

1126-01
Kirkby BESS
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Pipe Cover Slab
Structural Design Calculations

April 2026

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Kirkby BESS
Coopers Lane, Liverpool. L33 7TT

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Structural Design Calculations

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Barnsley Marshall Limited

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4&5 Prior Wharf, Harris Business Park, Stoke Prior, Worcester,
B60 4FG
Tel 01905 330550

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Date

Contents Amendment Record

This report has been issued and amended as follows:

Revision		Description	Date	Signed
1	P01	Preliminary issue	29/04/26	NJ

Calculation Sheet

Project: 1126-01 Kirkby BESS

Title: 900mm dia pipe cover slab

Sht. No: 1

Rev:

By: NJ

Date: 17/04/26

Chkd: JP

Date: 17/04/26



Calculations

Output

900mm dia. Pipe Cover Slab

- The 900mm diameter water main pipe is located at 2m deep (to its crown).
- The heaviest vehicle crossing it will be a articulated low loader lorry with a driving axial load of 11.5T and a non-driving axial load of 10T. Proposal shown below:

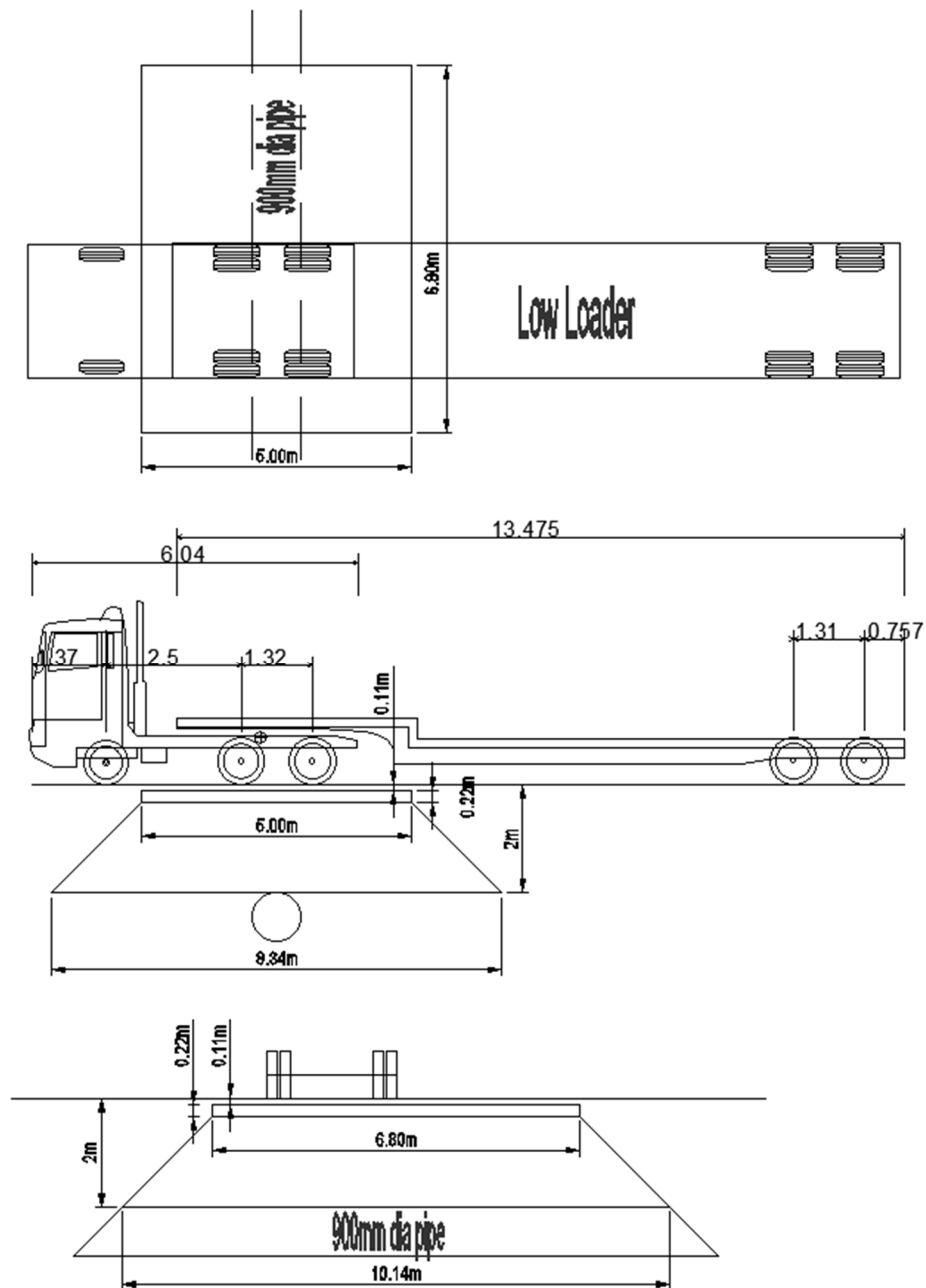


Figure 1

Calculation Sheet

Project: 1126-01 Kirkby BESS

Sht. No: 2

Rev:

Title: 900mm dia pipe cover slab

By: NJ

Date: 17/04/26

Chkd: JP

Date: 17/04/26



Calculations

Output

- It is proposed to construct a 5m x 6.8m, 220mm thick reinforced concrete cover slab over the pipe to spread the load from any vehicles.
- It is assumed that the ground over the pipe and the road construction will allow any vehicle loads to spread at 45° in all directions.
- From TEDDS analysis (on following pages) the maximum pressure at the base of the slab is 21.1kN/m², therefore conservatively if we take this to be the load under the area of the slab then:

$$= 21.1 \times 5 \times 6.8 = 717.4\text{kN}$$

Assuming 45° spread area at crown of pipe:

$$= 8.34 \times 10.14 = 84.6\text{m}^2$$

$$= 717.4 / 84.6 = 8.48\text{kN/m}^2$$

- Therefore (conservatively) the imposed load at the crown of the pipe from the axial load on the cover slab is 8.48kN/m².
- The maximum allowable load at the crown of the pipe is understood to be 8.5kN/m² and therefore the imposed load should be acceptable. It is assumed that the maximum allowable load does not need to include for the existing soil overburden pressure.
- If we use the less conservative average base pressure from the TEDDS analysis then:

$$= (21.1 + 8.3 + 7.7 + 20.5) / 4 = 14.4\text{kN/m}^2$$

$$= 14.4 \times 5 \times 6.8 = 489.6\text{kN}$$

Assuming 45° spread area at crown of pipe:

$$= 8.34 \times 10.14 = 84.6\text{m}^2$$

$$= 489.6 / 84.6 = 5.78\text{kN/m}^2$$

- Therefore, using the average base pressure, the imposed load at the crown of the pipe from the axial load on the cover slab is 5.78kN/m², therefore as this is less than the allowable 8.5kN/m² it should be acceptable.
- For slab design refer to TEDDS calculation design on following pages.
- Therefore, use a 220mm deep PAV2 slab, reinforced with one layer of A393 mesh in the bottom face (50mm cover).

PIPE COVER SLAB 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 3.3.07

Summary table

Description	Unit	Allowable	Actual	Utilisation	Result
Base pressure	kN/m ²	25	21.1	0.842	Pass
Description	Unit	Provided	Required	Utilisation	Result
Reinforcement x-direction	mm ²	2670	1614	0.604	Pass
Reinforcement y-direction	mm ²	1963	1227	0.625	Pass
Description	Unit	Allowable	Actual	Utilisation	Result
Shear x-axis	kN	552	94	0.170	Pass
Shear y-axis	kN	406	70	0.174	Pass
Punching shear	N/mm ²	0.524	0.195	0.373	Pass

Cover slab details

Length of slab;

$L_x = 5000$ mm

Width of slab;

$L_y = 6800$ mm

Slab area;

$A = L_x \times L_y = 34.000$ m²

Depth of slab;

$h = 220$ mm

Depth of soil over foundation;

$h_{soil} = 0$ mm

Level of water;

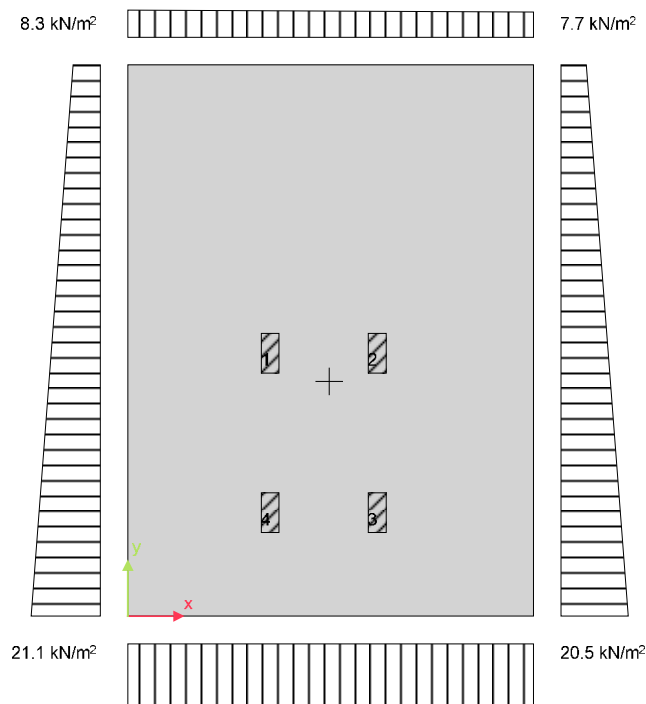
$h_{water} = 0$ mm

Density of water;

$\gamma_{water} = 9.8$ kN/m³

Density of concrete;

$\gamma_{conc} = 25.0$ kN/m³



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Wheel no.1 details

Length; $l_{x1} = 220$ mm
Width; $l_{y1} = 490$ mm
position in x-direction; $x_1 = 1754$ mm
position in y-direction; $y_1 = 3244$ mm

Wheel no.2 details

Length; $l_{x2} = 220$ mm
Width; $l_{y2} = 490$ mm
position in x-direction; $x_2 = 3074$ mm
position in y-direction; $y_2 = 3244$ mm

Wheel no.3 details

Length; $l_{x3} = 220$ mm
Width; $l_{y3} = 490$ mm
position in x-direction; $x_3 = 3074$ mm
position in y-direction; $y_3 = 1276$ mm

Wheel no.4 details

Length; $l_{x4} = 220$ mm
Width; $l_{y4} = 490$ mm
position in x-direction; $x_4 = 1754$ mm
position in y-direction; $y_4 = 1276$ mm

Soil properties

Density of soil; $\gamma_{\text{soil}} = 18.0$ kN/m³
Characteristic cohesion; $c'_k = 0$ kN/m²
Characteristic effective shear resistance angle; $\phi'_k = 30$ deg
Characteristic friction angle; $\delta_k = 25$ deg

Loads

Permanent surcharge load; $F_{\text{Gsur}} = 2.5$ kN/m²
Self weight; $F_{\text{swt}} = h \times \gamma_{\text{conc}} = 5.5$ kN/m²

Wheel no.1 loads

Axial load; $F_{\text{Gz1}} = 50.0$ kN

Wheel no.2 loads

Axial load; $F_{\text{Gz2}} = 58.0$ kN

Wheel no.3 loads

Axial load; $F_{\text{Gz3}} = 58.0$ kN

Wheel no.4 loads

Axial load; $F_{\text{Gz4}} = 50.0$ kN

Bearing resistance (Section 6.5.2)

Forces on slab

Force in z-direction; $F_{\text{dz}} = A \times (F_{\text{swt}} + F_{\text{Gsur}}) + F_{\text{Gz1}} + F_{\text{Gz2}} + F_{\text{Gz3}} + F_{\text{Gz4}} = 489.0$ kN

Moments on slab

Moment in x-direction; $M_{\text{dx}} = A \times (F_{\text{swt}} + F_{\text{Gsur}}) \times L_x / 2 + F_{\text{Gz1}} \times x_1 + F_{\text{Gz2}} \times x_2 + F_{\text{Gz3}} \times x_3 + F_{\text{Gz4}} \times x_4 = 1214.5$ kNm

Moment in y-direction; $M_{\text{dy}} = A \times (F_{\text{swt}} + F_{\text{Gsur}}) \times L_y / 2 + F_{\text{Gz1}} \times y_1 + F_{\text{Gz2}} \times y_2 + F_{\text{Gz3}} \times y_3 + F_{\text{Gz4}} \times y_4 = 1416.4$ kNm

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Eccentricity of base reaction

Eccentricity of base reaction in x-direction;

$$e_x = M_{dx} / F_{dz} - L_x / 2 = \mathbf{-16 \text{ mm}}$$

Eccentricity of base reaction in y-direction;

$$e_y = M_{dy} / F_{dz} - L_y / 2 = \mathbf{-504 \text{ mm}}$$

Slab base pressures

$$q_1 = F_{dz} \times (1 - 6 \times e_x / L_x - 6 \times e_y / L_y) / (L_x \times L_y) = \mathbf{21.1 \text{ kN/m}^2}$$

$$q_2 = F_{dz} \times (1 - 6 \times e_x / L_x + 6 \times e_y / L_y) / (L_x \times L_y) = \mathbf{8.3 \text{ kN/m}^2}$$

$$q_3 = F_{dz} \times (1 + 6 \times e_x / L_x - 6 \times e_y / L_y) / (L_x \times L_y) = \mathbf{20.5 \text{ kN/m}^2}$$

$$q_4 = F_{dz} \times (1 + 6 \times e_x / L_x + 6 \times e_y / L_y) / (L_x \times L_y) = \mathbf{7.7 \text{ kN/m}^2}$$

Minimum base pressure;

$$q_{\min} = \min(q_1, q_2, q_3, q_4) = \mathbf{7.7 \text{ kN/m}^2}$$

Maximum base pressure;

$$q_{\max} = \max(q_1, q_2, q_3, q_4) = \mathbf{21.1 \text{ kN/m}^2}$$

Presumed bearing capacity

Presumed bearing capacity;

$$P_{\text{bearing}} = \mathbf{25.0 \text{ kN/m}^2}$$

PASS - Presumed bearing capacity exceeds design base pressure

Design approach 1

Partial factors on actions - Combination1

Partial factor set;

A1

Permanent unfavourable action - Table A.3;

$$\gamma_G = \mathbf{1.35}$$

Permanent favourable action - Table A.3;

$$\gamma_{Gf} = \mathbf{1.00}$$

Variable unfavourable action - Table A.3;

$$\gamma_Q = \mathbf{1.50}$$

Variable favourable action - Table A.3;

$$\gamma_{Qf} = \mathbf{0.00}$$

Partial factors for spread foundations - Combination1

Resistance factor set;

R1

Bearing - Table A.5;

$$\gamma_{R,v} = \mathbf{1.00}$$

Sliding - Table A.5;

$$\gamma_{R,h} = \mathbf{1.00}$$

Forces on slab

Force in z-direction;

$$F_{dz} = \gamma_G \times (A \times (F_{\text{swt}} + F_{\text{Gsur}}) + F_{Gz1} + F_{Gz2} + F_{Gz3} + F_{Gz4}) = \mathbf{660.2 \text{ kN}}$$

Moments on slab

Moment in x-direction;

$$M_{dx} = \gamma_G \times (A \times (F_{\text{swt}} + F_{\text{Gsur}}) \times L_x / 2 + F_{Gz1} \times x_1 + F_{Gz2} \times x_2 + F_{Gz3} \times x_3 + F_{Gz4} \times x_4) = \mathbf{1639.6 \text{ kNm}}$$

Moment in y-direction;

$$M_{dy} = \gamma_G \times (A \times (F_{\text{swt}} + F_{\text{Gsur}}) \times L_y / 2 + F_{Gz1} \times y_1 + F_{Gz2} \times y_2 + F_{Gz3} \times y_3 + F_{Gz4} \times y_4) = \mathbf{1912.2 \text{ kNm}}$$

Eccentricity of base reaction

Eccentricity of base reaction in x-direction;

$$e_x = M_{dx} / F_{dz} - L_x / 2 = \mathbf{-16 \text{ mm}}$$

Eccentricity of base reaction in y-direction;

$$e_y = M_{dy} / F_{dz} - L_y / 2 = \mathbf{-504 \text{ mm}}$$

Effective area of base

Effective length;

$$L'_x = L_x + 2 \times e_x = \mathbf{4967 \text{ mm}}$$

Effective width;

$$L'_y = L_y + 2 \times e_y = \mathbf{5793 \text{ mm}}$$

Effective area;

$$A' = L'_x \times L'_y = \mathbf{28.775 \text{ m}^2}$$

Slab base pressure

Design base pressure;

$$f_{dz} = F_{dz} / A' = \mathbf{22.9 \text{ kN/m}^2}$$

Design approach 1

Partial factors on actions - Combination2

Partial factor set;

A2

Permanent unfavourable action - Table A.3;

$$\gamma_G = \mathbf{1.00}$$

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Permanent favourable action - Table A.3; $\gamma_{Gf} = 1.00$

Variable unfavourable action - Table A.3; $\gamma_{Q} = 1.30$

Variable favourable action - Table A.3; $\gamma_{Qf} = 0.00$

Partial factors for spread foundations - Combination2

Resistance factor set; R1

Bearing - Table A.5; $\gamma_{R,v} = 1.00$

Sliding - Table A.5; $\gamma_{R,h} = 1.00$

Forces on slab

Force in z-direction; $F_{dz} = \gamma_G \times (A \times (F_{swt} + F_{Gsur}) + F_{Gz1} + F_{Gz2} + F_{Gz3} + F_{Gz4}) = 489.0 \text{ kN}$

Moments on slab

Moment in x-direction; $M_{dx} = \gamma_G \times (A \times (F_{swt} + F_{Gsur}) \times L_x / 2 + F_{Gz1} \times x_1 + F_{Gz2} \times x_2 + F_{Gz3} \times x_3 + F_{Gz4} \times x_4) = 1214.5 \text{ kNm}$

Moment in y-direction; $M_{dy} = \gamma_G \times (A \times (F_{swt} + F_{Gsur}) \times L_y / 2 + F_{Gz1} \times y_1 + F_{Gz2} \times y_2 + F_{Gz3} \times y_3 + F_{Gz4} \times y_4) = 1416.4 \text{ kNm}$

Eccentricity of base reaction

Eccentricity of base reaction in x-direction; $e_x = M_{dx} / F_{dz} - L_x / 2 = -16 \text{ mm}$

Eccentricity of base reaction in y-direction; $e_y = M_{dy} / F_{dz} - L_y / 2 = -504 \text{ mm}$

Effective area of base

Effective length; $L'_x = L_x + 2 \times e_x = 4967 \text{ mm}$

Effective width; $L'_y = L_y + 2 \times e_y = 5793 \text{ mm}$

Effective area; $A' = L'_x \times L'_y = 28.775 \text{ m}^2$

Pad base pressure

Design base pressure; $f_{dz} = F_{dz} / A' = 17 \text{ kN/m}^2$

SLAB 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 3.3.07

Concrete details (Table 3.1 - Strength and deformation characteristics for concrete)

Concrete strength class; C28/35

Characteristic compressive cylinder strength; $f_{ck} = 28 \text{ N/mm}^2$

Characteristic compressive cube strength; $f_{ck,cube} = 35 \text{ N/mm}^2$

Mean value of compressive cylinder strength; $f_{cm} = f_{ck} + 8 \text{ N/mm}^2 = 36 \text{ N/mm}^2$

Mean value of axial tensile strength; $f_{ctm} = 0.3 \text{ N/mm}^2 \times (f_{ck} / 1 \text{ N/mm}^2)^{2/3} = 2.8 \text{ N/mm}^2$

5% fractile of axial tensile strength; $f_{ctk,0.05} = 0.7 \times f_{ctm} = 1.9 \text{ N/mm}^2$

Secant modulus of elasticity of concrete; $E_{cm} = 22 \text{ kN/mm}^2 \times [f_{cm} / 10 \text{ N/mm}^2]^{0.3} = 32308 \text{ N/mm}^2$

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); $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = 15.9 \text{ N/mm}^2$

Tens.strength coeff.for plain concrete (cl.12.3.1(1)); $\alpha_{ct,pl} = 0.80$

Des.tens.strength for plain concrete (exp.12.1); $f_{ctd,pl} = \alpha_{ct,pl} \times f_{ctk,0.05} / \gamma_C = 1.0 \text{ N/mm}^2$

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

Ultimate strain - Table 3.1; $\epsilon_{cu2} = 0.0035$

Shortening strain - Table 3.1; $\epsilon_{cu3} = 0.0035$

Effective compression zone height factor; $\lambda = 0.80$

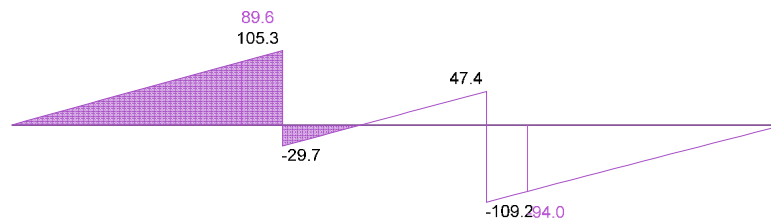
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Effective strength factor; $\eta = 1.00$
 Bending coefficient k_1 ; $K_1 = 0.40$
 Bending coefficient k_2 ; $K_2 = 1.00 \times (0.6 + 0.0014/\epsilon_{cu2}) = 1.00$
 Bending coefficient k_3 ; $K_3 = 0.40$
 Bending coefficient k_4 ; $K_4 = 1.00 \times (0.6 + 0.0014/\epsilon_{cu2}) = 1.00$

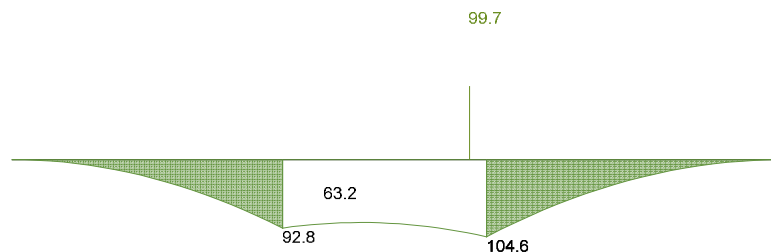
Reinforcement details

Characteristic yield strength of reinforcement; $f_{yk} = 500 \text{ N/mm}^2$
 Modulus of elasticity of reinforcement; $E_s = 210000 \text{ N/mm}^2$
 Partial factor for reinforcing steel (Table 2.1N); $\gamma_s = 1.15$
 Design yield strength of reinforcement; $f_{yd} = f_{yk} / \gamma_s = 435 \text{ N/mm}^2$
 Nominal cover to top of foundation; $c_{nom_t} = 50 \text{ mm}$
 Nominal cover to bottom of foundation; $c_{nom_b} = 50 \text{ mm}$
 Nominal cover to side of foundation; $c_{nom_s} = 50 \text{ mm}$

Shear diagram, x axis (kN)



Moment diagram, x axis (kNm)



Rectangular section in flexure (Section 6.1)

Design bending moment; $M_{Ed.x,max} = 99.7 \text{ kNm}$
 Depth to tension reinforcement; $d = h - c_{nom_b} - \phi_{x,bot} / 2 = 165 \text{ mm}$
 $K = M_{Ed.x,max} / (L_y \times d^2 \times f_{ck}) = 0.019$
 $K' = (2 \times \eta \times \alpha_{cc} / \gamma_c) \times (1 - \lambda \times (\delta - K_1) / (2 \times K_2)) \times (\lambda \times (\delta - K_1) / (2 \times K_2))$
 $K' = 0.207$

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

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_c))^{0.5}, 0.95) \times d = 157 \text{ mm}$
 Depth of neutral axis; $x = 2.5 \times (d - z) = 21 \text{ mm}$
 Area of tension reinforcement required; $A_{sx,bot,req} = M_{Ed.x,max} / (f_{yd} \times z) = 1463 \text{ mm}^2$
 Tension reinforcement provided; A393 Square mesh (10 ϕ bars @ 200 c/c)
 Area of tension reinforcement provided; $A_{sx,bot,prov} = 2670 \text{ mm}^2$
 Minimum area of reinforcement (exp.9.1N); $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times L_y \times d = 1614 \text{ mm}^2$
 Maximum area of reinforcement (cl.9.2.1.1(3)); $A_{s,max} = 0.04 \times L_y \times d = 44880 \text{ mm}^2$

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

Rectangular section in shear (Section 6.2)

Design shear force; $\text{abs}(V_{Ed.x,min}) = 94 \text{ kN}$

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Longitudinal reinforcement ratio;

Design shear resistance (exp.6.2a & 6.2b);

$$C_{Rd,c} = 0.18 / \gamma_c = \mathbf{0.120}$$

$$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = \mathbf{2.000}$$

$$\rho_l = \min(A_{sx,bot,prov} / (L_y \times d), 0.02) = \mathbf{0.003}$$

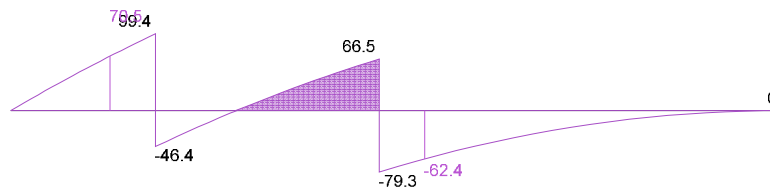
$$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.524 \text{ N/mm}^2}$$

$$V_{Rd,c} = \max(C_{Rd,c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times L_y \times d$$

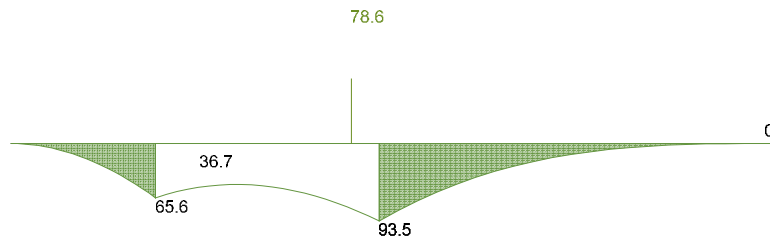
$$V_{Rd,c} = \mathbf{552.1 \text{ kN}}$$

PASS - Design shear resistance exceeds design shear force

Shear diagram, y axis (kN)



Moment diagram, y axis (kNm)



Rectangular section in flexure (Section 6.1)

Design bending moment;

Depth to tension reinforcement;

$$M_{Ed,y,max} = \mathbf{78.6 \text{ kNm}}$$

$$d = h - c_{nom,b} - \phi_{x,bot} - \phi_{y,bot} / 2 = \mathbf{155 \text{ mm}}$$

$$K = M_{Ed,y,max} / (L_x \times d^2 \times f_{ck}) = \mathbf{0.023}$$

$$K' = (2 \times \eta \times \alpha_{cc} / \gamma_c) \times (1 - \lambda \times (\delta - K_1) / (2 \times K_2)) \times (\lambda \times (\delta - K_1) / (2 \times K_2))$$

$$K' = \mathbf{0.207}$$

K' > K - No compression reinforcement is required

Lever arm;

Depth of neutral axis;

Area of tension reinforcement required;

Tension reinforcement provided;

Area of tension reinforcement provided;

Minimum area of reinforcement (exp.9.1N);

Maximum area of reinforcement (cl.9.2.1.1(3));

$$z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_c))^{0.5}, 0.95) \times d = \mathbf{147 \text{ mm}}$$

$$x = 2.5 \times (d - z) = \mathbf{19 \text{ mm}}$$

$$A_{sy,bot,req} = M_{Ed,y,max} / (f_{yd} \times z) = \mathbf{1227 \text{ mm}^2}$$

$$\mathbf{A393 \text{ Square mesh (10 } \phi \text{ bars @ 200 c/c)}}$$

$$A_{sy,bot,prov} = \mathbf{1963 \text{ mm}^2}$$

$$A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times L_x \times d = \mathbf{1115 \text{ mm}^2}$$

$$A_{s,max} = 0.04 \times L_x \times d = \mathbf{31000 \text{ mm}^2}$$

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

Rectangular section in shear (Section 6.2)

Design shear force;

Longitudinal reinforcement ratio;

Design shear resistance (exp.6.2a & 6.2b);

$$V_{Ed,y,max} = \mathbf{70.5 \text{ kN}}$$

$$C_{Rd,c} = 0.18 / \gamma_c = \mathbf{0.120}$$

$$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = \mathbf{2.000}$$

$$\rho_l = \min(A_{sy,bot,prov} / (L_x \times d), 0.02) = \mathbf{0.003}$$

$$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.524 \text{ N/mm}^2}$$

$$V_{Rd,c} = \max(C_{Rd,c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times L_x \times d$$

$$V_{Rd,c} = \mathbf{406 \text{ kN}}$$

PASS - Design shear resistance exceeds design shear force

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Punching shear (Section 6.4)

Strength reduction factor (exp 6.6N);	$v = 0.6 \times [1 - f_{ck} / 250 \text{ N/mm}^2] = \mathbf{0.533}$
Average depth to reinforcement;	$d = \mathbf{155 \text{ mm}}$
Maximum punching shear resistance (cl.6.4.5(3));	$V_{Rd,max} = 0.5 \times v \times f_{cd} = \mathbf{4.227 \text{ N/mm}^2}$ $k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = \mathbf{2.000}$
Longitudinal reinforcement ratio (cl.6.4.4(1));	$\rho_{lx} = A_{sx,bot,prov} / (L_y \times d) = \mathbf{0.003}$ $\rho_{ly} = A_{sy,bot,prov} / (L_x \times d) = \mathbf{0.003}$ $\rho_l = \min(\sqrt{(\rho_{lx} \times \rho_{ly})}, 0.02) = \mathbf{0.003}$ $C_{Rd,c} = 0.18 / \gamma_c = \mathbf{0.120}$ $v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.524 \text{ N/mm}^2}$
Design punching shear resistance (exp.6.47);	$V_{Rd,c} = \max(C_{Rd,c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) = \mathbf{0.524 \text{ N/mm}^2}$
Design punching shear resistance at 1d (exp. 6.50);	$V_{Rd,c1} = (2 \times d / d) \times V_{Rd,c} = \mathbf{1.048 \text{ N/mm}^2}$

Wheel No.1 - Punching shear perimeter at column face

Punching shear perimeter;	$u_0 = \mathbf{1420 \text{ mm}}$
Area within punching shear perimeter;	$A_0 = \mathbf{0.108 \text{ m}^2}$
Maximum punching shear force;	$V_{Ed,max} = \mathbf{66.2 \text{ kN}}$
Punching shear stress factor (fig 6.21N);	$\beta = \mathbf{1.500}$
Maximum punching shear stress (exp 6.38);	$V_{Ed,max} = \beta \times V_{Ed,max} / (u_0 \times d) = \mathbf{0.451 \text{ N/mm}^2}$
PASS - Maximum punching shear resistance exceeds maximum punching shear stress	

Wheel No.1 - Punching shear perimeter at 1d from column face

Punching shear perimeter;	$u_1 = \mathbf{2394 \text{ mm}}$
Area within punching shear perimeter;	$A_1 = \mathbf{0.403 \text{ m}^2}$
Design punching shear force;	$V_{Ed,1} = \mathbf{62.6 \text{ kN}}$
Punching shear stress factor (fig 6.21N);	$\beta = \mathbf{1.500}$
Design punching shear stress (exp 6.38);	$V_{Ed,1} = \beta \times V_{Ed,1} / (u_1 \times d) = \mathbf{0.253 \text{ N/mm}^2}$
PASS - Design punching shear resistance exceeds increased design punching shear stress	

Wheel No.1 - Punching shear perimeter at 2d from column face

Punching shear perimeter;	$u_2 = \mathbf{3368 \text{ mm}}$
Area within punching shear perimeter;	$A_2 = \mathbf{0.850 \text{ m}^2}$
Design punching shear force;	$V_{Ed,2} = \mathbf{57.2 \text{ kN}}$
Punching shear stress factor (fig 6.21N);	$\beta = \mathbf{1.500}$
Design punching shear stress (exp 6.38);	$V_{Ed,2} = \beta \times V_{Ed,2} / (u_2 \times d) = \mathbf{0.164 \text{ N/mm}^2}$
PASS - Design punching shear resistance exceeds design punching shear stress	

Wheel No.2 - Punching shear perimeter at column face

Punching shear perimeter;	$u_0 = \mathbf{1420 \text{ mm}}$
Area within punching shear perimeter;	$A_0 = \mathbf{0.108 \text{ m}^2}$
Maximum punching shear force;	$V_{Ed,max} = \mathbf{77 \text{ kN}}$
Punching shear stress factor (fig 6.21N);	$\beta = \mathbf{1.500}$
Maximum punching shear stress (exp 6.38);	$V_{Ed,max} = \beta \times V_{Ed,max} / (u_0 \times d) = \mathbf{0.525 \text{ N/mm}^2}$
PASS - Maximum punching shear resistance exceeds maximum punching shear stress	

Wheel No.2 - Punching shear perimeter at 1d from column face

Punching shear perimeter;	$u_1 = \mathbf{2394 \text{ mm}}$
Area within punching shear perimeter;	$A_1 = \mathbf{0.403 \text{ m}^2}$
Design punching shear force;	$V_{Ed,1} = \mathbf{73.4 \text{ kN}}$
Punching shear stress factor (fig 6.21N);	$\beta = \mathbf{1.500}$
Design punching shear stress (exp 6.38);	$V_{Ed,1} = \beta \times V_{Ed,1} / (u_1 \times d) = \mathbf{0.297 \text{ N/mm}^2}$

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PASS - Design punching shear resistance exceeds increased design punching shear stress

Wheel No.2 - Punching shear perimeter at 2d from column face

Punching shear perimeter; $u_2 = 3368$ mm
Area within punching shear perimeter; $A_2 = 0.850$ m²
Design punching shear force; $V_{Ed,2} = 68$ kN
Punching shear stress factor (fig 6.21N); $\beta = 1.500$
Design punching shear stress (exp 6.38); $V_{Ed,2} = \beta \times V_{Ed,2} / (u_2 \times d) = 0.195$ N/mm²

PASS - Design punching shear resistance exceeds design punching shear stress

Wheel No.3 - Punching shear perimeter at column face

Punching shear perimeter; $u_0 = 1420$ mm
Area within punching shear perimeter; $A_0 = 0.108$ m²
Maximum punching shear force; $V_{Ed,max} = 77$ kN
Punching shear stress factor (fig 6.21N); $\beta = 1.500$
Maximum punching shear stress (exp 6.38); $V_{Ed,max} = \beta \times V_{Ed,max} / (u_0 \times d) = 0.525$ N/mm²

PASS - Maximum punching shear resistance exceeds maximum punching shear stress

Wheel No.3 - Punching shear perimeter at 1d from column face

Punching shear perimeter; $u_1 = 2394$ mm
Area within punching shear perimeter; $A_1 = 0.403$ m²
Design punching shear force; $V_{Ed,1} = 73.4$ kN
Punching shear stress factor (fig 6.21N); $\beta = 1.500$
Design punching shear stress (exp 6.38); $V_{Ed,1} = \beta \times V_{Ed,1} / (u_1 \times d) = 0.297$ N/mm²

PASS - Design punching shear resistance exceeds increased design punching shear stress

Wheel No.3 - Punching shear perimeter at 2d from column face

Punching shear perimeter; $u_2 = 3368$ mm
Area within punching shear perimeter; $A_2 = 0.850$ m²
Design punching shear force; $V_{Ed,2} = 68$ kN
Punching shear stress factor (fig 6.21N); $\beta = 1.500$
Design punching shear stress (exp 6.38); $V_{Ed,2} = \beta \times V_{Ed,2} / (u_2 \times d) = 0.195$ N/mm²

PASS - Design punching shear resistance exceeds design punching shear stress

Wheel No.4 - Punching shear perimeter at column face

Punching shear perimeter; $u_0 = 1420$ mm
Area within punching shear perimeter; $A_0 = 0.108$ m²
Maximum punching shear force; $V_{Ed,max} = 66.2$ kN
Punching shear stress factor (fig 6.21N); $\beta = 1.500$
Maximum punching shear stress (exp 6.38); $V_{Ed,max} = \beta \times V_{Ed,max} / (u_0 \times d) = 0.451$ N/mm²

PASS - Maximum punching shear resistance exceeds maximum punching shear stress

Wheel No.4 - Punching shear perimeter at 1d from column face

Punching shear perimeter; $u_1 = 2394$ mm
Area within punching shear perimeter; $A_1 = 0.403$ m²
Design punching shear force; $V_{Ed,1} = 62.6$ kN
Punching shear stress factor (fig 6.21N); $\beta = 1.500$
Design punching shear stress (exp 6.38); $V_{Ed,1} = \beta \times V_{Ed,1} / (u_1 \times d) = 0.253$ N/mm²

PASS - Design punching shear resistance exceeds increased design punching shear stress

Wheel No.4 - Punching shear perimeter at 2d from column face

Punching shear perimeter; $u_2 = 3368$ mm

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Area within punching shear perimeter;

$$A_2 = 0.850 \text{ m}^2$$

Design punching shear force;

$$V_{Ed,2} = 57.2 \text{ kN}$$

Punching shear stress factor (fig 6.21N);

$$\beta = 1.500$$

Design punching shear stress (exp 6.38);

$$v_{Ed,2} = \beta \times V_{Ed,2} / (u_2 \times d) = 0.164 \text{ N/mm}^2$$

PASS - Design punching shear resistance exceeds design punching shear stress

