **0.006**

Redistribution ratio; $\delta = 1.0$

K' factor; $K' = 0.598 \times \delta - 0.18 \times \delta^2 - 0.21 =$ **0.208**

***K < K' - Compression reinforcement is not required***

Lever arm; $z = \min(0.95 \times d_{x\_n}, d_{x\_n}/2 \times (1 + \sqrt{(1 - 3.53 \times K)})) =$ **339.2** mm

Area of reinforcement required for bending; $A_{sx\_n\_m} = M_{x\_n} / (f_{yd} \times z) =$ **132** mm²/m

Minimum area of reinforcement required; $A_{sx\_n\_min} = \max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{x\_n}, 0.0013 \times b \times d_{x\_n}) =$ **476** mm²/m

Area of reinforcement required; $A_{sx\_n\_req} = \max(A_{sx\_n\_m}, A_{sx\_n\_min}) =$ **476** mm²/m

***PASS - Area of reinforcement provided exceeds area required***

**Check reinforcement spacing**

Reinforcement service stress; $\sigma_{sx\_n} = (f_{yk} / \gamma_S) \times \min((A_{sx\_n\_m}/A_{sx\_n}), 1.0) \times q_{SLS} / q =$ **18.5** N/mm²

Maximum allowable spacing (Table 7.3N); $s_{max\_x\_n} =$ **300** mm

Actual bar spacing; $s_{x\_n} =$ **250** mm

***PASS - The reinforcement spacing is acceptable***

**Reinforcement design at continuous support in long span direction (cl.6.1)**

Bending moment coefficient; $\beta_{sy\_n} =$ **0.0320**

Design bending moment; $M_{y\_n} = \beta_{sy\_n} \times q \times l_x^2 =$ **20.2** kNm/m

Reinforcement provided; 16 mm dia. bars at 250 mm centres

Area provided; $A_{sy\_n} = 804$ mm²/m

Effective depth to tension reinforcement; $d_{y\_n} = h - c_{nom\_t} - \phi_{x\_n} - \phi_{y\_n} / 2 =$ **341.0** mm

K factor; $K = M_{y\_n} / (b \times d_{y\_n}^2 \times f_{ck}) =$ **0.007**

Redistribution ratio; $\delta = 1.0$

K' factor; $K' = 0.598 \times \delta - 0.18 \times \delta^2 - 0.21 =$ **0.208**

***K < K' - Compression reinforcement is not required***

Lever arm; $z = \min(0.95 \times d_{y\_n}, d_{y\_n}/2 \times (1 + \sqrt{(1 - 3.53 \times K)})) =$ **324.0** mm

Area of reinforcement required for bending; $A_{sy\_n\_m} = M_{y\_n} / (f_{yd} \times z) =$ **143** mm²/m

Minimum area of reinforcement required; $A_{sy\_n\_min} = \max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{y\_n}, 0.0013 \times b \times d_{y\_n}) =$ **455** mm²/m

Area of reinforcement required; $A_{sy\_n\_req} = \max(A_{sy\_n\_m}, A_{sy\_n\_min}) =$ **455** mm²/m

***PASS - Area of reinforcement provided exceeds area required***

**Check reinforcement spacing**

Reinforcement service stress; $\sigma_{sy\_n} = (f_{yk} / \gamma_S) \times \min((A_{sy\_n\_m}/A_{sy\_n}), 1.0) \times q_{SLS} / q =$ **20.0** N/mm²

Maximum allowable spacing (Table 7.3N); $s_{max\_y\_n} =$ **300** mm

Actual bar spacing; $s_{y\_n} =$ **250** mm

***PASS - The reinforcement spacing is acceptable***

**Shear capacity check at short span continuous support**

Shear force; $V_{x\_n} = q \times l_x / 2 =$ **63.0** kN/m

Effective depth factor (cl. 6.2.2); $k = \min(2.0, 1 + (200\ mm / d_{x\_n})^{0.5}) =$ **1.748**

Reinforcement ratio; $\rho_l = \min(0.02, A_{sx\_n} / (b \times d_{x\_n})) =$ **0.0023**

Minimum shear resistance (Exp. 6.3N); $V_{Rd,c\_min} = 0.035\ N/mm^2 \times k^{1.5} \times (f_{ck} / 1\ N/mm^2)^{0.5} \times b \times d_{x\_n}$

$V_{Rd,c\_min} =$ **144.4** kN/m

Shear resistance constant (cl. 6.2.2); $C_{Rd,c} = 0.18\ N/mm^2 / \gamma_C =$ **0.12** N/mm²

Shear resistance (Exp. 6.2a);

$V_{Rd,c\_x\_n} = \max(V_{Rd,c\_min}, C_{Rd,c} \times k \times (100 \times \rho_l \times (f_{ck} / 1\ N/mm^2))^{0.333} \times b \times d_{x\_n}) =$ **144.4** kN/m

***PASS - Shear capacity is adequate***

**Shear capacity check at long span continuous support**

Shear force; $V_{y\_n} = q \times l_x / 2 = \mathbf{63.0}$ kN/m

Effective depth factor (cl. 6.2.2); $k = \min(2.0, 1 + (200\text{ mm} / d_{y\_n})^{0.5}) = \mathbf{1.766}$

Reinforcement ratio; $\rho_l = \min(0.02, A_{sy\_n} / (b \times d_{y\_n})) = \mathbf{0.0024}$

Minimum shear resistance (Exp. 6.3N); $V_{Rd,c\_min} = 0.035\text{ N/mm}^2 \times k^{1.5} \times (f_{ck} / 1\text{ N/mm}^2)^{0.5} \times b \times d_{y\_n}$

$V_{Rd,c\_min} = \mathbf{140.0}$ kN/m

Shear resistance constant (cl. 6.2.2); $C_{Rd,c} = 0.18\text{ N/mm}^2 / \gamma_C = \mathbf{0.12}\text{ N/mm}^2$

Shear resistance (Exp. 6.2a);

$V_{Rd,c\_y\_n} = \max(V_{Rd,c\_min}, C_{Rd,c} \times k \times (100 \times \rho_l \times (f_{ck} / 1\text{ N/mm}^2))^{0.333} \times b \times d_{y\_n}) = \mathbf{140.0}$ kN/m

***PASS - Shear capacity is adequate***

**Basic span-to-depth deflection ratio check (cl. 7.4.2)**

Reference reinforcement ratio; $\rho_0 = (f_{ck} / 1\text{ N/mm}^2)^{0.5} / 1000 = \mathbf{0.0050}$

Required tension reinforcement ratio; $\rho = \max(0.0035, A_{sx\_p\_req} / (b \times d_{x\_p})) = \mathbf{0.0035}$

Required compression reinforcement ratio; $\rho' = A_{scx\_p\_req} / (b \times d_{x\_p}) = \mathbf{0.0000}$

Stuctural system factor (Table 7.4N); $K_\delta = \mathbf{1.5}$

Basic limit span-to-depth ratio (Exp. 7.16);

$ratio_{lim\_x\_bas} = K_\delta \times [11 + 1.5 \times (f_{ck}/1\text{ N/mm}^2)^{0.5} \times \rho_0/\rho + 3.2 \times (f_{ck}/1\text{ N/mm}^2)^{0.5} \times (\rho_0/\rho - 1)^{1.5}] = \mathbf{39.31}$

Mod span-to-depth ratio limit;

$ratio_{lim\_x} = \min(40 \times K_\delta, \min(1.5, (500\text{ N/mm}^2 / f_{yk}) \times (A_{sx\_p} / A_{sx\_p\_m})) \times ratio_{lim\_x\_bas}) = \mathbf{58.96}$

Actual span-to-eff. depth ratio; $ratio_{act\_x} = l_x / d_{x\_p} = \mathbf{14.01}$

***PASS - Actual span-to-effective depth ratio is acceptable***

**Reinforcement summary**

Midspan in short span direction; **16 mm dia. bars at 250 mm centres B1**

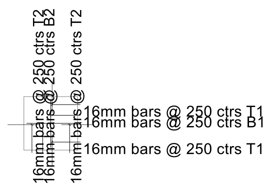
Midspan in long span direction; **16 mm dia. bars at 250 mm centres B2**

Continuous support in short span direction; **16 mm dia. bars at 250 mm centres T1**

Continuous support in long span direction; **16 mm dia. bars at 250 mm centres T2**

**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.



16mm bars @ 250 ctrs T2
16mm bars @ 250 ctrs B2
16mm bars @ 250 ctrs T2
16mm bars @ 250 ctrs T1
16mm bars @ 250 ctrs B1
16mm bars @ 250 ctrs T1


;

# 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 | $mm^2$ | 226 | 0 | 0.000 | PASS |
| Bottom reinforcement | $mm^2$ | 603 | 533 | 0.884 | PASS |
| Shear reinforcement | $mm^2/m$ | 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; $f_{ck}$ = **40** N/mm$^2$

Mean value of compressive cylinder strength; $f_{cm} = f_{ck} + 8$ N/mm$^2$ = **48** N/mm$^2$

Mean value of axial tensile strength; $f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck}/$ 1 N/mm$^2)^{2/3}$ = **3.5** N/mm$^2$

Secant modulus of elasticity of concrete; $E_{cm} = 22$ kN/mm$^2 \times (f_{cm}$ / 10 N/mm$^2)^{0.3} \times$ AAF = **35220** N/mm$^2$

Ultimate strain - Table 3.1; $\varepsilon_{cu2}$ = **0.0035**

Shortening strain - Table 3.1; $\varepsilon_{cu3}$ = **0.0035**

Effective compression zone height factor; $\lambda$ = **0.80**

Effective strength factor; $\eta$ = **1.00**

Coefficient $k_1$; $k_1$ = **0.40**

Coefficient $k_2$; $k_2 = 1.0 \times (0.6 + 0.0014 / \varepsilon_{cu2})$ = **1.00**

Coefficient $k_3$; $k_3$ = **0.40**

Coefficient $k_4$; $k_4 = 1.0 \times (0.6 + 0.0014 / \varepsilon_{cu2})$ = **1.00**

Partial factor for concrete -Table 2.1N; $\gamma_C$ = **1.50**

Compressive strength coefficient - cl.3.1.6(1); $\alpha_{cc}$ = **0.85**

Design compressive concrete strength - exp.3.15; $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **22.7** N/mm$^2$

Compressive strength coefficient - cl.3.1.6(1); $\alpha_{ccw}$ = **1.00**

Design compressive concrete strength - exp.3.15; $f_{cwd} = \alpha_{ccw} \times f_{ck} / \gamma_C$ = **26.7** N/mm$^2$

Maximum aggregate size; $h_{agg}$ = **20** mm

Density of reinforced concrete; $\rho$ = **2500** kg/m$^3$

Monolithic simple support moment factor; $\beta_1$ = **0.25**

**Reinforcement details**

Characteristic yield strength of reinforcement; $f_{yk}$ = **500** N/mm$^2$

Partial factor for reinforcing steel - Table 2.1N; $\gamma_S$ = **1.15**

Design yield strength of reinforcement; $f_{yd} = f_{yk} / \gamma_S$ = **435** N/mm$^2$

**Nominal cover to reinforcement**

Nominal cover to top reinforcement; $c_{nom\_t}$ = **30** mm

Nominal cover to bottom reinforcement; $c_{nom\_b}$ = **30** mm

Nominal cover to side reinforcement; $c_{nom\_s}$ = **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; $b_{min}$ = **120** mm

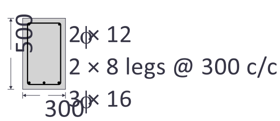
**<u>Section 1 - Main RC Beam</u>**

**Rectangular section details**

Section width; b = **300** mm

Section depth; h = **500** mm

***PASS - Minimum dimensions for fire resistance met***




**<u>Positive moment - section 6.1</u>**

Design bending moment; $M = M_{pos\_s1}$ = **100.0** kNm

Effective depth of tension reinforcement; d = **454** mm

Redistribution ratio; $\delta = \min(\delta_{pos\_s1}, 1)$ = **1.000**

$K = M / (b \times d^2 \times f_{ck})$ = **0.040**

$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2))$ = **0.207**

***K' > K - No compression reinforcement is required***

Lever arm; $z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d)$ = **431** mm

Depth of neutral axis; $x = 2 \times (d - z) / \lambda$ = **57** mm

Area of tension reinforcement required; $A_{s,req} = M / (f_{yd} \times z)$ = **533** mm$^2$

Tension reinforcement provided; $3 \times 16\phi$

Area of tension reinforcement provided; $A_{s,prov}$ = **603** mm$^2$

Minimum area of reinforcement - exp.9.1N; $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d$ = **249** mm$^2$

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{s,max} = 0.04 \times b \times h$ = **6000** mm$^2$

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

**Crack control - Section 7.3**

Maximum crack width; $w_k$ = **0.30** mm

Design value modulus of elasticity reinf – 3.2.7(4); $E_s$ = **200000** N/mm$^2$

Mean value of concrete tensile strength; $f_{ct,eff} = f_{ctm}$ = **3.5** N/mm$^2$

Stress distribution coefficient; $k_c$ = **0.4**

Non-uniform self-equilibrating stress coefficient; $k = \min(\max(1 + (300\text{ mm} - \min(h, b)) \times 0.35 / 500\text{ mm}, 0.65), 1)$ = **1.00**

Actual tension bar spacing; $s_{bar} = (b - (2 \times (c_{nom\_s} + \phi_{s1\_v}) + \phi_{s1\_b\_L1} \times N_{s1\_b\_L1})) / (N_{s1\_b\_L1} - 1) + \phi_{s1\_b\_L1}$ = **104** mm

Maximum stress permitted - Table 7.3N; $\sigma_s$ = **317** N/mm$^2$

Steel to concrete modulus of elast. ratio; $\alpha_{cr} = E_s / E_{cm}$ = **5.68**

Distance of the Elastic NA from bottom of beam; $y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1))$ = **246** mm

Area of concrete in the tensile zone; $A_{ct} = b \times y$ = **73870** mm$^2$

Minimum area of reinforcement required - exp.7.1; $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s$ = **327** mm$^2$

***PASS - Area of tension reinforcement provided exceeds minimum required for crack control***

Quasi-permanent moment; $M_{QP} = M_{pos\_QP\_s1}$ = **65.0**kNm

Permanent load ratio; $R_{PL} = M_{QP} / M$ = **0.65**

Service stress in reinforcement; $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL}$ = **250** N/mm$^2$

Maximum bar spacing - Tables 7.3N; $s_{bar,max}$ = **187.7** mm

***PASS - Maximum bar spacing exceeds actual bar spacing for crack control***

**Minimum bar spacing (Section 8.2)**

Top bar spacing; $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{s1\_v}) + \phi_{s1\_t\_L1} \times N_{s1\_t\_L1})) / (N_{s1\_t\_L1} - 1)$ = **200.0** mm

Minimum allowable top bar spacing; $s_{top,min} = \max(\phi_{s1\_t\_L1} \times k_{s1}, h_{agg} + k_{s2}, 20\text{mm})$ = **25.0** mm

***PASS - Actual bar spacing exceeds minimum allowable***

Bottom bar spacing; $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{s1\_v}) + \phi_{s1\_b\_L1} \times N_{s1\_b\_L1})) / (N_{s1\_b\_L1} - 1)$ = **88.0** mm

Minimum allowable bottom bar spacing; $s_{bot,min} = \max(\phi_{s1\_b\_L1} \times k_{s1}, h_{agg} + k_{s2}, 20\text{mm})$ = **25.0** mm

***PASS - Actual bar spacing exceeds minimum allowable***

**<u>Section in shear (section 6.2)</u>**

Angle of comp. shear strut for maximum shear; $\theta_{max}$ = 45 deg

Strength reduction factor - cl.6.2.3(3); $v_1 = 0.6 \times (1 - f_{ck} / 250\text{ N/mm}^2)$ = **0.504**

Compression chord coefficient - cl.6.2.3(3); $\alpha_{cw}$ = **1.00**

Minimum area of shear reinforcement - exp.9.5N; $A_{sv,min} = 0.08\text{ N/mm}^2 \times b \times (f_{ck} / 1\text{ N/mm}^2)^{0.5} / f_{yk}$ = **304** mm$^2$/m

Design shear force at support ; $V_{Ed,max} = V_{Ed,max\_s1}$ = **50** kN

Min lever arm in shear zone; $z$ = **431** mm

Maximum design shear resistance - exp.6.9; $V_{Rd,max} = \alpha_{cw} \times b \times z \times v_1 \times f_{cwd} / (\cot(\theta_{max}) + \tan(\theta_{max}))$ = **870** kN

***PASS - Design shear force at support is less than maximum design shear resistance***

Design shear force ; $V_{Ed}$ = **50** kN

Design shear stress; $v_{Ed} = V_{Ed} / (b \times z)$ = **0.386** N/mm$^2$

Angle of concrete compression strut - cl.6.2.3; $\theta = \min(\max(0.5 \times \text{Asin}(\min(2 \times v_{Ed} / (\alpha_{cw} \times f_{cwd} \times v_1),1)), 21.8\text{ deg}), 45\text{deg})$ = **21.8** deg

Area of shear reinforcement required - exp.6.8; $A_{sv,des} = v_{Ed} \times b / (f_{yd} \times \cot(\theta))$ = **107** mm$^2$/m

Area of shear reinforcement required; $A_{sv,req} = \max(A_{sv,min}, A_{sv,des})$ = **304** mm$^2$/m

Shear reinforcement provided; 2 × 8 legs @ 300 c/c

Area of shear reinforcement provided; $A_{sv,prov}$ = **335** $mm^2/m$

***PASS - Area of shear reinforcement provided exceeds minimum required***

Maximum longitudinal spacing - exp.9.6N; $s_{vl,max} = 0.75 \times d$ = **341** mm

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

;

Tekla Tedds
Kelly Mweu Changilwa
+254703206460
mweukelly@gmail.com



# EN RC PARKING COLUMN

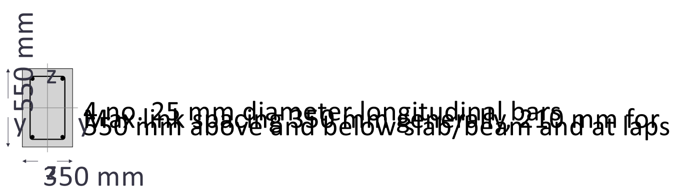
## RC COLUMN DESIGN (EN 1992)

**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 |




### 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)); $\gamma_C$ = **1.50**

Coefficient $\alpha_{cc}$ (3.1.6(1)); $\alpha_{cc}$ = **0.85**

Maximum aggregate size; $d_g$ = **20** mm

**Reinforcement details**

Nominal cover to links; $c_{nom}$ = **50** mm

Longitudinal bar diameter; $\phi$ = **25** mm

| | |
|---|---|
| Link diameter; | $\phi_v$ = **8** mm |
| Total number of longitudinal bars; | N = **4** |
| No. of bars per face parallel to y axis; | $N_y$ = ;**2** |
| No. of bars per face parallel to z axis; | $N_z$ = ;**2** |
| Area of longitudinal reinforcement; | $A_s = N \times \pi \times \phi^2 / 4$ = **1963** mm$^2$ |
| Characteristic yield strength; | $f_{yk}$ = **500** N/mm$^2$ |
| Partial safety factor for reinft (2.4.2.4(1)); | $\gamma_S$ = **1.15** |
| Modulus of elasticity of reinft (3.2.7(4)); | $E_s$ = **200** kN/mm$^2$ |

**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; | $\mu_{fi}$ = **0.70** |

**Axial load and bending moments from frame analysis**

| | |
|---|---|
| Design axial load; | $N_{Ed}$ = **2000.0** kN |
| Moment about y axis at top; | $M_{topy}$ = ;**75.0**; kNm |
| Moment about y axis at bottom; | $M_{btmy}$ = ;**75.0**; kNm |
| Moment about z axis at top; | $M_{topz}$ = ;**50.0**; kNm |
| Moment about z axis at bottom; | $M_{btmz}$ = ;**50.0**; kNm |

**Column effective lengths**

| | |
|---|---|
| Effective length for buckling about y axis; | $l_{0y}$ = ;**3000**; mm |
| Effective length for buckling about z axis; | $l_{0z}$ = ;**3000**; mm |

**Calculated column properties**

**Concrete properties**

| | |
|---|---|
| Area of concrete; | $A_c = h \times b$ = **192500** mm$^2$ |
| Characteristic compression cylinder strength; | $f_{ck}$ = **40** N/mm$^2$ |
| Design compressive strength (3.1.6(1)); | $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **22.7** N/mm$^2$ |
| Mean value of cylinder strength (Table 3.1); | $f_{cm} = f_{ck}$ + 8 MPa = **48.0** N/mm$^2$ |
| Secant modulus of elasticity (Table 3.1); | $E_{cm}$ = 22000 MPa $\times$ ($f_{cm}$ / 10 MPa)$^{0.3}$ = **35.2** kN/mm$^2$ |

**Rectangular stress block factors**

| | |
|---|---|
| Depth factor (3.1.7(3)); | $\lambda_{sb}$ = 0.8 |
| Stress factor (3.1.7(3)); | $\eta$ = 1.0 |

**Strain limits**

| | |
|---|---|
| Compression strain limit (Table 3.1); | $\varepsilon_{cu3}$ = **0.00350** |
| Pure compression strain limit (Table 3.1); | $\varepsilon_{c3}$ = **0.00175** |

**Design yield strength of reinforcement**

| | |
|---|---|
| Design yield strength (3.2.7(2)); | $f_{yd} = f_{yk} / \gamma_S$ = **434.8** N/mm$^2$ |

**Check nominal cover for fire and bond requirements**

| | |
|---|---|
| Min. cover reqd for bond (to links) (4.4.1.2(3)); | $c_{min,b} = \max(\phi_v, \phi - \phi_v)$ = **17** mm |
| Min axis distance for fire (EN1992-1-2 T 5.2a); | $a_{fi}$ = **40** mm |
| Allowance for deviations from min cover (4.4.1.3); | $\Delta c_{dev}$ = **10** mm |
| Min allowable nominal cover; | $c_{nom\_min} = \max(a_{fi} - \phi / 2 - \phi_v, c_{min,b} + \Delta c_{dev})$ = **27.0** mm |

***PASS - the nominal cover is greater than the minimum required***

**Effective depths of bars for bending about y axis**

Area per bar; $A_{bar} = \pi \times \phi^2 / 4 = \mathbf{491}$ mm$^2$

Spacing of bars in faces parallel to z axis (c/c); $s_z = (h - 2 \times (c_{nom} + \phi_v) - \phi) / (N_z - 1) = \mathbf{409}$ mm

Layer 1 (in tension face); $d_{y1} = h - c_{nom} - \phi_v - \phi / 2 = \mathbf{480}$ mm

Layer 2; $d_{y2} = d_{y1} - s_z = \mathbf{70}$ mm

Effective depth about y axis; $d_y = d_{y1} = \mathbf{480}$ mm

**Effective depths of bars for bending about z axis**

Area of per bar; $A_{bar} = \pi \times \phi^2 / 4 = \mathbf{491}$ mm$^2$

Spacing of bars in faces parallel to y axis (c/c); $s_y = (b - 2 \times (c_{nom} + \phi_v) - \phi) / (N_y - 1) = \mathbf{209}$ mm

Layer 1 (in tension face); $d_{z1} = b - c_{nom} - \phi_v - \phi / 2 = \mathbf{280}$ mm

Layer 2; $d_{z2} = d_{z1} - s_y = \mathbf{70}$ mm

Effective depth about z axis; $d_z = d_{z1} = \mathbf{280}$ mm

**Column slenderness about y axis**

Radius of gyration; $i_y = h / \sqrt{(12)} = \mathbf{15.9}$ cm

Slenderness ratio (5.8.3.2(1)); $\lambda_y = l_{0y} / i_y = \mathbf{18.9}$

**Column slenderness about z axis**

Radius of gyration; $i_z = b / \sqrt{(12)} = \mathbf{10.1}$ cm

Slenderness ratio (5.8.3.2(1)); $\lambda_z = l_{0z} / i_z = \mathbf{29.7}$

**<u>Design bending moments</u>**

**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); $e_{iy} = l_{0y} /400 = \mathbf{7.5}$ mm

Min end moment about y axis; $M_{01y} = \min(\text{abs}(M_{topy}), \text{abs}(M_{btmy})) + e_{iy} \times N_{Ed} = \mathbf{90.0}$ kNm

Max end moment about y axis; $M_{02y} = \max(\text{abs}(M_{topy}), \text{abs}(M_{btmy})) + e_{iy} \times N_{Ed} = \mathbf{90.0}$ kNm

**Slenderness limit for buckling about y axis (cl. 5.8.3.1)**

Factor A; $A = \mathbf{0.7}$

Mechanical reinforcement ratio; $\omega = A_s \times f_{yd} / (A_c \times f_{cd}) = \mathbf{0.196}$

Factor B; $B = \sqrt{(1 + 2 \times \omega)} = \mathbf{1.180}$

Moment ratio; $r_{my}$ = ;**1.000**

Factor C; $C_y = 1.7 - r_{my} = \mathbf{0.700}$

Relative normal force; $n = N_{Ed} / (A_c \times f_{cd}) = \mathbf{0.458}$

Slenderness limit; $\lambda_{limy} = 20 \times A \times B \times C_y / \sqrt{(n)} = \mathbf{17.1}$

***$\lambda_y$>=$\lambda_{limy}$ - Second order effects must be considered***

**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); $e_{iz} = l_{0z} /400 = \mathbf{7.5}$ mm

Min end moment about z axis; $M_{01z} = \min(\text{abs}(M_{topz}), \text{abs}(M_{btmz})) + e_{iz} \times N_{Ed} = \mathbf{65.0}$ kNm

Max end moment about z axis; $M_{02z} = \max(\text{abs}(M_{topz}), \text{abs}(M_{btmz})) + e_{iz} \times N_{Ed} = \mathbf{65.0}$ kNm

Slenderness limit for buckling about y axis (cl. 5.8.3.1)

Factor A; $A = \mathbf{0.7}$

Mechanical reinforcement ratio; $\omega = A_s \times f_{yd} / (A_c \times f_{cd}) = \mathbf{0.196}$

| | |
|---|---|
| Factor B; | $B = \sqrt{(1 + 2 \times \omega)}$ = **1.180** |
| Moment ratio; | $r_{mz}$ = ;**1.000** |
| Factor C; | $C_z = 1.7 - r_{mz}$ = **0.700** |
| Relative normal force; | $n = N_{Ed} / (A_c \times f_{cd})$ = **0.458** |
| Slenderness limit; | $\lambda_{limz} = 20 \times A \times B \times C_z / \sqrt{(n)}$ = **17.1** |

***$\lambda_z$>=$\lambda_{limz}$ - Second order effects must be considered***

**Local second order bending moment about y axis (cl. 5.8.8.2 & 5.8.8.3)**

| | |
|---|---|
| Relative humidity of ambient environment; | RH = **50** % |
| Column perimeter in contact with atmosphere; | u = **1800** mm |
| Age of concrete at loading; | $t_0$ = **28** day |
| Parameter $n_u$; | $n_u = 1 + \omega$ = **1.196** |
| Approx value of n at max moment of resistance; | $n_{bal}$ = **0.4** |
| Axial load correction factor; | $K_r = \min(1.0\ ,\ (n_u - n) / (n_u - n_{bal}))$ = **0.927** |
| Reinforcement design strain; | $\varepsilon_{yd} = f_{yd} / E_s$ = **0.00217** |
| Basic curvature; | $curve_{basic\_y} = \varepsilon_{yd} / (0.45 \times d_y)$ = **0.0000101** $mm^{-1}$ |
| Notional size of column; | $h_0 = 2 \times A_c / u$ = **214** mm |
| Factor $\alpha_1$ (Annex B.1(1)); | $\alpha_1 = (35\ MPa / f_{cm})^{0.7}$ = **0.802** |
| Factor $\alpha_2$ (Annex B.1(1)); | $\alpha_2 = (35\ MPa / f_{cm})^{0.2}$ = **0.939** |
| Relative humidity factor (Annex B.1(1)); | $\phi_{RH} = [1 + ((1 - RH / 100\%) / (0.1\ mm^{-1/3} \times (h_0)^{1/3})) \times \alpha_1] \times \alpha_2$ = **1.568** |
| Concrete strength factor (Annex B.1(1)); | $\beta_{fcm} = 16.8 \times (1\ MPa)^{1/2} / \sqrt{(f_{cm})}$ = **2.425** |
| Concrete age factor (Annex B.1(1)); | $\beta_{t0} = 1 / (0.1 + (t_0 / 1\ day)^{0.2})$ = **0.488** |
| Notional creep coefficient (Annex B.1(1)); | $\phi_0 = \phi_{RH} \times \beta_{fcm} \times \beta_{t0}$ = **1.857** |
| Final creep development factor; (at t = ∞); | $\beta_{c\infty}$ = 1.0 |
| Final creep coefficient (Annex B.1(1)); | $\phi_\infty = \phi_0 \times \beta_{c\infty}$ = **1.857** |
| Ratio of SLS to ULS moments; | $r_{My}$ = **0.80** |
| Effective creep ratio; | $\phi_{efy} = \phi_\infty \times r_{My}$ = **1.486** |
| Factor $\beta$; | $\beta_y = 0.35 + f_{ck} / 200\ MPa - \lambda_y / 150$ = **0.424** |
| Creep factor; | $K_{\phi y} = \max(1.0\ ,\ 1 + \beta_y \times \phi_{efy})$ = **1.630** |
| Modified curvature; | $curve_{mod\_y} = K_r \times K_{\phi y} \times curve_{basic\_y}$ = **0.0000152** $mm^{-1}$ |
| Curvature distribution factor; | c = **10** |
| Deflection; | $e_{2y} = curve_{mod\_y} \times l_{0y}^2 / c$ = **13.7** mm |
| Nominal 2nd order moment; | $M_{2y} = N_{Ed} \times e_{2y}$ = **27.4** kNm |

**Design bending moment about y axis (cl. 5.8.8.2 & 6.1(4))**

| | |
|---|---|
| Equivalent moment from frame analysis; | $M_{0ey} = \max(0.6 \times M_{02y} + 0.4 \times M_{01y},\ 0.4 \times M_{02y})$ = ;**90.0**; kNm |
| Design moment; | $M_{Edy} = \max(M_{02y},\ M_{0ey} + M_{2y},\ M_{01y} + 0.5 \times M_{2y},\ N_{Ed} \times \max(h/30,\ 20\ mm))$ |
| | $M_{Edy}$ = **117.4** kNm |

**Local second order bending moment about z axis (cl. 5.8.8.2 & 5.8.8.3)**

| | |
|---|---|
| Basic curvature; | $curve_{basic\_z} = \varepsilon_{yd} / (0.45 \times d_z)$ = **0.0000173** $mm^{-1}$ |
| Ratio of SLS to ULS moments; | $r_{Mz}$ = **0.80** |
| Effective creep ratio (5.8.4(2)); | $\phi_{efz} = \phi_\infty \times r_{Mz}$ = **1.486** |
| Factor $\beta$; | $\beta_z = 0.35 + f_{ck} / 200\ MPa - \lambda_z / 150$ = **0.352** |
| Creep factor; | $K_{\phi z} = \max(1.0\ ,\ 1 + \beta_z \times \phi_{efz})$ = **1.523** |

Modified curvature; $curve_{mod\_z} = K_r \times K_{\phi z} \times curve_{basic\_z} = \mathbf{0.0000244}$ mm$^{-1}$

Curvature distribution factor; $c = \mathbf{10}$

Deflection; $e_{2z} = curve_{mod\_z} \times l_{0z}^2 / c = \mathbf{22.0}$ mm

Nominal 2nd order moment; $M_{2z} = N_{Ed} \times e_{2z} = \mathbf{43.9}$ kNm

**Design bending moment about z axis (cl. 5.8.8.2 & 6.1(4))**

Equivalent moment from frame analysis; $M_{0ez} = \max(0.6 \times M_{02z} + 0.4 \times M_{01z}, 0.4 \times M_{02z})$ = ;**65.0**; kNm

Design moment; $M_{Edz} = \max(M_{02z}, M_{0ez} + M_{2z}, M_{01z} + 0.5 \times M_{2z}, N_{Ed} \times \max(b/30, 20\text{ mm}))$

$M_{Edz} = \mathbf{108.9}$ kNm

**<u>Moment capacity about y axis with axial load (2000.0 kN)</u>**

**Moment of resistance of concrete**

By iteration:-

Position of neutral axis; $y = \mathbf{310.0}$ mm

Concrete compression force (3.1.7(3)); $F_{yc} = \eta \times f_{cd} \times \min(\lambda_{sb} \times y , h) \times b = \mathbf{1967.6}$ kN

Moment of resistance; $M_{Rdyc} = F_{yc} \times [h / 2 - (\min(\lambda_{sb} \times y , h)) / 2] = \mathbf{297.1}$ kNm

**Moment of resistance of reinforcement**

Strain in layer 1; $\varepsilon_{y1} = \varepsilon_{cu3} \times (1 - d_{y1} / y) = \mathbf{-0.00191}$

Stress in layer 1; $\sigma_{y1} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{y1}) = \mathbf{-382.7}$ N/mm$^2$

Force in layer 1; $F_{y1} = N_y \times A_{bar} \times \sigma_{y1} = \mathbf{-375.7}$ kN

Moment of resistance of layer 1; $M_{Rdy1} = F_{y1} \times (h / 2 - d_{y1}) = \mathbf{76.8}$ kNm

Strain in layer 2; $\varepsilon_{y2} = \varepsilon_{cu3} \times (1 - d_{y2} / y) = \mathbf{0.00270}$

Stress in layer 2; $\sigma_{y2} = \min(f_{yd}, E_s \times \varepsilon_{y2}) - \eta \times f_{cd} = \mathbf{412.1}$ N/mm$^2$

Force in layer 2; $F_{y2} = N_y \times A_{bar} \times \sigma_{y2} = \mathbf{404.6}$ kN

Moment of resistance of layer 2; $M_{Rdy2} = F_{y2} \times (h / 2 - d_{y2}) = \mathbf{82.7}$ kNm

Resultant concrete/steel force; $F_y = \mathbf{1996.5}$ kN

***PASS - This is within half of one percent of the applied axial load***

**Combined moment of resistance**

Moment of resistance about y axis; $M_{Rdy} = \mathbf{456.7}$ kNm

***PASS - The moment capacity about the y axis exceeds the design bending moment***

**<u>Moment capacity about z axis with axial load (2000.0 kN)</u>**

**Moment of resistance of concrete**

By iteration:-

Position of neutral axis; $z = \mathbf{191.8}$ mm

Concrete compression force (3.1.7(3)); $F_{zc} = \eta \times f_{cd} \times \min(\lambda_{sb} \times z , b) \times h = \mathbf{1913.0}$ kN

Moment of resistance; $M_{Rdzc} = F_{zc} \times [b / 2 - (\min(\lambda_{sb} \times z , b)) / 2] = \mathbf{188.0}$ kNm

**Moment of resistance of reinforcement**

Strain in layer 1; $\varepsilon_{z1} = \varepsilon_{cu3} \times (1 - d_{z1} / z) = \mathbf{-0.00160}$

Stress in layer 1; $\sigma_{z1} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{z1}) = \mathbf{-320.0}$ N/mm$^2$

Force in layer 1; $F_{z1} = N_z \times A_{bar} \times \sigma_{z1} = \mathbf{-314.2}$ kN

Moment of resistance of layer 1; $M_{Rdz1} = F_{z1} \times (b / 2 - d_{z1}) = \mathbf{32.8}$ kNm

Strain in layer 2; $\varepsilon_{z2} = \varepsilon_{cu3} \times (1 - d_{z2} / z) = \mathbf{0.00221}$

Stress in layer 2; $\sigma_{z2} = \min(f_{yd}, E_s \times \varepsilon_{z2}) - \eta \times f_{cd} = \mathbf{412.1}$ N/mm$^2$

Force in layer 2; $F_{z2} = N_z \times A_{bar} \times \sigma_{z2} = \mathbf{404.6}$ kN

Moment of resistance of layer 2; $M_{Rdz2} = F_{z2} \times (b / 2 - d_{z2}) = \mathbf{42.3}$ kNm

Resultant concrete/steel force; $F_z = \mathbf{2003.5}$ kN

***PASS - This is within half of one percent of the applied axial load***

**Combined moment of resistance**

Moment of resistance about z axis; $M_{Rdz} = \mathbf{263.1}$ kNm

***PASS - The moment capacity about the z axis exceeds the design bending moment***

**Biaxial bending**

**Determine if a biaxial bending check is required (5.8.9(3))**

Ratio of column slenderness ratios; $ratio_{\lambda} = \max(\lambda_y, \lambda_z) / \min(\lambda_y, \lambda_z) = \mathbf{1.57}$

Eccentricity in direction of y axis; $e_y = M_{Edz} / N_{Ed} = \mathbf{54.5}$ mm

Eccentricity in direction of z axis; $e_z = M_{Edy} / N_{Ed} = \mathbf{58.7}$ mm

Equivalent depth; $h_{eq} = i_y \times \sqrt{(12)} = \mathbf{550}$ mm

Equivalent width; $b_{eq} = i_z \times \sqrt{(12)} = \mathbf{350}$ mm

Relative eccentricity in direction of y axis; $e_{rel\_y} = e_y / b_{eq} = \mathbf{0.156}$

Relative eccentricity in direction of z axis; $e_{rel\_z} = e_z / h_{eq} = \mathbf{0.107}$

Ratio of relative eccentricities; $ratio_e = \min(e_{rel\_y}, e_{rel\_z}) / \max(e_{rel\_y}, e_{rel\_z}) = \mathbf{0.686}$

***$ratio_e$ > 0.2 - Biaxial bending check is required***

**Biaxial bending (5.8.9(4))**

Design axial resistance of section; $N_{Rd} = (A_c \times f_{cd}) + (A_s \times f_{yd}) = \mathbf{5217.0}$ kN

Ratio of applied to resistance axial loads; $ratio_N = N_{Ed} / N_{Rd} = \mathbf{0.383}$

Exponent a; $a = \mathbf{1.24}$

Biaxial bending utilisation; $UF = (M_{Edy} / M_{Rdy})^a + (M_{Edz} / M_{Rdz})^a = \mathbf{0.523}$

***PASS - The biaxial bending capacity is adequate***

;

Kelly Mweu Changilwa
+254703206460
mweukelly@gmail.com

# EN PAD FOOTING

## 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 | $kN/m^2$ | 766.9 | 226.1 | 0.295 | Pass |
| **Description** | **Unit** | **Provided** | **Required** | **Utilisation** | **Result** |
| Reinforcement x-direction | $mm^2$ | 5890 | 5706 | 0.969 | Pass |
| Reinforcement y-direction | $mm^2$ | 5890 | 5624 | 0.955 | Pass |
| **Description** | **Unit** | **Allowable** | **Actual** | **Utilisation** | **Result** |
| Punching shear | $N/mm^2$ | 4.284 | 0.366 | 0.086 | Pass |
| | | | | | |
| | | | | | |

**Pad foundation details**

Length of foundation; $L_x$ = **1800** mm

Width of foundation; $L_y$ = **1800** mm

Foundation area; $A = L_x \times L_y$ = **3.240** $m^2$

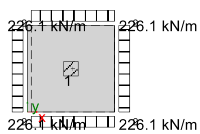
Depth of foundation; h = **1800** mm

Depth of soil over foundation; $h_{soil}$ = **2200** mm

Level of water; $h_{water}$ = **0** mm

Density of water; $\gamma_{water}$ = **9.8** $kN/m^3$

Density of concrete; $\gamma_{conc}$ = **24.5** $kN/m^3$




**Column no.1 details**

Length of column; $l_{x1}$ = **300** mm

Width of column; $l_{y1}$ = **300** mm

position in x-direction; $x_1$ = **900** mm

position in y-direction; $y_1$ = **900** mm

**Soil properties**

Density of soil; $\gamma_{soil}$ = **20.0** kN/m$^3$

Characteristic cohesion; $c'_k$ = **0** kN/m$^2$

Characteristic effective shear resistance angle; $\phi'_k$ = **25** deg

Characteristic friction angle; $\delta_k$ = **19.3** deg

**Foundation loads**

Self weight; $F_{swt} = h \times \gamma_{conc}$ = **44.1** kN/m$^2$

Soil weight; $F_{soil} = h_{soil} \times \gamma_{soil}$ = **44.0** kN/m$^2$

**Column no.1 loads**

Permanent axial load; $F_{Gz1}$ = **200.0** kN

Variable axial load; $F_{Qz1}$ = **165.0** kN

Permanent moment in x-direction; $M_{Gx1}$ = **15.0** kNm

Variable moment in x-direction; $M_{Qx1}$ = **10.0** kNm

**Design approach 1**

**Partial factors on actions - Combination1**

Partial factor set; A1

Permanent unfavourable action - Table A.3; $\gamma_G$ = **1.35**

Permanent favourable action - Table A.3; $\gamma_{Gf}$ = **1.00**

Kelly Mweu Changilwa
+254703206460
mweukelly@gmail.com

Variable unfavourable action - Table A.3; $\gamma_Q$ = **1.50**

Variable favourable action - Table A.3; $\gamma_{Qf}$ = **0.00**

**Partial factors for soil parameters - Combination1**

Soil factor set; M1

Angle of shearing resistance - Table A.4; $\gamma_{\phi'}$ = **1.00**

Effective cohesion - Table A.4; $\gamma_{c'}$ = **1.00**

Weight density - Table A.4; $\gamma_{\gamma}$ = **1.00**

**Partial factors for spread foundations - Combination1**

Resistance factor set; R1

Bearing - Table A.5; $\gamma_{R.v}$ = **1.00**

Sliding - Table A.5; $\gamma_{R.h}$ = **1.00**

**Bearing resistance (Section 6.5.2)**

**Forces on foundation**

Force in z-direction; $F_{dz} = \gamma_G \times (A \times (F_{swt} + F_{soil}) + F_{Gz1}) + \gamma_Q \times F_{Qz1}$ = **902.8** kN

**Moments on foundation**

Moment in x-direction; $M_{dx} = \gamma_G \times (A \times (F_{swt} + F_{soil}) \times L_x / 2 + F_{Gz1} \times x_1) + \gamma_G \times M_{Gx1} + \gamma_Q \times F_{Qz1} \times x_1 + \gamma_Q \times M_{Qx1}$ = **847.8** kNm

Moment in y-direction; $M_{dy} = \gamma_G \times (A \times (F_{swt} + F_{soil}) \times L_y / 2 + F_{Gz1} \times y_1) + \gamma_Q \times F_{Qz1} \times y_1$ = **812.6** kNm

**Eccentricity of base reaction**

Eccentricity of base reaction in x-direction; $e_x = M_{dx} / F_{dz} - L_x / 2$ = **39** mm

Eccentricity of base reaction in y-direction; $e_y = M_{dy} / F_{dz} - L_y / 2$ = **0** mm

**Effective area of base**

Effective length; $L'_x = L_x - 2 \times e_x$ = **1722** mm

Effective width; $L'_y = L_y - 2 \times e_y$ = **1800** mm

Effective area; $A' = L'_x \times L'_y$ = **3.099** $m^2$

**Pad base pressure**

Design base pressure; $f_{dz} = F_{dz} / A'$ = **291.3** $kN/m^2$

**Ultimate bearing capacity under drained conditions (Annex D.4)**

Design angle of shearing resistance; $\phi'_d = \mathrm{atan}(\tan(\phi'_k) / \gamma_{\phi'})$ = **25.000** deg

Design effective cohesion; $c'_d = c'_k / \gamma_{c'}$ = **0.000** $kN/m^2$

Effective overburden pressure; $q = (h + h_{soil}) \times \gamma_{soil} - h_{water} \times \gamma_{water}$ = **80.000** $kN/m^2$

Design effective overburden pressure; $q' = q / \gamma_{\gamma}$ = **80.000** $kN/m^2$

Bearing resistance factors; $N_q = \mathrm{Exp}(\pi \times \tan(\phi'_d)) \times (\tan(45\ \mathrm{deg} + \phi'_d / 2))^2$ = **10.662**

$N_c = (N_q - 1) \times \cot(\phi'_d)$ = **20.721**

$N_{\gamma} = 2 \times (N_q - 1) \times \tan(\phi'_d)$ = **9.011**

Foundation shape factors; $s_q = 1 + (L'_x / L'_y) \times \sin(\phi'_d)$ = **1.404**

$s_{\gamma} = 1 - 0.3 \times (L'_x / L'_y)$ = **0.713**

$s_c = (s_q \times N_q - 1) / (N_q - 1)$ = **1.446**

Load inclination factors; H = **0.0** kN

$m_y = [2 + (L'_y / L'_x)] / [1 + (L'_y / L'_x)] = \mathbf{1.489}$

$m_x = [2 + (L'_x / L'_y)] / [1 + (L'_x / L'_y)] = \mathbf{1.511}$

$m = m_x = \mathbf{1.511}$

$i_q = [1 - H / (F_{dz} + A' \times c'_d \times \cot(\phi'_d))]^m = \mathbf{1.000}$

$i_\gamma = [1 - H / (F_{dz} + A' \times c'_d \times \cot(\phi'_d))]^{m+1} = \mathbf{1.000}$

$i_c = i_q - (1 - i_q) / (N_c \times \tan(\phi'_d)) = \mathbf{1.000}$

Ultimate bearing capacity; $n_f = c'_d \times N_c \times s_c \times i_c + q' \times N_q \times s_q \times i_q + 0.5 \times \gamma_{soil} \times L'_x \times N_\gamma \times s_\gamma \times i_\gamma = \mathbf{1308.4}$ kN/m$^2$

***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; $\gamma_G = \mathbf{1.00}$

Permanent favourable action - Table A.3; $\gamma_{Gf} = \mathbf{1.00}$

Variable unfavourable action - Table A.3; $\gamma_Q = \mathbf{1.30}$

Variable favourable action - Table A.3; $\gamma_{Qf} = \mathbf{0.00}$

**Partial factors for soil parameters - Combination2**

Soil factor set; M2

Angle of shearing resistance - Table A.4; $\gamma_{\phi'} = \mathbf{1.25}$

Effective cohesion - Table A.4; $\gamma_{c'} = \mathbf{1.25}$

Weight density - Table A.4; $\gamma_\gamma = \mathbf{1.00}$

**Partial factors for spread foundations - Combination2**

Resistance factor set; R1

Bearing - Table A.5; $\gamma_{R.v} = \mathbf{1.00}$

Sliding - Table A.5; $\gamma_{R.h} = \mathbf{1.00}$

**Bearing resistance (Section 6.5.2)**

**Forces on foundation**

Force in z-direction; $F_{dz} = \gamma_G \times (A \times (F_{swt} + F_{soil}) + F_{Gz1}) + \gamma_Q \times F_{Qz1} = \mathbf{699.9}$ kN

**Moments on foundation**

Moment in x-direction; $M_{dx} = \gamma_G \times (A \times (F_{swt} + F_{soil}) \times L_x / 2 + F_{Gz1} \times x_1) + \gamma_G \times M_{Gx1} + \gamma_Q \times F_{Qz1} \times x_1 + \gamma_Q \times M_{Qx1} = \mathbf{657.9}$ kNm

Moment in y-direction; $M_{dy} = \gamma_G \times (A \times (F_{swt} + F_{soil}) \times L_y / 2 + F_{Gz1} \times y_1) + \gamma_Q \times F_{Qz1} \times y_1 = \mathbf{629.9}$ kNm

**Eccentricity of base reaction**

Eccentricity of base reaction in x-direction; $e_x = M_{dx} / F_{dz} - L_x / 2 = \mathbf{40}$ mm

Eccentricity of base reaction in y-direction; $e_y = M_{dy} / F_{dz} - L_y / 2 = \mathbf{0}$ mm

**Effective area of base**

Effective length; $L'_x = L_x - 2 \times e_x = \mathbf{1720}$ mm

Effective width; $L'_y = L_y - 2 \times e_y = \mathbf{1800}$ mm

Effective area; $A' = L'_x \times L'_y = \mathbf{3.096}$ m$^2$

**Pad base pressure**

Design base pressure; $f_{dz} = F_{dz} / A' = \mathbf{226.1}$ kN/m$^2$

**Ultimate bearing capacity under drained conditions (Annex D.4)**

Design angle of shearing resistance; $\phi'_d = \text{atan}(\tan(\phi'_k) / \gamma_{\phi'}) = \mathbf{20.458}$ deg

Design effective cohesion; $c'_d = c'_k / \gamma_{c'} = \mathbf{0.000}$ kN/m$^2$

Effective overburden pressure; $q = (h + h_{soil}) \times \gamma_{soil} - h_{water} \times \gamma_{water} = \mathbf{80.000}$ kN/m$^2$

Design effective overburden pressure; $q' = q / \gamma_\gamma = \mathbf{80.000}$ kN/m$^2$

Bearing resistance factors; $N_q = \text{Exp}(\pi \times \tan(\phi'_d)) \times (\tan(45 \text{ deg} + \phi'_d / 2))^2 = \mathbf{6.698}$

$N_c = (N_q - 1) \times \cot(\phi'_d) = \mathbf{15.273}$

$N_\gamma = 2 \times (N_q - 1) \times \tan(\phi'_d) = \mathbf{4.251}$

Foundation shape factors; $s_q = 1 + (L'_x / L'_y) \times \sin(\phi'_d) = \mathbf{1.334}$

$s_\gamma = 1 - 0.3 \times (L'_x / L'_y) = \mathbf{0.713}$

$s_c = (s_q \times N_q - 1) / (N_q - 1) = \mathbf{1.393}$

Load inclination factors; $H = \mathbf{0.0}$ kN

$m_y = [2 + (L'_y / L'_x)] / [1 + (L'_y / L'_x)] = \mathbf{1.489}$

$m_x = [2 + (L'_x / L'_y)] / [1 + (L'_x / L'_y)] = \mathbf{1.511}$

$m = m_x = \mathbf{1.511}$

$i_q = [1 - H / (F_{dz} + A' \times c'_d \times \cot(\phi'_d))]^m = \mathbf{1.000}$

$i_\gamma = [1 - H / (F_{dz} + A' \times c'_d \times \cot(\phi'_d))]^{m+1} = \mathbf{1.000}$

$i_c = i_q - (1 - i_q) / (N_c \times \tan(\phi'_d)) = \mathbf{1.000}$

Ultimate bearing capacity; $n_f = c'_d \times N_c \times s_c \times i_c + q' \times N_q \times s_q \times i_q + 0.5 \times \gamma_{soil} \times L'_x \times N_\gamma \times s_\gamma \times i_\gamma = \mathbf{766.9}$ kN/m$^2$

***PASS - Ultimate bearing capacity exceeds design base pressure***

<u>**PAD FOUNDATION EXAMPLE**</u>

**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; $f_{ck} = \mathbf{40}$ N/mm$^2$

Characteristic compressive cube strength; $f_{ck,cube} = \mathbf{50}$ N/mm$^2$

Mean value of compressive cylinder strength; $f_{cm} = f_{ck} + 8$ N/mm$^2$ $= \mathbf{48}$ N/mm$^2$

Mean value of axial tensile strength; $f_{ctm} = 0.3$ N/mm$^2$ $\times (f_{ck}/ 1 \text{ N/mm}^2)^{2/3} = \mathbf{3.5}$ N/mm$^2$

5% fractile of axial tensile strength; $f_{ctk,0.05} = 0.7 \times f_{ctm} = \mathbf{2.5}$ N/mm$^2$

Secant modulus of elasticity of concrete; $E_{cm} = 22 \text{ kN/mm}^2 \times [f_{cm}/10 \text{ N/mm}^2]^{0.3} = \mathbf{35220}$ N/mm$^2$

Partial factor for concrete (Table 2.1N); $\gamma_C = \mathbf{1.50}$

Compressive strength coefficient (cl.3.1.6(1)); $\alpha_{cc} = \mathbf{0.85}$

Design compressive concrete strength (exp.3.15); $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = \mathbf{22.7}$ N/mm$^2$

Tens.strength coeff.for plain concrete (cl.12.3.1(1)); $\alpha_{ct,pl} = \mathbf{0.80}$

Des.tens.strength for plain concrete (exp.12.1); $f_{ctd,pl} = \alpha_{ct,pl} \times f_{ctk,0.05} / \gamma_C = \mathbf{1.3}$ N/mm$^2$

Maximum aggregate size; $h_{agg} = \mathbf{20}$ mm

Ultimate strain - Table 3.1; $\varepsilon_{cu2} = \mathbf{0.0035}$

| Tekla Tedds<br>Kelly Mweu Changilwa<br>+254703206460<br>mweukelly@gmail.com | Project<br>REVIEW AND ANALYSIS OF AN UNDERGROUND | | | Job Ref.<br>FCE 590 | |
|---|---|---|---|---|---|
| | Section<br>HFB STRUCTURAL ANALYSIS | | | Sheet no./rev.<br>89 | |
| | Calc. by<br>KELLY C. | Date<br>03/05/2023 | Chk'd<br>PROF | By<br>MUMENYA | |

Shortening strain - Table 3.1; $\varepsilon_{cu3}$ = **0.0035**

Effective compression zone height factor; $\lambda$ = **0.80**

Effective strength factor; $\eta$ = **1.00**

Bending coefficient $k_1$; $K_1$ = **0.40**

Bending coefficient $k_2$; $K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ =**1.00**

Bending coefficient $k_3$; $K_3$ =**0.40**

Bending coefficient $k_4$; $K_4 =1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ = **1.00**

**Reinforcement details**

Characteristic yield strength of reinforcement; $f_{yk}$ = **500** N/mm$^2$

Modulus of elasticity of reinforcement; $E_s$ = **210000** N/mm$^2$

Partial factor for reinforcing steel (Table 2.1N); $\gamma_S$ = **1.15**

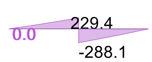
Design yield strength of reinforcement; $f_{yd} = f_{yk} / \gamma_S$ = **435** N/mm$^2$

Nominal cover to top of foundation; $c_{nom\_t}$ = **50** mm

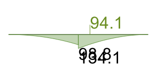
Nominal cover to bottom of foundation; $c_{nom\_b}$ = **50** mm

Nominal cover to side of foundation; $c_{nom\_s}$ = **50** mm

**Shear diagram, x axis (kN)**




**Moment diagram, x axis (kNm)**




**Rectangular section in flexure (Section 6.1)**

Design bending moment; $M_{Ed.x.max}$ = **94.1** kNm

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{x.bot} / 2$ = **1738** mm

$K = M_{Ed.x.max} / (L_y \times d^2 \times f_{ck})$ = **0.000**

$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$

$K'$ = **0.207**

***K' > K - No compression reinforcement is required***

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = **1651** mm

Depth of neutral axis; $x = 2.5 \times (d - z)$ = **217** mm

Area of tension reinforcement required; $A_{sx.bot.req} = M_{Ed.x.max} / (f_{yd} \times z)$ = **131** mm$^2$

Tension reinforcement provided; 12 No.25 $\phi$ bars bottom (150 c/c)

Area of tension reinforcement provided; $A_{sx.bot.prov}$ = **5890** mm$^2$

Minimum area of reinforcement (exp.9.1N); $A_{s.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times L_y \times d$ = **5706** mm$^2$

Maximum area of reinforcement (cl.9.2.1.1(3)); $A_{s.max} = 0.04 \times L_y \times d =$ **125100** $mm^2$

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

**Crack control (Section 7.3)**

Limiting crack width; $w_{max} =$ **0.3** mm

Variable load factor (EN1990 – Table A1.1); $\psi_2 =$ **0.3**

Serviceability bending moment; $M_{sls.x.max} =$ **45.8** kNm

Tensile stress in reinforcement; $\sigma_s = M_{sls.x.max} / (A_{sx.bot.prov} \times z) =$ **4.7** $N/mm^2$

Load duration factor; $k_t =$ **0.4**

Effective depth of concrete in tension; $h_{c.ef} = \min(2.5 \times (h - d), (h - x) / 3, h / 2) =$ **156** mm

Effective area of concrete in tension; $A_{c.eff} = h_{c.ef} \times L_y =$ **281250** $mm^2$

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm} =$ **3.5** $N/mm^2$

Reinforcement ratio; $\rho_{p.eff} = A_{sx.bot.prov} / A_{c.eff} =$ **0.021**

Modular ratio; $\alpha_e = E_s / E_{cm} =$ **5.962**

Bond property coefficient; $k_1 =$ **0.8**

Strain distribution coefficient; $k_2 =$ **0.5**

$k_3 = 3.4 =$**3.4**

$k_4 =$ **0.425**

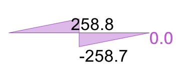
Maximum crack spacing (exp.7.11); $s_{r.max} = k_3 \times c_{nom\_b} + k_1 \times k_2 \times k_4 \times \phi_{x.bot} / \rho_{p.eff} =$ **373** mm

Maximum crack width (exp.7.8); $w_k = s_{r.max} \times \max([\sigma_s – k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff})] / E_s,$
$0.6 \times \sigma_s / E_s) =$ **0.005** mm

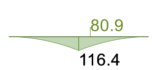
***PASS - Maximum crack width is less than limiting crack width***

Library item: Crack width output

**Shear diagram, y axis (kN)**




**Moment diagram, y axis (kNm)**




**Rectangular section in flexure (Section 6.1)**

Design bending moment; $M_{Ed.y.max} =$ **80.9** kNm

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2 =$ **1713** mm

$K = M_{Ed.y.max} / (L_x \times d^2 \times f_{ck}) =$ **0.000**

$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$

$K' =$ **0.207**

***K' > K - No compression reinforcement is required***

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d =$ **1627** mm

Depth of neutral axis; $x = 2.5 \times (d - z) =$ **214** mm

Area of tension reinforcement required; $A_{sy.bot.req} = M_{Ed.y.max} / (f_{yd} \times z) =$ **114** mm$^2$

Tension reinforcement provided; 12 No.25 $\phi$ bars bottom (150 c/c)

Area of tension reinforcement provided; $A_{sy.bot.prov} =$ **5890** mm$^2$

Minimum area of reinforcement (exp.9.1N); $A_{s.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times L_x \times d =$ **5624** mm$^2$

Maximum area of reinforcement (cl.9.2.1.1(3)); $A_{s.max} = 0.04 \times L_x \times d =$ **123300** mm$^2$

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

**Crack control (Section 7.3)**

Limiting crack width; $w_{max} =$ **0.3** mm

Variable load factor (EN1990 – Table A1.1); $\psi_2 =$ **0.3**

Serviceability bending moment; $M_{sls.y.max} =$ **39** kNm

Tensile stress in reinforcement; $\sigma_s = M_{sls.y.max} / (A_{sy.bot.prov} \times z) =$ **4.1** N/mm$^2$

Load duration factor; $k_t =$ **0.4**

Effective depth of concrete in tension; $h_{c.ef} = \min(2.5 \times (h - d), (h - x) / 3, h / 2) =$ **219** mm

Effective area of concrete in tension; $A_{c.eff} = h_{c.ef} \times L_x =$ **393750** mm$^2$

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm} =$ **3.5** N/mm$^2$

Reinforcement ratio; $\rho_{p.eff} = A_{sy.bot.prov} / A_{c.eff} =$ **0.015**

Modular ratio; $\alpha_e = E_s / E_{cm} =$ **5.962**

Bond property coefficient; $k_1 =$ **0.8**

Strain distribution coefficient; $k_2 =$ **0.5**

$k_3 = 3.4 =$**3.4**

$k_4 =$ **0.425**

Maximum crack spacing (exp.7.11); $s_{r.max} = k_3 \times (c_{nom\_b} + \phi_{x.bot}) + k_1 \times k_2 \times k_4 \times \phi_{y.bot} / \rho_{p.eff} =$ **539** mm

Maximum crack width (exp.7.8); $w_k = s_{r.max} \times \max([\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff})] / E_s,$
$0.6 \times \sigma_s / E_s) =$ **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); $v = 0.6 \times [1 - f_{ck} / 250 \text{ N/mm}^2] =$ **0.504**

Average depth to reinforcement; $d =$ **1713** mm

Maximum punching shear resistance (cl.6.4.5(3)); $v_{Rd.max} = 0.5 \times v \times f_{cd} =$ **5.712** N/mm$^2$

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) =$ **1.342**

Longitudinal reinforcement ratio (cl.6.4.4(1)); $\rho_{lx} = A_{sx.bot.prov} / (L_y \times d) =$ **0.002**

$\rho_{ly} = A_{sy.bot.prov} / (L_x \times d) =$ **0.002**

$\rho_l = \min(\sqrt{(\rho_{lx} \times \rho_{ly})}, 0.02) =$ **0.002**

$C_{Rd,c} = 0.18 / \gamma_C =$**0.120**

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} =$ **0.344** N/mm$^2$

Design punching shear resistance (exp.6.47); $v_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) =$ **0.344** N/mm$^2$

Design punching shear resistance at 1d (exp. 6.50); $v_{Rd.c1} = (2 \times d / d) \times v_{Rd.c} =$ **0.688** N/mm$^2$

**Column No.1 - Punching shear perimeter at column face**

Punching shear perimeter; $u_0 =$ **1200** mm

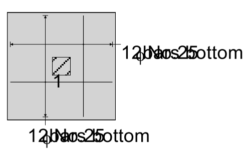
Area within punching shear perimeter; $A_0$ = **0.090** $m^2$

Maximum punching shear force; $V_{Ed.max}$ = **502** kN

Punching shear stress factor (fig 6.21N); $\beta$ = **1.500**

Maximum punching shear stress (exp 6.38); $v_{Ed.max} = \beta \times V_{Ed.max} / (u_0 \times d)$ = **0.366** $N/mm^2$

***PASS - Maximum punching shear resistance exceeds maximum punching shear stress***




;

# EN STAIR DESIGN

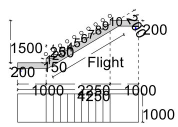
## RC STAIR DESIGN (EN 1992)

**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 | $mm^2$ | 679 | 556(crk) | 0.82 | PASS |
| Bottom long. reinf. -Upper land. | $mm^2$ | 471 | 310 | 0.66 | PASS |
| Bottom long. reinf. -Lower land. | $mm^2$ | 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; $N_{steps}$ = **10**

Going; G = **250** mm

Rise; R = **150** mm

Waist; $h_{waist}$ = **200** mm

Breadth; b = **1000** mm

Length of the tread span; $L_{mid} = (N_{steps} - 1) \times G =$ **2250** mm

Overall height of stairs; $h_{stairs} = N_{steps} \times R =$ **1500** mm

Angle of stairs; $\alpha_{stairs} = \text{atan}(R / G) =$ **30.96** °

**Upper landing - Simple end support**

Length of the upper landing; $L_{up}$ = **1000** mm

Depth of the upper landing; $h_{up}$ = **200** mm

Width of the supporting element; $w_{up}$ = **200** mm

Distance to the centre of the support; $d_{s,up}$ = **100** mm

**Lower landing - Simple end support**

Length of the lower landing; $L_{low}$ = **1000** mm

Depth of the lower landing; $h_{low}$ = **200** mm

Width of the supporting element; $w_{low}$ = **200** mm

Distance to the centre of the support; $d_{s,low}$ = **100** mm

**Effective span – 5.3.2.2**

Overall length; $L_{total} = L_{low} + L_{mid} + L_{up}$ = **4250** mm

Clear distance between the supports faces; $L_n = L_{total} - w_{up} - w_{low}$ = **3850** mm

Effective span variable – cl.5.3.2.2(1); $a_{up} = \min(0.5 \times h_{up}, 0.5 \times w_{up})$ = **100** mm

Effective span variable – cl.5.3.2.2(1); $a_{low} = \min(0.5 \times h_{low}, 0.5 \times w_{low})$ = **100** mm

Length of the effective span – exp.5.8; $L_{span} = L_n + a_{low} + a_{up}$ = **4050** mm

**Concrete details - Table 3.1.**

Concrete strength class; C40/50

Characteristic compressive cylinder strength; $f_{ck}$ = **40** N/mm$^2$

Mean value of compressive cylinder strength $f_{cm} = f_{ck} + 8$ N/mm$^2$ = **48.0** N/mm$^2$

Mean value of axial tensile strength $f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck}/ 1$ N/mm$^2)^{2/3}$ = **3.5** N/mm$^2$

Secant modulus of elasticity of concrete $E_{cm} = 22$ kN/mm$^2 \times (f_{cm}/10$ N/mm$^2)^{0.3} \times$ AAF = **35220.5** N/mm$^2$

Partial factor for concrete – Table 2.1N; $\gamma_C$ = **1.5**

Density of concrete; $\gamma_{conc}$ = **24.50** kN/m$^3$

Compression chord coefficient - cl.6.2.3(3); $\alpha_{cw}$ = **1**

Compressive strength coefficient – cl.3.1.6(1); $\alpha_{cc}$ = **0.85**

Design compr. concrete strength - exp.3.15 $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **22.7** N/mm$^2$

Effective strength factor – exp.3.21; $\eta$ = **1.00**

Effect. compr. zone height factor – exp.3.19; $\lambda$ = **0.80**

Ultimate strain - Table 3.1; $\varepsilon_{cu2}$ = **0.0035**

Shortening strain - Table 3.1; $\varepsilon_{cu3}$ = **0.0035**

$k_1$ = **0.40**

$k_2 = 1.0 \times (0.6 + 0.0014 / \varepsilon_{cu2})$ = **1.00**

$k_3$ = **0.40**

$k_4 = 1.0 \times (0.6 + 0.0014 / \varepsilon_{cu2})$ = **1.00**

Maximum aggregate size; $h_{agg}$ = **20** mm

**Reinforcing steel details**

Characteristic yield strength of reinforcement; $f_{yk}$ = **500** N/mm$^2$

Partial factor for reinforcing steel - Table 2.1N; $\gamma_S$ = **1.15**

Design yield strength of reinforcement; $f_{yd} = f_{yk} / \gamma_S$= **435** N/mm$^2$

**Loading details**

Self weight slab; $g_{self,slab} = h_{waist} / \cos(\alpha_{stairs}) \times (\gamma_{conc}) \times b$ = **5.7** kN/m

Self weight steps; $g_{self,steps} = R / 2 \times (\gamma_{conc}) \times b = \mathbf{1.8}$ kN/m

Average self weight; $g_{self,aver} = g_{self,slab} + g_{self,steps} = \mathbf{7.6}$ kN/m

Loading from finishes; $g_{fin} = \mathbf{1.2}$ kN/m$^2$

Imposed variable load; $q_k = \mathbf{3.0}$ kN/m$^2$

Permanent action factor; $\gamma_G = \mathbf{1.35}$

Imposed action factor; $\gamma_Q = \mathbf{1.50}$

Quasi-permanent value of variable action $\psi_2 = \mathbf{0.30}$

Design load; $F_{Ed} = \gamma_G \times (g_{self,aver} + g_{fin} \times b) + \gamma_Q \times q_k \times b = \mathbf{16.3}$ kN /m

Quasi-permanent load; $F_{QP} = 1.0 \times (g_{self,aver} + g_{fin} \times b) + \psi_2 \times q_k \times b = \mathbf{9.7}$ kN /m

**Midspan design**

$\alpha = 0.125$

Design moment $M = \text{abs}(\alpha \times F_{Ed} \times L_{span}^2) = \mathbf{33.5}$ kNm

Nominal cover to reinforcement $c_{nom} = \mathbf{25}$ mm

Diameter of bar for long. direction $\phi_l = \mathbf{12}$ mm

Depth of reinforcement $d = h - c_{nom} - \phi_l / 2 = \mathbf{169}$ mm

Redistribution ratio $\delta = \mathbf{1.00}$

K coefficient $K = M / (b \times d^2 \times f_{ck}) = \mathbf{0.029}$

K' coefficient $K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) = \mathbf{0.207}$

***K < K' -Compression reinforcement is not required***

Lever arm $z = \min(0.5 \times d \times (1 + (1 - 2 \times K_z / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}), 0.95 \times d)$

$z = \mathbf{161}$ mm

Tension area required in longitudinal direction $A_{s,req} = M / (f_{yd} \times z) = \mathbf{479}$ mm$^2$

Minimum area of reinforcement – exp.9.1.N $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk} \times b \times d, 0.0013 \times b \times d) = \mathbf{308}$ mm$^2$

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{s,max} = 0.04 \times b \times h = \mathbf{8000}$ mm$^2$

**Tension reinforcement check**

Diameter of bars for longitudinal direction $\phi_l = \mathbf{12}$ mm

Number of bars in longitudinal direction $N = \mathbf{6}$

Diameter of bars for transverse direction $\phi_t = \mathbf{12}$ mm

Bar spacing in transverse direction $s = \mathbf{250}$ mm

Tension area provided in longitudinal direction $A_{s,prov} = N \times \pi \times \phi_l^2 /4 = \mathbf{679}$ mm$^2$

***PASS - Tension reinforcement area is greater than area required***

Tension secondary area required in transverse direction – cl.9.3.1.1(2)

$A_{s,req,t} = 0.2 \times \max(A_{s,prov}, A_{s,req}) = \mathbf{136}$ mm$^2$

Tension area provided in transverse direction $A_{s,prov,t} = \pi \times \phi_t^2 /4 \times (b / s) = \mathbf{452}$ mm$^2$

***PASS - Tension reinforcement in transverse direction is greater than area required***

**Minimum bar spacing - Section 8.2**

Bar spacing (clear distance) in long. direction; $s_{bar,l} = (b - (2 \times c_{nom} + N \times \phi_l)) / (N - 1) = \mathbf{176}$ mm

$k_{spc1} = \mathbf{1}$

$k_{spc2} = \mathbf{5}$ mm

Minimum allowable bar spacing – cl.8.2(2); $s_{min,l} = \max(k_{spc1} \times \phi_l, h_{agg} + k_{spc2}, 20 \text{ mm}) = \mathbf{25}$ mm

***PASS - Actual bar spacing exceeds minimum allowable***

Bar spacing (clear distance) in transv. direction; $s_{bar,t} = s - \phi_t =$ **238** mm

Minimum allowable bar spacing – cl.8.2(2); $s_{min,t} = \max(k_{spc1} \times \phi_t, h_{agg} + k_{spc2}, 20 \text{ mm}) =$ **25** mm

***PASS - Actual bar spacing exceeds minimum allowable***

**Crack control - Section 7.3**

Maximum crack width $w_k =$ **0.3** mm

Design value modulus of elasticity reinf – 3.2.7(4) $E_s =$ **200000** N/mm$^2$

Mean value of concrete tensile strength $f_{ct,eff} = f_{ctm} =$ **3.5** N/mm$^2$

Stress distribution coefficient $k_c =$ **0.4**

Non-uniform self-equilibrating stress coefficient $k = \min(\max(1 + (300 \text{ mm} - \min(h, b)) \times 0.35 / 500 \text{ mm}, 0.65), 1) =$ **1.00**

Actual tension bar spacing $s_{bar} = (b - (2 \times (c_{nom} + \phi / 2))) / (N - 1) =$ **188** mm

Maximum stress permitted - Table 7.3N $\sigma_s =$ **250** N/mm$^2$

Steel to concrete modulus of elast. ratio $\alpha_{cr} = E_s / E_{cm} =$ **5.68**

Distance of the Elastic NA from tension face $y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1))$

$y =$ **99** mm

Area of concrete in the tensile zone $A_{ct} = b \times y =$ **98922** mm$^2$

Minimum area of reinforcement required - exp.7.1 $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s =$ **556** mm$^2$

***PASS - Area of tension reinforcement provided exceeds minimum required for crack control***

Quasi-permanent moment $M_{QP} = \text{abs}(\alpha \times F_{QP} \times L_{span}^2) =$ **19.8**kNm

Permanent load ratio $R_{PL} = M_{QP} / M =$ **0.59**

Service stress in reinforcement $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL} =$ **182** N/mm$^2$

Maximum bar spacing - Tables 7.3N $s_{bar,max} =$ **272.9** mm

***PASS - Maximum bar spacing exceeds actual bar spacing for crack control***

**Deflection control - Section 7.4**

Reference reinforcement ratio $\rho_0 = (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / 1000 =$ **0.00632**

Required tension reinforcement ratio $\rho = A_{s,req} / (b \times d_{mid}) =$ **0.00284**

Required compression reinforcement ratio $\rho' = A'_{s,req} / (b \times d_{mid}) =$ **0.00000**

Structural system factor - Table 7.4N $K_b =$ **1.00**

Basic allow. span to depth ratio -**exp.7.16(a)**

$$std_{basic} = K_b \times [11 + 1.5 \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} \times \rho_0 / \rho + 3.2 \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} \times (\rho_0 / \rho - 1)^{3/2}] = \mathbf{59.783}$$

Reinforcement factor - exp.7.17 $K_s = \min(A_{s,prov} / A_{s,req} \times 500 \text{ N/mm}^2 / f_{yk}, 1.5) =$ **1.416**

Long span supp. brittle partition factor-cl.7.4.2(2) $F1 =$ **1.00**

Allowable span to depth ratio $std_{allow} = \min(std_{basic} \times K_s \times F1, 40 \times K_b) =$ **40.000**

Actual span to depth ratio $std_{actual} = L_{span} / d_{mid} =$ **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 $c_{nom} =$ **25** mm

Diameter of bar for long. direction $\phi_l =$**10** mm

Depth of reinforcement $d = h - c_{nom} - \phi_l / 2 =$ **170** mm

Calculated reinforcement at midspan cl. 9.3.1.2(1) $A_{s,span} = M_{mid} / (f_{yd} \times z_{mid}) =$ **479** mm$^2$

Minimum area of reinforcement – exp.9.1.N $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk} \times b \times d, 0.0013 \times b \times d, 0.5 \times A_{s,span})$

$A_{s,min} =$ **310** mm$^2$

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{s,max} = 0.04 \times b \times h =$ **8000** mm$^2$

**Tension reinforcement check**

Diameter of bars for longitudinal direction $\phi_l =$ **10** mm

Number of bars in longitudinal direction N = **6**

Diameter of bars for transverse direction $\phi_t =$ **10** mm

Bar spacing in transverse direction s = **300** mm

Tension area provided in longitudinal direction $A_{s,prov} = N \times \pi \times \phi_l^2 /4 =$ **471** mm$^2$

***PASS - Tension reinforcement area is greater than area required***

Tension secondary area required in transverse direction – cl.9.3.1.1(2)

$A_{s,req,t} = 0.2 \times \max(A_{s,prov}, A_{s,req}) =$ **94** mm$^2$

Tension area provided in transverse direction $A_{s,prov,t} = \pi \times \phi_t^2 /4 \times (b / s) =$ **262** mm$^2$

***PASS - Tension reinforcement in transverse direction is greater than area required***

**Minimum bar spacing - Section 8.2**

Bar spacing (clear distance) in long. direction; $s_{bar,l} = (b - (2 \times c_{nom} + N \times \phi_l)) / (N - 1) =$ **178** mm

$k_{spc1} =$ **1**

$k_{spc2} =$ **5** mm

Minimum allowable bar spacing – cl.8.2(2); $s_{min,l} = \max(k_{spc1} \times \phi_l, h_{agg} + k_{spc2}, 20 \text{ mm}) =$ **25** mm

***PASS - Actual bar spacing exceeds minimum allowable***

Bar spacing (clear distance) in transv. direction; $s_{bar,t} = s - \phi_t =$ **290** mm

Minimum allowable bar spacing – cl.8.2(2); $s_{min,t} = \max(k_{spc1} \times \phi_t, h_{agg} + k_{spc2}, 20 \text{ mm}) =$ **25** mm

***PASS - Actual bar spacing exceeds minimum allowable***

**Shear capacity check - Section 6.2**

Shear coefficient; $\beta = 0.500$

Design shear force; $V = \beta \times F_{Ed} \times L_{span} =$ **33.0** kN

Effective depth; d = **170** mm

Shear stress; $v = V / (b \times d) =$ **0.19** N/mm$^2$

Design shear resist. coefficient. – cl.6.2.2(1) $C_{Rd,c} = 0.18 / \gamma_C =$ **0.120**

Tension reinforcement provided; $A_{sl} = N \times \pi \times \phi_l^2 /4=$ **471** mm$^2$

Design shear resist. size factor. – cl.6.2.2(1) $K_{sh} = \min(1 + (200 \text{ mm} / d)^{1/2}, 2) =$ **2.00**

Design shear resist. factor. – cl.6.2.2(1) $\rho_1 = A_{sl} / (b \times d) =$ **0.00277**

Design shear resist. factor. – exp.6.3.N $v_{min} = (0.035 \text{ (N)}^{1/2} / \text{mm}) \times K_{sh}{}^{3/2} \times f_{ck}{}^{1/2}=$ **0.63** N/mm$^2$

Minimum design shear resistance – exp.6.2(b) $V_{Rd,cmin} = (v_{min}) \times b \times d =$ **106.4** kN

Design shear resist. without reinf. – exp.6.2(a) $V_{Rd,c1} = (C_{Rd,c} \times K_{sh} \times (100 \text{ N}^2 / \text{mm}^4 \times \rho_1 \times f_{ck})^{1/3}) \times b \times d$

$V_{Rd,c1} =$ **91.0** kN

$V_{Rd,c} = \max(V_{Rd,c1}, V_{Rd,cmin}) =$ **106.4** kN

***PASS - Design force is less than maximum allowable- No shear reinforcement is required;***

**<u>Lower landing support</u>**

Design moment at support M = **0.0** kNm

Nominal cover to reinforcement $c_{nom} =$ **25** mm

Diameter of bar for long. direction $\phi_l =$**10** mm

Depth of reinforcement $d = h - c_{nom} - \phi_l / 2 =$ **170** mm

Calculated reinforcement at midspan cl. 9.3.1.2(1) $A_{s,span} = M_{mid} / (f_{yd} \times z_{mid}) =$ **479** mm$^2$

Minimum area of reinforcement – exp.9.1.N $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk} \times b \times d, 0.0013 \times b \times d, 0.5 \times A_{s,span})$

$A_{s,min}$ = **310** mm$^2$

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{s,max} = 0.04 \times b \times h$ = **8000** mm$^2$

**Tension reinforcement check**

Diameter of bars for longitudinal direction $\phi_l$ = **10** mm

Number of bars in longitudinal direction N = **6**

Diameter of bars for transverse direction $\phi_t$ = **10** mm

Bar spacing in transverse direction s = **300** mm

Tension area provided in longitudinal direction $A_{s,prov} = N \times \pi \times \phi_l^2 /4$ = **471** mm$^2$

***PASS - Tension reinforcement area is greater than area required***

Tension secondary area required in transverse direction – cl.9.3.1.1(2)

$A_{s,req,t} = 0.2 \times \max(A_{s,prov}, A_{s,req})$ = **94** mm$^2$

Tension area provided in transverse direction $A_{s,prov,t} = \pi \times \phi_t^2 /4 \times (b / s)$ = **262** mm$^2$

***PASS - Tension reinforcement in transverse direction is greater than area required***

**Minimum bar spacing - Section 8.2**

Bar spacing (clear distance) in long. direction; $s_{bar,l} = (b - (2 \times c_{nom} + N \times \phi_l)) / (N - 1)$ = **178** mm

$k_{spc1}$ = **1**

$k_{spc2}$ = **5** mm

Minimum allowable bar spacing – cl.8.2(2); $s_{min,l} = \max(k_{spc1} \times \phi_l, h_{agg} + k_{spc2}, 20\text{ mm})$ = **25** mm

***PASS - Actual bar spacing exceeds minimum allowable***

Bar spacing (clear distance) in transv. direction; $s_{bar,t} = s - \phi_t$ = **290** mm

Minimum allowable bar spacing – cl.8.2(2); $s_{min,t} = \max(k_{spc1} \times \phi_t, h_{agg} + k_{spc2}, 20\text{ mm})$ = **25** mm

***PASS - Actual bar spacing exceeds minimum allowable***

**Shear capacity check - Section 6.2**

Shear coefficient; $\beta = 0.500$

Design shear force; $V = \beta \times F_{Ed} \times L_{span}$ = **33.0** kN

Effective depth; d = **170** mm

Shear stress; $v = V / (b \times d)$ = **0.19** N/mm$^2$

Design shear resist. coefficient. – cl.6.2.2(1) $C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**

Tension reinforcement provided; $A_{sl} = N \times \pi \times \phi_l^2 /4$= **471** mm$^2$

Design shear resist. size factor. – cl.6.2.2(1) $K_{sh} = \min(1 + (200\text{ mm} / d)^{1/2}, 2)$ = **2.00**

Design shear resist. factor. – cl.6.2.2(1) $\rho_1 = A_{sl} / (b \times d)$ = **0.00277**

Design shear resist. factor. – exp.6.3.N $v_{min} = (0.035\ (N)^{1/2} / mm) \times K_{sh}{}^{3/2} \times f_{ck}{}^{1/2}$= **0.63** N/mm$^2$

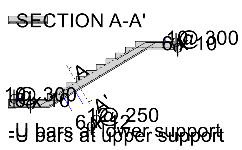
Minimum design shear resistance – exp.6.2(b) $V_{Rd,cmin} = (v_{min}) \times b \times d$ = **106.4** kN

Design shear resist. without reinf. – exp.6.2(a) $V_{Rd,c1} = (C_{Rd,c} \times K_{sh} \times (100\ N^2 / mm^4 \times \rho_1 \times f_{ck})^{1/3}) \times b \times d$

$V_{Rd,c1}$ = **91.0** kN

$V_{Rd,c} = \max(V_{Rd,c1}, V_{Rd,cmin})$ = **106.4** kN

***PASS - Design force is less than maximum allowable- No shear reinforcement is required;***



SECTION A-A'
A
A'
12@250
=U bars at lower support
=U bars at upper support


;
;

Kelly Mweu Changilwa
+254703206460
mweukelly@gmail.com



# EN WALL DESIGN

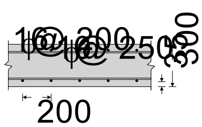
## RC WALL DESIGN (EN 1992)

**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; $h = \mathbf{300}$ mm

Length; $b = \mathbf{1000}$ mm/m

Clear height between restraints; $l = \mathbf{3000}$ mm

Stability about minor axis; **Unbraced**

**Concrete details**

Concrete strength class; **C40/50**

Partial safety factor for concrete (2.4.2.4(1)); $\gamma_C = \mathbf{1.50}$

Coefficient $\alpha_{cc}$ (3.1.6(1)); $\alpha_{cc} = \mathbf{0.85}$

Maximum aggregate size; $d_g = \mathbf{20}$ mm

**Reinforcement details**

Reinforcement in outer layer; **Vertical**

Nominal cover to outer layer; $c_{nom} = \mathbf{35}$ mm

Vertical bar diameter; $\phi_v = \mathbf{16}$ mm

Spacing of vertical reinforcement; $s_v = \mathbf{200}$ mm

Area of vertical reinforcement (per face); $A_{sv} = \mathbf{1005}$ mm$^2$/m

Horizontal bar diameter; $\phi_h = \mathbf{10}$ mm

Spacing of horizontal reinforcement; $s_h$ = **250** mm

Area of horizontal reinforcement (per face); $A_{sh}$ = **314** mm$^2$/m

Characteristic yield strength; $f_{yk}$ = **500** N/mm$^2$

Partial safety factor for reinft (2.4.2.4(1)); $\gamma_S$ = **1.15**

Modulus of elasticity of reinft (3.2.7(4)); $E_s$ = **200.0** kN/mm$^2$

**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; $\mu_{fi}$ = **0.70**

**Axial load and bending moments from frame analysis**

Design axial load; $N_{Ed}$ = **500.0** kN/m

Moment about minor axis at top; $M_{top}$ = **20.0** kNm/m

Moment about minor axis at bottom; $M_{btm}$ = **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.; $N_{Ed\_SLS}$ = **325.0** kN/m

Moment at top due to quasi-permanent SLS.; $M_{top\_SLS}$ = **13.0** kNm/m

Moment at btm due to quasi-permanent SLS.; $M_{btm\_SLS}$ = **19.5** kNm/m

Duration of applied loading; **Long term**

Maximum allowable crack width; $w_{k\_max}$ = **1.0** mm

## Calculated wall properties

**Concrete properties**

Area of concrete; $A_c = h \times b$ = **300000** mm$^2$/m

Characteristic compression cylinder strength; $f_{ck}$ = **40** N/mm$^2$

Design compressive strength (3.1.6(1)); $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **22.7** N/mm$^2$

Mean value of cylinder strength (Table 3.1); $f_{cm} = f_{ck}$ + 8 MPa = **48.0** N/mm$^2$

Mean value of tensile strength; $f_{ctm}$ = **3.51** N/mm$^2$

Secant modulus of elasticity (Table 3.1); $E_{cm}$ = 22000 MPa $\times$ ($f_{cm}$ / 10 MPa)$^{0.3}$ = **35.2** kN/mm$^2$

**Rectangular stress block factors**

Depth factor (3.1.7(3)); $\lambda_{sb}$ = 0.8

Stress factor (3.1.7(3)); $\eta$ = 1.0

**Strain limits**

Compression strain limit (Table 3.1); $\varepsilon_{cu3}$ = **0.00350**

Pure compression strain limit (Table 3.1); $\varepsilon_{c3}$ = **0.00175**

**Design yield strength of reinforcement**

Design yield strength (3.2.7(2)); $f_{yd} = f_{yk} / \gamma_S$ = **434.8** MPa

**Check nominal cover for fire and bond requirements**

Min. cover reqd for bond (4.4.1.2(3)); $C_{min,b} = \max(\phi_v, \phi_h - \phi_v)$ = **16** mm

Min axis distance for fire (EN1992-1-2 T 5.4); $a_{fi}$ = **10** mm

Allowance for deviations from min cover (4.4.1.3); $\Delta c_{dev}$ = **5** mm

Kelly Mweu Changilwa
+254703206460
mweukelly@gmail.com


Min allowable nominal cover; $c_{nom\_min} = \max(a_{fi} - \phi_v / 2, c_{min,b} + \Delta c_{dev})$ = ;**21**; mm

***PASS - the nominal cover is greater than the minimum required***

**Effective depth of vertical bars**

Effective depth; $d = h - c_{nom} - \phi_v / 2$ = **257** mm

Depth to compression face bars; $d' = c_{nom} + \phi_v / 2$ = **43** mm

**Wall effective length**

Wall effective length; $l_0 = f \times l$ = **2100** mm

**Column slenderness**

Radius of gyration about minor axis; $i = h / \sqrt{(12)}$ = **8.7** cm

Minor axis slenderness ratio (5.8.3.2(1)); $\lambda = l_0 / i$ = **24.2**

**<u>Design bending moments</u>**

**Frame analysis moments combined with moments due to imperfections (cl. 5.2 & 6.1(4))**

Ecc. due to geometric imperfections; $e_i = l_0 /400$ = **5.2** mm

Minimum end moment about minor axis; $M_{01} = \min(\text{abs}(M_{top}), \text{abs}(M_{btm})) + e_i \times N_{Ed}$ = **22.6** kNm/m

Maximum end moment about minor axis; $M_{02} = \max(\text{abs}(M_{top}), \text{abs}(M_{btm})) + e_i \times N_{Ed}$ = **32.6** kNm/m

**Slenderness limit for buckling about minor axis (cl. 5.8.3.1)**

Factor A; A = **0.7**

Mechanical reinforcement ratio; $\omega = 2 \times A_{sv} \times f_{yd} / (A_c \times f_{cd})$ = **0.129**

Factor B; $B = \sqrt{(1 + 2 \times \omega)}$ = **1.121**

Moment ratio; $r_m$ = ;**1.000**

Factor C; $C = 1.7 - r_m$ = **0.700**

Relative normal force; $n = N_{Ed} / (A_c \times f_{cd})$ = **0.074**

Slenderness limit; $\lambda_{lim} = 20 \times A \times B \times C / \sqrt{(n)}$ = **40.5**

***$\lambda<\lambda_{lim}$ - Second order effects may be ignored***

**Design bending moment**

Design moment about minor axis; $M_{Ed} = \max(M_{02}, N_{Ed} \times \max(h/30, 20 \text{ mm}))$ = **32.6** kNm/m

**<u>Moment capacity about minor axis with axial load $N_{Ed}$</u>**

**Moment of resistance of concrete**

By iteration:-

Position of neutral axis; z = **47.7** mm

Concrete compression force (3.1.7(3)); $F_c = \eta \times f_{cd} \times \min(\max(\lambda_{sb} \times z, 0 \text{ mm}), h) \times b$ = **865.4** kN/m

Moment of resistance; $M_{Rdc} = F_c \times [h / 2 - (\min(\lambda_{sb} \times z, h)) / 2]$ = **113.3** kNm/m

**Moment of resistance of reinforcement**

Strain in tension face bars; $\varepsilon = \varepsilon_{cu3} \times (1 - d / z)$ = **-0.01535**

Stress in tension face bars; $\sigma = \text{if}(\varepsilon< 0, \max(-1\times f_{yd}, E_s \times \varepsilon), \min(f_{yd}, E_s \times \varepsilon))$ = **-434.8** N/mm$^2$

Force in tension face bars; $F_s = \text{if}(d > \lambda_{sb} \times z, A_{sv} \times \sigma, A_{sv} \times (\sigma - \eta \times f_{cd}))$ = **-437.1** kN/m

Strain in compression face bars; $\varepsilon' = \varepsilon_{cu3} \times (1 - d' / z)$ = **0.00035**

Stress in compression face bars; $\sigma' = \text{if}(\varepsilon'< 0, \max(-1\times f_{yd}, E_s \times \varepsilon'), \min(f_{yd}, E_s \times \varepsilon'))$ = **69.3** N/mm$^2$

Force in compression face bars; $F_s' = \text{if}(d' > \lambda_{sb} \times z, A_{sv} \times \sigma', A_{sv} \times (\sigma' - \eta \times f_{cd}))$ = **69.7** kN/m

Resultant concrete/steel force; $F = F_c + F_s + F_s'$ = **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; $M_{Rds} = F_s \times (d - h / 2) =$ **-46.8** kNm/m

Moment of resistance of compression face bars; $M_{Rds}' = F_s' \times (h / 2 - d') =$ **7.5** kNm/m

**Combined moment of resistance**

Moment of resistance about minor axis; $M_{Rd} = M_{Rdc} + M_{Rds}' - M_{Rds} =$ **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; $M_{01\_SLS}=\min(\text{abs}(M_{top\_SLS}),\text{abs}(M_{btm\_SLS}))+e_i\times N_{Ed\_SLS} =$ **14.7** kNm/m

Max 1st order moment about minor axis; $M_{02\_SLS}=\max(\text{abs}(M_{top\_SLS}),\text{abs}(M_{btm\_SLS}))+e_i\times N_{Ed\_SLS} =$ **21.2** kNm/m

Moment ratio; $r_{m\_SLS} =$ ;**1.000**

Factor C; $C_{SLS} = 1.7 - r_{m\_SLS} =$ **0.700**

Relative normal force; $n_{SLS} = N_{Ed\_SLS} / (A_c \times f_{cd}) =$ **0.048**

Slenderness limit; $\lambda_{lim\_SLS} = 20 \times A \times B \times C_{SLS} / \sqrt{(n_{SLS})} =$ **50.3**

***$\lambda<\lambda_{lim\_SLS}$ - Second order effects may be ignored***

**Design bending moment (cl. 7.3.4)**

Design moment about minor axis; $M_{Ed\_SLS} = M_{02\_SLS} =$ **21.2** kNm/m

Cover to tension reinforcement; $c = h - d - \phi_v / 2 =$ **35.0** mm

Ratio of steel to concrete modulii; $\alpha_e = E_s / E_{cm} =$ **5.7**

Area of reinft in concrete units; $A_{s,eff} = 2 \times \alpha_e \times A_{sv} =$ **11417** mm²/m

Combined area of steel/conc in conc units; $A_{eff} = b \times h + A_{s,eff} =$ **311417** mm²/m

Reinforcement ratio per face; $\rho = A_{sv} /(b \times d) =$ **0.004**

Neutral axis depth with pure bending; $x_b = d \times [-2\times\alpha_e\times\rho + \sqrt{(4\times\alpha_e^2\times\rho^2 + 2\times\alpha_e\times\rho\times(1+d'/d))}] =$ **48.2** mm

Second moment of area of cracked section; $I_c = b\times x_b^3/3 + \alpha_e\times\rho\times b\times d\times[(x_b-d')^2 + (d-x_b)^2] =$ **286364393** mm⁴/m

Strain in tension face steel due to bending; $\varepsilon_{sb} = M_{Ed\_SLS} \times (x_b - d) / (E_{cm} \times I_c) =$ **-0.00044**

Strain in comp face steel due to bending; $\varepsilon_{sb}' = M_{Ed\_SLS} \times (x_b - d') / (E_{cm} \times I_c) =$ **0.00001**

Strain due to axial load; $\varepsilon_{axial} = N_{Ed\_SLS} / (A_{eff} \times E_{cm}) =$ **0.00003**

Resultant strain in tension face steel; $\varepsilon_s = \varepsilon_{sb} + \varepsilon_{axial} =$ **-0.00041**

Resultant strain in comp face steel; $\varepsilon_s' = \varepsilon_{sb}' + \varepsilon_{axial} =$ **0.00004**

Stress in tension steel; $\sigma_s = \min(f_{yd}, \text{abs}(E_s \times \varepsilon_s)) =$ **81.9** MPa

Depth to neutral axis; $x = [(\varepsilon_s' \times d) - (\varepsilon_s \times d')] / (\varepsilon_s' - \varepsilon_s) =$ **62.3** mm

Effective depth of concrete in tension; $h_{c,ef} = \min(2.5\times(h-d), (h-x)/3, h/2) =$ **79.2** mm

Effective area of concrete in tension; $A_{c,eff} = h_{c,ef} \times b =$ **79232** mm²/m

Load duration factor; $k_t =$ **0.4**

Reinforcement ratio; $\rho_{p,eff} = A_{sv} / A_{c,eff} =$ **0.013**

Mean value of conc tensile strength; $f_{ct,eff} = f_{ctm} =$ **3.51** MPa

Difference between reinft and concrete strains; $\varepsilon_{diff} = \max([\sigma_s-k_t\times f_{ct,eff}\times(1+\alpha_e\times\rho_{p,eff})/\rho_{p,eff}]/E_s, 0.6\times\sigma_s/E_s) =$ **0.00025**

Greater tensile strain; $\varepsilon_1 = \varepsilon_s \times (h - x) / (d - x) =$ **-0.00050**

Lesser tensile strain; $\varepsilon_2 = \min(0, \varepsilon_s' \times x / (x - d')) =$ **0.00000**

Factor $k_1$; $k_{1cs} =$ **0.8**

Factor $k_2$; $k_{2cs} = (\varepsilon_1 + \varepsilon_2) / (2 \times \varepsilon_1) =$ **0.500**

Factor $k_3$; $k_{3cs}$ = 3.40= **3.4**

Factor $k_4$; $k_{4cs}$ = **0.425**

Maximum crack spacing; $s_{r,max} = k_{3cs} \times c + k_{1cs} \times k_{2cs} \times k_{4cs} \times \phi_v / \rho_{p,eff}$ = **333.4** mm

Crack width; $w_k = s_{r,max} \times \varepsilon_{diff}$ = **0.082** mm

Allowable crack width; $w_{k\_max}$ = **1.0** mm

***PASS - The maximum crack width is less than the maximum allowable***

;