

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

Pipe Cover Slab
Structural Design Calculations

April 2026

Barnsley Marshall Limited

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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
2	P02	Updated to include 400mm diameter pipe cover slab	27/05/26	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:

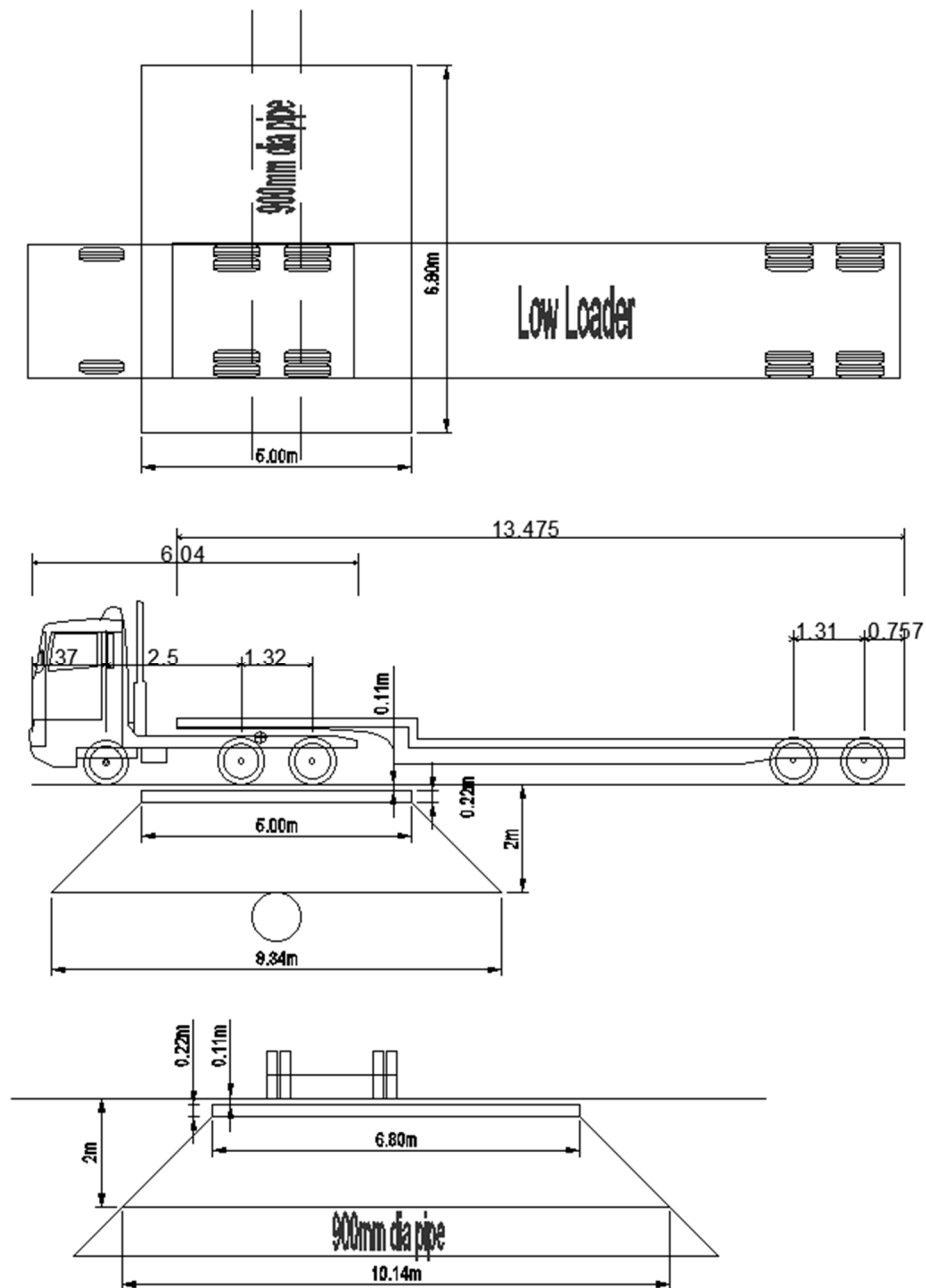


Figure 1

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;

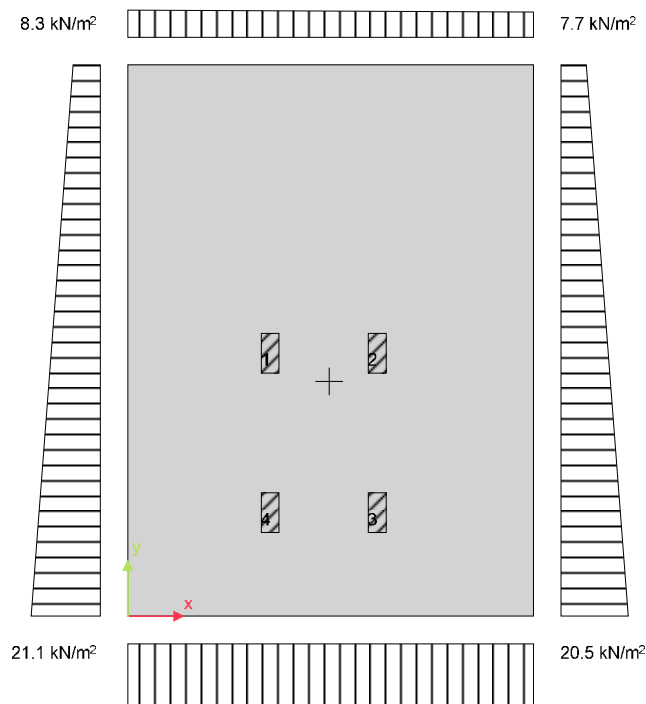
$h_{water} = 0$ mm

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; $\lambda = 0.80$

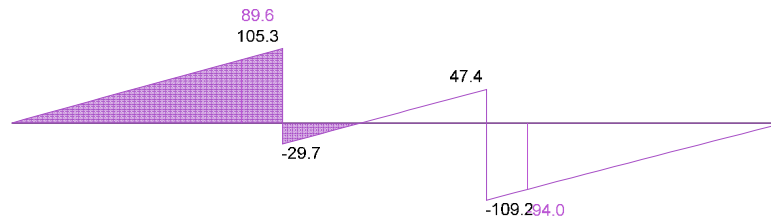
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Effective strength factor; $\eta = 1.00$
 Bending coefficient k_1 ; $K_1 = 0.40$
 Bending coefficient k_2 ; $K_2 = 1.00 \times (0.6 + 0.0014/\epsilon_{cu2}) = 1.00$
 Bending coefficient k_3 ; $K_3 = 0.40$
 Bending coefficient k_4 ; $K_4 = 1.00 \times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

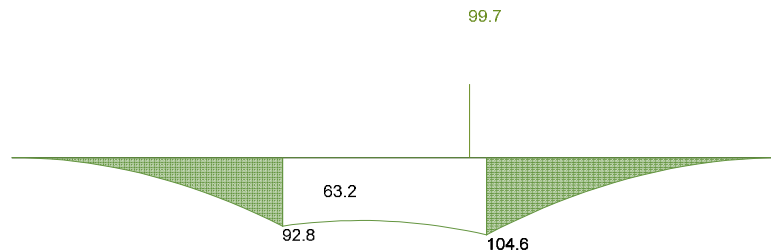
Reinforcement details

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);

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

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

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

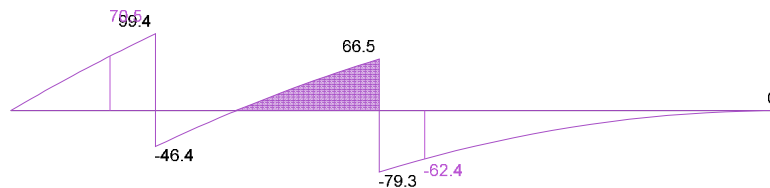
$$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.524 \text{ N/mm}^2}$$

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

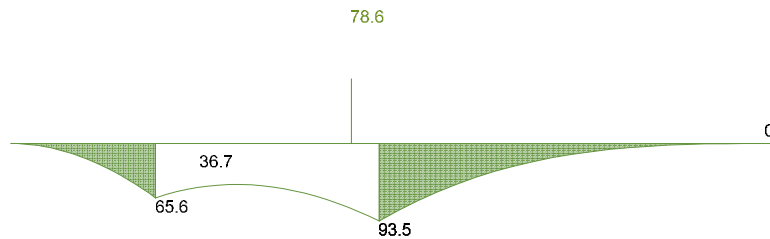
$$V_{Rd,c} = \mathbf{552.1 \text{ kN}}$$

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;

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

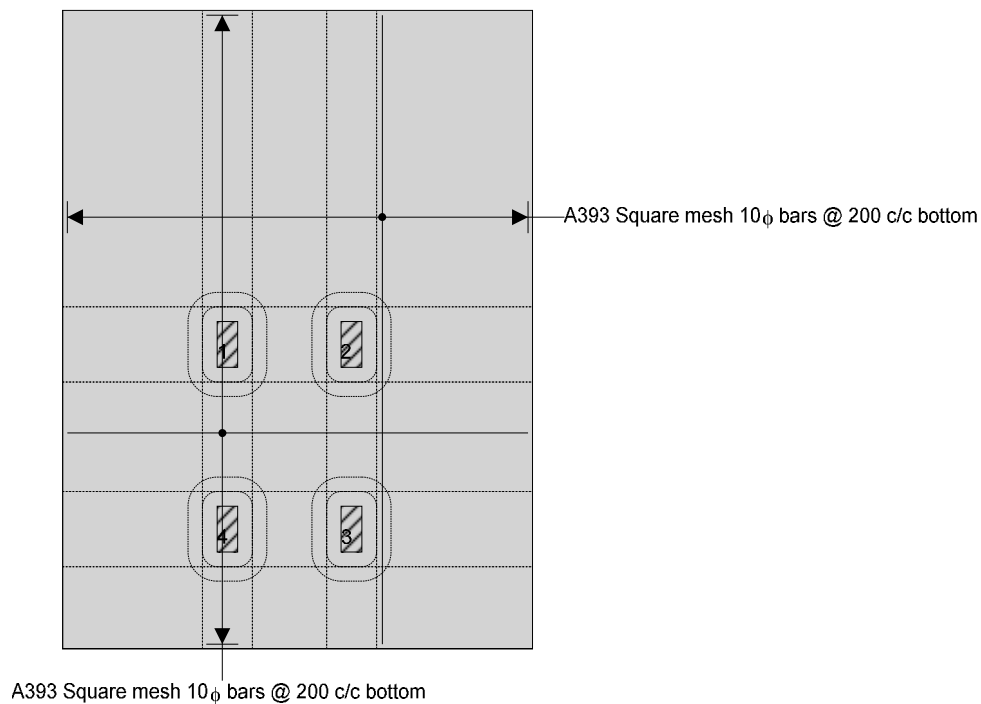
Punching shear stress factor (fig 6.21N);

$$\beta = 1.500$$

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



Calculation Sheet

Project: 1126-01 Kirkby BESS

Sht. No: 3

Rev:

By: NJ

Date: 17/04/26

Title: Pipe cover slabs

Chkd: JP

Date: 17/04/26



Calculations

Output

400mm dia. Pipe Cover Slab

- The 400mm diameter water main pipe is located at 0.75m 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. Therefore, cover slab is to be designed for a point load of $115\text{kN} / 2 = 57.5\text{kN}$ (for each wheel point load on the axial).
- Min. length of slab to be 1.1m to allow for greater then 300mm of bearing each side. Slab is designed to span over the pipe and is to be supported on each side. The design is to consider a single way spanning slab.
- Slab is to be designed for a max. moment 23.7kNm (ULS) - from wheel point load, 1.12kNm (ULS) from self-weight and 1.7kNm (ULS) from permanent surcharge (finishes on top of slab). Therefore, total max. moment = 26.52kNm
- Refer to TEDDS calculations on following pages.
- Therefore, use 220mm thick RC28/35 slab with A393 mesh in the bottom face 50mm cover.

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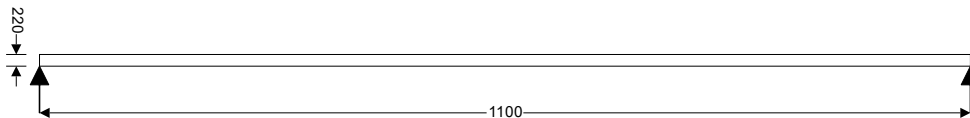
RC SLAB DESIGN

In accordance with EN1992-1-1:2004 + A1:2014 incorporating Corrigenda January 2008, November 2010 and January 2014 and the UK national annex

Tedds calculation version 1.0.25

Design summary

Description	Unit	Provided	Required	Utilisation	Result
Support 1					
Shear	kN/m	86.4	52.9	0.612	PASS
Span 1					
Tension reinf.	mm ² /m	393	389	0.990	PASS
Tension bar spacing	mm	200	300	0.667	PASS
Allow. span-to-depth ratio		6.67	29.49	0.226	PASS
Support 2					
Shear	kN/m	86.4	52.9	0.612	PASS
Cover					
Min cover bottom	mm	50	20	0.400	PASS



Slab definition

Slab reference name	Pipe Cover Slab
Overall slab depth	h = 220 mm
Number of spans	N_{spans} = 1
First support	Simple
Last support	Simple

Nominal cover to bottom reinforcement	C_{nom_b} = 50 mm
---------------------------------------	----------------------------------

Loading

Ratio of quasi-permanent to ultimate load	r_q = 0.300
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Concrete properties

Concrete strength class	C28/35
Characteristic cylinder strength	f_{ck} = 28 N/mm²
Partial factor (Table 2.1N)	γ_C = 1.50
Compressive strength factor (cl. 3.1.6)	α_{cc} = 0.85
Design compressive strength (cl. 3.1.6)	f_{cd} = 15.9 N/mm²
Mean axial tensile strength (Table 3.1)	f_{ctm} = 0.30 N/mm² × (f_{ck} / 1 N/mm²)^{2/3} = 2.8 N/mm²
Maximum aggregate size	d_g = 20 mm

Reinforcement properties

Characteristic yield strength	f_{yk} = 500 N/mm²
Partial factor (Table 2.1N)	γ_S = 1.15
Design yield strength (fig. 3.8)	f_{yd} = f_{yk} / γ_S = 434.8 N/mm²

Concrete cover to reinforcement

Nominal cover to bottom reinforcement	C_{nom_b} = 50 mm
Fire resistance period to bottom of slab	R_{btm} = 30 min
Axis distance to bottom reinf (Table 5.8)	a_{fi_b} = 10 mm

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Max bar diameter in bottom $\phi_{\max_b} = 10 \text{ mm}$
Min. btm cover requirement with regard to bond $C_{\min,b_b} = \phi_{\max_b} = 10 \text{ mm}$
Reinforcement fabrication **Not subject to QA system**
Cover allowance for deviation $\Delta C_{\text{dev}} = 10 \text{ mm}$
Min. required nominal cover to bottom reinf $C_{\text{nom_b_min}} = \max(a_{\text{fi_b}} - \phi_{\max_b} / 2, C_{\min,b_b} + \Delta C_{\text{dev}}) = 20.0 \text{ mm}$
PASS - There is sufficient cover to the bottom reinforcement

Bending design checks

Redistribution ratio $\delta = 1.0$
Limiting value of K $K' = 0.598 \times \delta - 0.18 \times \delta^2 - 0.21 = 0.208$

Reinforcement design at midspan of span 1 (cl.6.1)

Length of span 1 $l_1 = 1100 \text{ mm}$
Design bending moment $M_{p1} = 26.5 \text{ kNm/m}$
Reinforcement provided **A393 mesh**
Area provided $A_{sp1} = 393 \text{ mm}^2/\text{m}$
Effective depth to tension reinforcement $d_{p1} = h - C_{\text{nom_b}} - \phi_{p1} / 2 = 165.0 \text{ mm}$
K factor $K = M_{p1} / (b \times d_{p1}^2 \times f_{ck}) = 0.035$
 $K < K'$ - Compression reinforcement is not required
Lever arm $z = \min(0.95 \times d_{p1}, d_{p1} / 2 \times (1 + \sqrt{(1 - 3.53 \times K)}))$
 $z = 156.7 \text{ mm}$
Area of reinforcement required for bending $A_{sp1_m} = M_{p1} / (f_{yd} \times z) = 389 \text{ mm}^2/\text{m}$
Minimum area required $A_{sp1_min} = \max(0.26 \times (f_{ctm}/f_{yk}), 0.0013) \times b \times d_{p1} = 237 \text{ mm}^2/\text{m}$
Area of reinforcement required $A_{sp1_req} = \max(A_{sp1_m}, A_{sp1_min}) = 389 \text{ mm}^2/\text{m}$
PASS - Area of tension reinforcement provided is adequate (0.990)

Check reinforcement spacing

Reinforcement service stress $\sigma_s = (f_{yk} / \gamma_s) \times \min((A_{sp1_m}/A_{sp1}), 1.0) \times r_q = 129.2 \text{ N/mm}^2$
Maximum allowable spacing (Table 7.3N) $s_{\max_p1} = 300 \text{ mm}$
Actual bar spacing $s_{p1} = 200 \text{ mm}$
PASS - The reinforcement spacing is acceptable

Shear design checks

Shear resistance constant (cl. 6.2.2) $C_{Rd,c} = 0.18 \text{ N/mm}^2 / \gamma_c = 0.12 \text{ N/mm}^2$

Shear capacity check at support 1

Shear force $V_1 = 52.9 \text{ kN/m}$
Reinforcement provided **A393 mesh**
Area provided $A_{sd1} = 393 \text{ mm}^2/\text{m}$
Effective depth $d_{d1} = h - C_{\text{nom_b}} - \phi_{d1} / 2 = 165.0 \text{ mm}$
Effective depth factor (cl. 6.2.2) $k = \min(2.0, 1 + (200 \text{ mm} / d_{d1})^{0.5}) = 2.000$
Reinforcement ratio $\rho_l = \min(0.02, A_{sd1} / (b \times d_{d1})) = 0.0024$
Minimum shear resistance (Exp. 6.3N) $V_{Rd,c_min} = 0.035 \text{ N/mm}^2 \times k^{1.5} \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} \times b \times d_{d1}$
 $V_{Rd,c_min} = 86.4 \text{ kN/m}$
Shear resistance (Exp. 6.2a) $V_{Rd,c1} = \max(V_{Rd,c_min}, C_{Rd,c} \times k \times (100 \times \rho_l \times (f_{ck} / 1 \text{ N/mm}^2))^{0.333} \times b \times d_{d1})$
 $V_{Rd,c1} = 86.4 \text{ kN/m}$
PASS - Shear capacity is adequate (0.612)

Shear capacity check at support 2

Shear force $V_2 = 52.9 \text{ kN/m}$

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Reinforcement provided	A393 mesh
Area provided	$A_{sd2} = 393 \text{ mm}^2/\text{m}$
Effective depth	$d_{d2} = h - C_{nom_b} - \phi_{d2} / 2 = 165.0 \text{ mm}$
Effective depth factor (cl. 6.2.2)	$k = \min(2.0, 1 + (200 \text{ mm} / d_{d2})^{0.5}) = 2.000$
Reinforcement ratio	$\rho_l = \min(0.02, A_{sd2} / (b \times d_{d2})) = 0.0024$
Minimum shear resistance (Exp. 6.3N)	$V_{Rd,c_min} = 0.035 \text{ N/mm}^2 \times k^{1.5} \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} \times b \times d_{d2}$ $V_{Rd,c_min} = 86.4 \text{ kN/m}$
Shear resistance (Exp. 6.2a)	$V_{Rd,c2} = \max(V_{Rd,c_min}, C_{Rd,c} \times k \times (100 \times \rho_l \times (f_{ck} / 1 \text{ N/mm}^2))^{0.333} \times b \times d_{d2})$ $V_{Rd,c2} = 86.4 \text{ kN/m}$
PASS - Shear capacity is adequate (0.612)	

Deflection checks

Basic span-to-depth ratio deflection check span 1 (cl. 7.4.2)

Reference reinforcement ratio	$\rho_0 = (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / 1000 = 0.0053$
Required tension reinforcement ratio	$\rho = \max(0.0035, A_{sp1_m} / (b \times d_{p1})) = 0.0035$
Required compression reinforcement ratio	$\rho' = A_{scp1_req} / (b \times d_{p1}) = 0.0000$
Structural system factor (Table 7.4N)	$K_\delta = 1.0$
Basic span-to-depth ratio limit (Exp. 7.16a)	$ratio_{lim1_bas} = K_\delta \times [11 + 1.5 \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} \times \rho_0 / \rho + 3.2 \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} \times (\rho_0 / \rho - 1)^{1.5}]$ $ratio_{lim1_bas} = 29.20$
Modified span-to-depth ratio limit	$ratio_{lim1} = \min(40 \times K_\delta, \min(1.5, (500 \text{ N/mm}^2 / f_{yk}) \times (A_{sp1} / A_{sp1_m})) \times ratio_{lim1_bas}) = 29.49$
Actual span-to-depth ratio	$ratio_{act1} = l_1 / d_{p1} = 6.67$
PASS - Span-to-depth ratio is acceptable (0.226)	

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.

