ghw Consulting Engineers Ltd
Civil & Structural

18-3470

Henthorn Road, Clitheroe.

STRUCTURAL CALCULATIONS

September 2018

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CALCULATION SHEET

Project No: 18-3470	**Sheet No: 01**	**Rev:**
Project: Henthorn Road, Clitheroe.		
Calcs By: SWH		
Checked By: WG		**Date: Sep-18**

Introduction

1 We have designed the Tank for a 100 Year working life.

2 10 kN/m^2 Surcharge Loading has been adopted.

3 We assume dewatering will be undertaken during the installation of the Tank.

4 We assume that a service search has been carried out by others.

5 We asume that suitable self-compacting backfill will be used to all sides of the manhole, suitable for ground conditons.

6 Density of backfill to be 20 kN/m^3.

7 Ground to be suitable for the bearing pressures exherted via the Tank.

8 Design has been carried out in accordance with BS EN 1992-1-1:2004.

Significant Risks.

a/ Excavations to deep foundations - Contactor to provide method statement.

b/ Construction of drainage, manholes, sewers and external works - Contactor to provide method statement.

c/ The contractor shall give due diligence and consideration to undertake any necessary temporary propping and support during all works with due diligence and reference to the Site Investigation results

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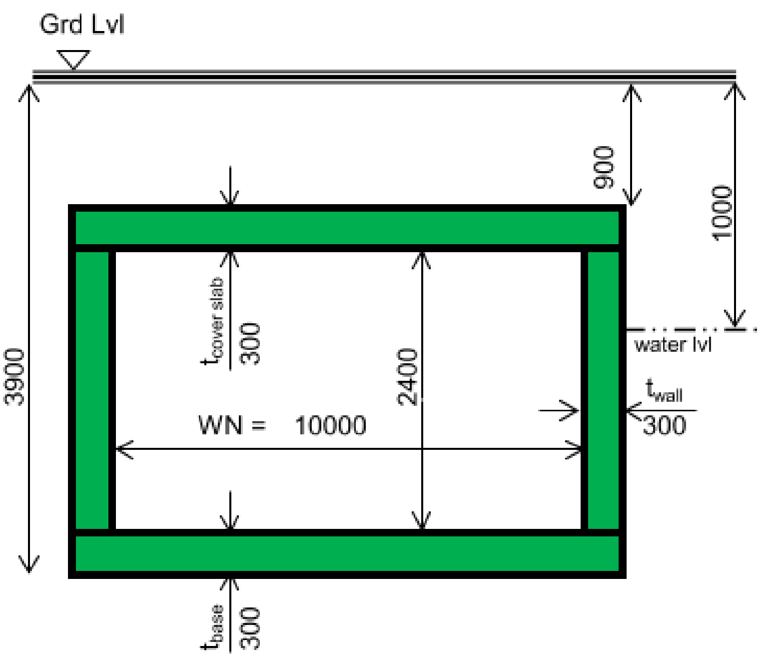
CALCULATION SHEET

Project No: 18-3470	**Sheet No: 02**	**Rev:**
Project: Henthorn Road, Clitheroe.		
Calcs By: SWH		
Checked By: WG		**Date: Sep-18**

Concrete Tank Design 48000 x 10000



Ground Cover = 900 mm

Depth to Water = 1000 mm
Lvl measured from Grd Lvl

Soil Density γ_s = 20.0 kN/m^3

LN = 48000 mm

WN = 10000 mm

t_{wall} = 300 mm

t_{base} = 300 mm

$t_{cover\ slab}$ = 300 mm

Depth to u/s Base = 3900 mm
from Grd Lvl

Concrete Density = 25 kN/m^3

Surcharge Loading = 10.0 kN/m^2

Internal Wall Thickness = 300 mm

Internal wall Height = 2400 mm
(From top of base)

Design Life = 100 Years

Exposure Class = XC3/4, XD1 & XF2

Concrete mix = C45 / 55 f_{ck} = 55 N/mm^2

Min Concrete cover C_{min} = 35 mm

Deviation Allowance C_{dev} = 5 mm

Concrete Cover C_{nom} = **40** mm

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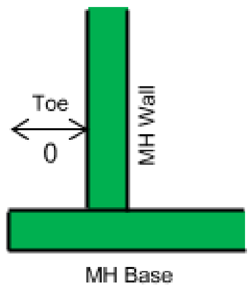
CALCULATION SHEET

Project No: 18-3470	**Sheet No: 03**	**Rev:**
Project: Henthorn Road, Clitheroe.		
Calcs By: SWH		
Checked By: WG		**Date: Sep-18**

Floatation Check

Ground Cover = 900 mm

Toe width = 0 mm
(to all 4 sides)



Inner Tank length LN = 48000 mm

Plan Area = 0.0 m^2
of Toes

Inner Tank width WN = 10000 mm

Base Thickness = 300 mm

Wall Thickness = 300 mm

Cover Slab Thickess = 300 mm

Depth to Water Lvl, measured from Grd Lvl = 1000 mm

Depth to u/s Base from Grd Lvl = 3900 mm

Soil Density = 20 kN/m^3 Concrete Density = 25 kN/m^3

Load Factor = 0.9 Uplift Factor = 1.1

External Tank area = 515.2 m^2 Total Volume of Tank = 1236.5 m^2
(Excluding Cover & Base)

Weight of Cover Slab = 3864 kN Displaced Plan Area = 515.2 m^2

Weight of Tank Walls = 2113 kN Displaced Volume = 1494.1 m^3

Weight of base slab = 3864 kN Internal volume of Tank = 1152.0 m^3

Weight of Grd cover Soil = 9274 kN Area of Cover Slab = 515.2 m^2

Weight of Soil above Toes = 0 kN Area of Base Slab = 515.2 m^2

Buoyancy Uplift Force = 14941 kN Total Dead Load = 19115 kN

***Factored Uplift Force =* 16435.1 kN** ***Total Factored Dead Load =* 17203.5 kN**

SATISFACTORY

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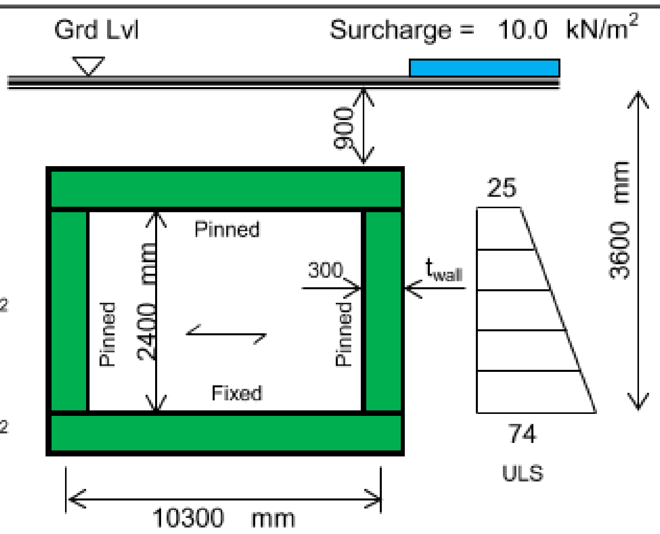
CALCULATION SHEET

Project No: 18-3470	**Sheet No: 04**	**Rev:**
Project: Henthorn Road, Clitheroe.		
Calcs By: SWH		
Checked By: WG		**Date: Sep-18**



Walls

K_o = 0.5

Vertical Soil Pressure P_s = 72 kN/m^2
at the top of the Base Slab

Vertical Water Pressure (uplift) P_w = -26 kN/m^2
(γ_w.h)

Vert. Eff. Overburden Pressure P_{ve} = 46 kN/m^2

Lateral Surcharge pressure S_t = 5.0 kN/m^2

Total SLS Lateral Pressure = 18 kN/m^2
Total SLS Lateral Pressure = 54 kN/m^2

lateral Soil Pressure P_{sh} = 23 kN/m^2
(K_o.P_{ve})

Total Design Lateral Pressure = 74 kN/m^2
at Base $1.35(P_{sh+Pw}) + 1.5(S_t)$

ULS Bm @ Base = 36.8 kNm	ULS Span Bm = 18.5 kNm
Quasi Permanent Bm = 25.3 kNm	Quasi Permanent Bm = 12.7 kNm
Assumed Outer Face = 12 mm Tension Bar Dia	Assumed Inner Face = 12 mm Tension Bar Dia
d = 254 mm	d = 254 mm
k = M/bd^2.f_{ck} = 0.010	k = M/bd^2.f_{ck} = 0.005
L. A. factor z/d = [1-√1-3.53k] = 0.95	L. A. factor z/d = [1-√1-3.53k] = 0.95
z = 241 mm	z = 241 mm
A_s = M/0.87f_yz = 351 mm^2	A_s = M/0.87f_yz = 177 mm^2
USE H 12 @ 125 mm c/c in each face + H10 @ 150mm c/c distribution.	**USE H 12 @ 125 mm c/c in each face + H10 @ 150mm c/c distribution.**
(A_{sprov} = 905 mm^2)	(A_{sprov} = 905 mm^2)

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Cover Slab

Dead

	kN/m^3		m		kN/m^2		γ		ULS
Soil =	20	x	0.9	=	18.0	x	1.35	=	18.00
Cover Slab =	25	x	0.3	=	7.5	x	1.35	=	10.1
					25.5				28.13

Imposed

	kN/m^2				ULS
Traffic =	10	x	1.5	=	15

Total ULS Load = 44.0 kN/m^2

Span = 5.2 m

ULS R = 114 kN

Quasi Permanent Loading = 28.5 kN/m^2

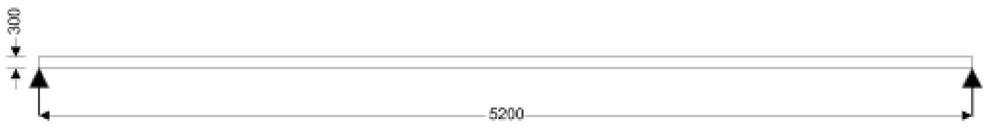
ULS Bm = 149 kNm

Quasi Permanent Bm = 96.3 kNm

RC SLAB DESIGN

In accordance with EN1992-1-1:2004 incorporating corrigendum January 2008 and the UK national annex

Tedds calculation version 1.0.17



Slab definition

Slab reference name	**Cover Slab**
Overall slab depth	h = **300** mm
Number of spans	N_{spans} = **1**
First support	**Simple**
Last support	**Simple**
Nominal cover to bottom reinforcement	c_{nom_b} = **40** mm

Loading

Ratio of quasi-permanent to ultimate load	r_q = **0.300**

Concrete properties

Concrete strength class	C45/55
Characteristic cylinder strength	f_{ck} = **45** N/mm^2
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} = **25.5** N/mm^2
Mean axial tensile strength (Table 3.1)	$f_{ctm} = 0.30\ \text{N/mm}^2 \times (f_{ck} / 1\ \text{N/mm}^2)^{2/3}$ = **3.8** 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)	γ_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 bottom reinforcement	c_{nom_b} = **40** mm
Fire resistance period to bottom of slab	R_{btm} = **60** min
Axis distance to bottom reinft (Table 5.8)	a_{fi_b} = **20** mm
Max bar diameter in bottom	ϕ_{max_b} = **20** mm
Min. btm cover requirement with regard to bond	$c_{min,b\_b} = \phi_{max\_b}$ = **20** mm
Reinforcement fabrication	**Not subject to QA system**
Cover allowance for deviation	Δc_{dev} = **10** mm
Min. required nominal cover to bottom reinft	$c_{nom\_b\_min} = \max(a_{fi\_b} - \phi_{max\_b} / 2, c_{min,b\_b} + \Delta c_{dev})$ = **30.0** mm

PASS - There is sufficient cover to the bottom reinforcement

Bending design checks

Redistribution ratio	δ = **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 = **5200** mm

Design bending moment — M_{p1} = **149.0** kNm/m

Reinforcement provided — **20 mm dia. bars at 150 mm centres**

Area provided — A_{sp1} = **2094** mm^2/m

Effective depth to tension reinforcement — $d_{p1} = h - c_{nom\_b} - \phi_{p1} / 2$ = **250.0** mm

K factor — $K = M_{p1} / (b \times d_{p1}^2 \times f_{ck})$ = **0.053**

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 = **237.5** mm

Area of reinforcement required for bending — $A_{sp1\_m} = M_{p1} / (f_{yd} \times z)$ = **1443** mm^2/m

Minimum area required — $A_{sp1\_min} = \max(0.26 \times (f_{ctm}/f_{yk}), 0.0013) \times b \times d_{p1}$ = **493** mm^2/m

Area of reinforcement required — $A_{sp1\_req} = \max(A_{sp1\_m}, A_{sp1\_min})$ = **1443** mm^2/m

PASS - Area of tension reinforcement provided is adequate (0.689)

Check reinforcement spacing

Reinforcement service stress — $\sigma_s = (f_{yk} / \gamma_S) \times \min((A_{sp1\_m}/A_{sp1}), 1.0) \times r_q$ = **89.9** N/mm^2

Maximum allowable spacing (Table 7.3N) — s_{max_p1} = **300** mm

Actual bar spacing — s_{p1} = **150** mm

PASS - The reinforcement spacing is acceptable

Shear design checks

Shear resistance constant (cl. 6.2.2) — $C_{Rd,c}$ = 0.18 N/mm^2 / γ_C = **0.12** N/mm^2

Shear capacity check at support 1

Shear force — V_1 = **114.0** kN/m

Reinforcement provided — **20 mm dia. bars at 150 mm centres**

Area provided — A_{sd1} = **2094** mm^2/m

Effective depth — $d_{d1} = h - c_{nom\_b} - \phi_{d1} / 2$ = **250.0** mm

Effective depth factor (cl. 6.2.2) — $k = \min(2.0, 1 + (200 \text{ mm} / d_{d1})^{0.5})$ = **1.894**

Reinforcement ratio — $\rho_l = \min(0.02, A_{sd1} / (b \times d_{d1}))$ = **0.0084**

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} = **153.0** kN/m

Shear resistance (Exp. 6.2a) — $V_{Rd,c1} = \max(V_{Rd,c\_min}, C_{Rd,c} \times k \times (100 \times \rho_l \times (f_{ck}/1 \text{ N/mm}^2))^{0.333} \times b \times d_{d1})$

$V_{Rd,c1}$ = **190.3** kN/m

PASS - Shear capacity is adequate (0.599)

Shear capacity check at support 2

Shear force — V_2 = **114.0** kN/m

Reinforcement provided — **20 mm dia. bars at 150 mm centres**

Area provided — A_{sd2} = **2094** mm^2/m

Effective depth — $d_{d2} = h - c_{nom\_b} - \phi_{d2} / 2$ = **250.0** mm

Effective depth factor (cl. 6.2.2) — $k = \min(2.0, 1 + (200 \text{ mm} / d_{d2})^{0.5})$ = **1.894**

Reinforcement ratio — $\rho_l = \min(0.02, A_{sd2} / (b \times d_{d2}))$ = **0.0084**

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} = **153.0** kN/m

Shear resistance (Exp. 6.2a) — $V_{Rd,c2} = \max(V_{Rd,c\_min}, C_{Rd,c} \times k \times (100 \times \rho_l \times (f_{ck}/1 \text{ N/mm}^2))^{0.333} \times b \times d_{d2})$

$V_{Rd,c2}$ = **190.3** kN/m

PASS - Shear capacity is adequate (0.599)

Deflection checks

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

Reference reinforcement ratio $\rho_0 = (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / 1000 = \mathbf{0.0067}$

Required tension reinforcement ratio $\rho = \max(0.0035, A_{sp1\_m} / (b \times d_{p1})) = \mathbf{0.0058}$

Required compression reinforcement ratio $\rho' = A_{scp1\_req} / (b \times d_{p1}) = \mathbf{0.0000}$

Structural system factor (Table 7.4N) $K_\delta = \mathbf{1.0}$

Basic span-to-depth ratio limit $ratio_{lim1\_bas} = K_\delta \times [11 + 1.5 \times (f_{ck}/1 \text{ N/mm}^2)^{0.5} \times \rho_0/\rho + 3.2 \times (f_{ck}/1 \text{ N/mm}^2)^{0.5} \times (\rho_0/\rho - 1)^{1.5}]$

(Exp. 7.16a) $ratio_{lim1\_bas} = \mathbf{24.10}$

Modified span-to-depth ratio limit

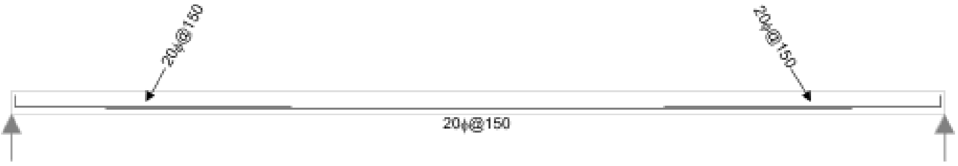
$ratio_{lim1} = \min(40 \times K_\delta, \min(1.5, (500 \text{ N/mm}^2/ f_{yk}) \times (A_{sp1} / A_{sp1\_m})) \times ratio_{lim1\_bas}) = \mathbf{34.98}$

Actual span-to-depth ratio $ratio_{act1} = l_1 / d_{p1} = \mathbf{20.80}$

PASS - Span-to-depth ratio is acceptable (0.595)

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.



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CALCULATION SHEET

Project No: 18-3470	**Sheet No: 06**	**Rev:**
Project: Henthorn Road, Clitheroe.		
Calcs By: SWH		
Checked By: WG		**Date: Sep-18**

Internal Wall Design

Beam

Design as Continuous Beam

Depth of Beam = 300 mm   Width of Beam = 300

Span = 1.5 m

			kN/m^2		m				kN/m
Loading :- SLS	Dead	Soil =	18.0	x	10.4	/	2	=	93.6
		Slab =	7.5	x	10.4	/	2	=	39.0
									132.6
	Imposed	Traffic =	10.0	x	10.4	/	2	=	52

Column

Height = 2.0 m

Loading :- SLS	Dead =	132.6	x	1.5	=	199	kN
	Imposed =	52.0	x	1.5	=	78	kN
						277	

RC MEMBER ANALYSIS & DESIGN (EN1992-1-1:2004)

In accordance with EN1992-1-1:2004 incorporating Corrigenda January 2008 and the UK national annex

Tedds calculation version 3.2.00

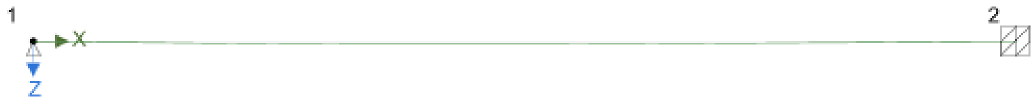
ANALYSIS

Tedds calculation version 1.0.24

Results

Total deflection

1.35G + 1.5Q + 1.5RQ (Strength) - Total deflection



1.0G + 1.0Q + 1.0RQ (Service) - Total deflection

1.0G + 1.0ψ_2Q (Quasi) - Total deflection

Node deflections

Load combination: 1.35G + 1.5Q + 1.5RQ (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.04645	
2	0	0	0	

Load combination: 1.0G + 1.0Q + 1.0RQ (Service)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.03338	
2	0	0	0	

Load combination: 1.0G + 1.0ψ_2Q (Quasi)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.02689	
2	0	0	0	

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.5Q + 1.5RQ (Strength)	0	390.8
1.0G + 1.0Q + 1.0RQ (Service)	0	280.8
1.0G + 1.0ψ_2Q (Quasi)	0	226.2

Element end forces

Load combination: 1.35G + 1.5Q + 1.5RQ (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	1.5	1	0	-147.9	0
		2	0	-242.9	-71.2

Load combination: 1.0G + 1.0Q + 1.0RQ (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	1.5	1	0	-106.3	0
		2	0	-174.5	-51.2

Load combination: 1.0G + 1.0ψ_2Q (Quasi)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	1.5	1	0	-85.6	0
		2	0	-140.6	-41.2

Forces

Strength combinations - Moment envelope (kNm)

Strength combinations - Shear envelope (kN)

Member results

Envelope - Strength combinations

Member	Position (m)	Shear force (kN)		Moment (kNm)	
Beam	0	147.9		0	

Member	Position (m)	Shear force (kN)		Moment (kNm)	
	0.568	0		42 (max)	
	1.5	-242.9 (max abs)		-71.2 (min)	

Concrete details - Table 3.1. Strength and deformation characteristics for concrete

Concrete strength class	C45/55
Aggregate type	Quartzite
Aggregate adjustment factor - cl.3.1.3(2)	AAF = **1.0**
Characteristic compressive cylinder strength	f_{ck} = **45** N/mm^2
Mean value of compressive cylinder strength	$f_{cm} = f_{ck} + 8$ N/mm^2 = **53** N/mm^2
Mean value of axial tensile strength	$f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck}/ 1$ N/mm$^2)^{2/3}$ = **3.8** 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 = **36283** N/mm^2
Ultimate strain - Table 3.1	ε_{cu2} = **0.0035**
Shortening strain - Table 3.1	ε_{cu3} = **0.0035**
Effective compression zone height factor	λ = **0.80**
Effective strength factor	η = **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	γ_C = **1.50**
Compressive strength coefficient - cl.3.1.6(1)	α_{cc} = **0.85**
Design compressive concrete strength - exp.3.15	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **25.5** N/mm^2
Compressive strength coefficient - cl.3.1.6(1)	α_{ccw} = **1.00**
Design compressive concrete strength - exp.3.15	$f_{cwd} = \alpha_{ccw} \times f_{ck} / \gamma_C$ = **30.0** N/mm^2
Maximum aggregate size	h_{agg} = **20** mm
Monolithic simple support moment factor	β_1 = **0.25**

Reinforcement details

Characteristic yield strength of reinforcement	f_{yk} = **500** N/mm^2
Partial factor for reinforcing steel - Table 2.1N	γ_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} = **40** mm
Nominal cover to bottom reinforcement	c_{nom_b} = **40** mm
Nominal cover to side reinforcement	c_{nom_s} = **40** 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

Beam - Span 1

Rectangular section details

Section width	b = **300** mm
Section depth	h = **300** mm

PASS - Minimum dimensions for fire resistance met

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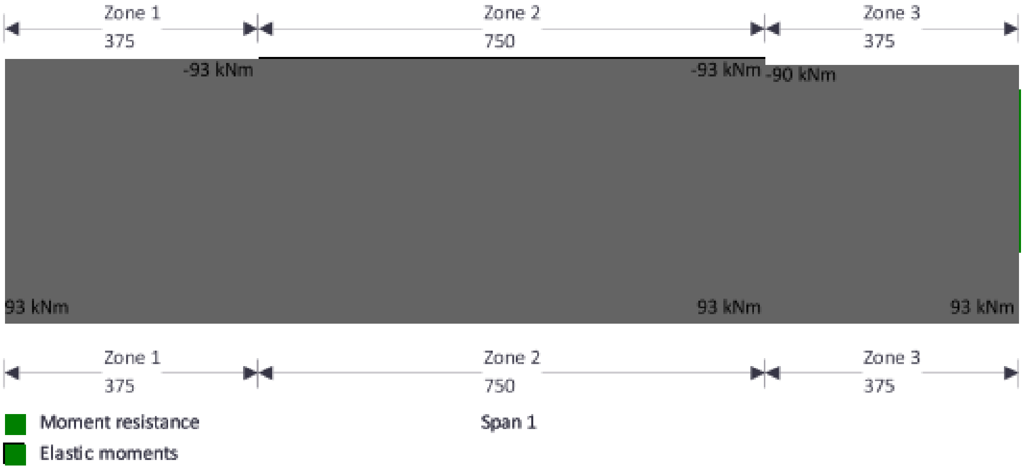
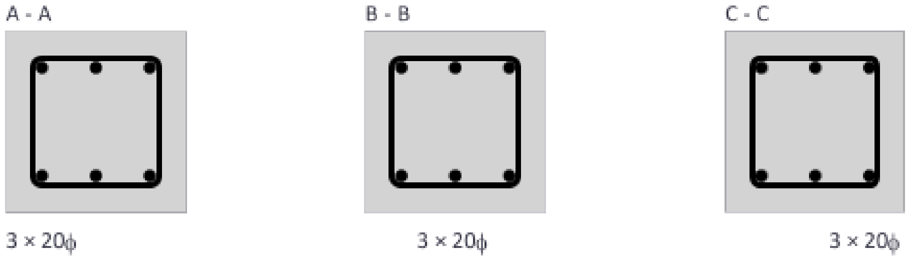
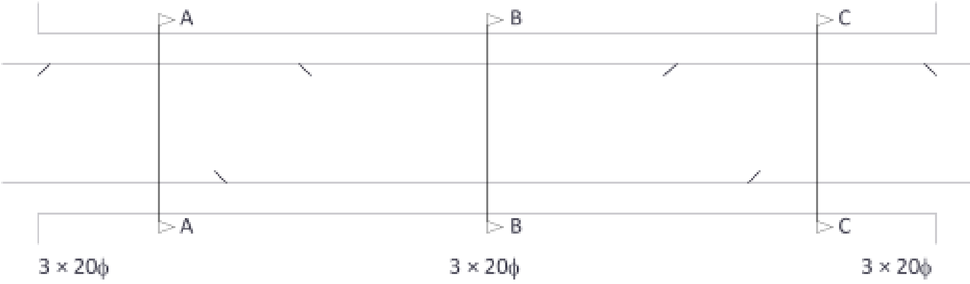
	Calcs for Beam to Internal Wall.				Start page no./Revision 4	
	Calcs by SWH	Calcs date 18/09/2018	Checked by WG	Checked date	Approved by	Approved date

Moment design







Zone 1 (0 mm - 375 mm) Positive moment - section 6.1

Design bending moment $M = abs(M_{m1\_s1\_z1\_max\_red})$ = **37.1** kNm

Effective depth of tension reinforcement d = **240** mm

Redistribution ratio $\delta = \min(M_{pos\_red\_z1} / M_{pos\_z1}, 1)$ = **1.000**

$K = M / (b \times d^2 \times f_{ck})$ = **0.048**

$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2))$ = **0.207**

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d)$ = **228** mm

Depth of neutral axis $x = 2 \times (d - z) / \lambda$ = **30** mm

Area of tension reinforcement required $A_{s,req} = M / (f_{yd} \times z)$ = **375** mm²

Tension reinforcement provided 3 × 20ϕ

Area of tension reinforcement provided $A_{s,prov}$ = **942** mm²

Minimum area of reinforcement - exp.9.1N $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d$ = **142** mm²

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{s,max} = 0.04 \times b \times h$ = **3600** mm²

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

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	$f_{ct,eff} = f_{ctm}$ = **3.8** N/mm^2
Stress distribution coefficient	k_c = **0.4**
Non-uniform self-equilibrating stress coefficient	k = min(max(1 + (300 mm - min(h, b)) × 0.35 / 500 mm, 0.65), 1) = **1.00**
Actual tension bar spacing	$s_{bar} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / (N_{m1\_s1\_z1\_b\_L1} - 1) + \phi_{m1\_s1\_z1\_b\_L1}$ = **90** mm
Maximum stress permitted - Table 7.3N	σ_s = **328** N/mm^2
Steel to concrete modulus of elast. ratio	$\alpha_{cr} = E_s / E_{cm}$ = **5.51**
Distance of the Elastic NA from bottom of beam	$y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1))$ = **146** mm
Area of concrete in the tensile zone	$A_{ct} = b \times y$ = **43782** mm^2
Minimum area of reinforcement required - exp.7.1	$A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s$ = **203** mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent moment	$M_{QP} = \max(\beta_1 \times abs(M_{m1\_s1\_z2\_neg\_quasi}), abs(M_{m1\_s1\_z1\_pos\_quasi}))$ = **21.5**kNm
Permanent load ratio	$R_{PL} = M_{QP} / M$ = **0.58**
Service stress in reinforcement	$\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL}$ = **100** N/mm^2
Maximum bar spacing - Tables 7.3N	$s_{bar,max}$ = **300** mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Zone 1 (0 mm - 375 mm) Negative moment - section 6.1

Design bending moment	$M = \max(\beta_1 \times abs(M_{m1\_s1\_max\_red}), abs(M_{m1\_s1\_z1\_min\_red}))$ = **10.5** kNm
Effective depth of tension reinforcement	d = **240** mm
Redistribution ratio	δ = 1 = **1.000**
	$K = M / (b \times d^2 \times f_{ck})$ = **0.013**
	$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2))$ = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d)$ = **228** mm
Depth of neutral axis	$x = 2 \times (d - z) / \lambda$ = **30** mm
Area of tension reinforcement required	$A_{s,req} = M / (f_{yd} \times z)$ = **106** mm^2
Tension reinforcement provided	3 × 20ϕ
Area of tension reinforcement provided	$A_{s,prov}$ = **942** mm^2
Minimum area of reinforcement - exp.9.1N	$A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d$ = **142** mm^2
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{s,max} = 0.04 \times b \times h$ = **3600** mm^2

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

Crack control - Section 7.3

Maximum crack width	w_k = **0.3** mm
Design value modulus of elasticity reinf – 3.2.7(4)	E_s = **200000** N/mm^2
Mean value of concrete tensile strength	$f_{ct,eff} = f_{ctm}$ = **3.8** N/mm^2
Stress distribution coefficient	k_c = **0.4**
Non-uniform self-equilibrating stress coefficient	k = min(max(1 + (300 mm - min(h, b)) × 0.35 / 500 mm, 0.65), 1) = **1.00**

Actual tension bar spacing $s_{bar} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_t\_L1} \times N_{m1\_s1\_z1\_t\_L1})) / (N_{m1\_s1\_z1\_t\_L1} - 1) + \phi_{m1\_s1\_z1\_t\_L1}$ = **90** mm

Maximum stress permitted - Table 7.3N σ_s = **328** N/mm^2

Steel to concrete modulus of elast. ratio $\alpha_{cr} = E_s / E_{cm}$ = **5.51**

Distance of the Elastic NA from bottom of beam $y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1))$ = **146** mm

Area of concrete in the tensile zone $A_{ct} = b \times y$ = **43782** mm^2

Minimum area of reinforcement required - exp.7.1 $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s$ = **203** mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent moment $M_{QP} = \max(\beta_1 \times \mathrm{abs}(M_{m1\_s1\_z2\_pos\_quasi}), \mathrm{abs}(M_{m1\_s1\_z1\_neg\_quasi}))$ = **6.1**kNm

Permanent load ratio $R_{PL} = M_{QP} / M$ = **0.58**

Service stress in reinforcement $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL}$ = **28** N/mm^2

Maximum bar spacing - Tables 7.3N $s_{bar,max}$ = **300** mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Minimum bar spacing (Section 8.2)

Top bar spacing $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_t\_L1} \times N_{m1\_s1\_z1\_t\_L1})) / (N_{m1\_s1\_z1\_t\_L1} - 1)$ = **70.0** mm

Minimum allowable top bar spacing $s_{top,min} = \max(\phi_{m1\_s1\_z1\_t\_L1} \times k_{s1}, h_{agg} + k_{s2}, 20mm)$ = **25.0** mm

PASS - Actual bar spacing exceeds minimum allowable

Bottom bar spacing $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / (N_{m1\_s1\_z1\_b\_L1} - 1)$ = **70.0** mm

Minimum allowable bottom bar spacing $s_{bot,min} = \max(\phi_{m1\_s1\_z1\_b\_L1} \times k_{s1}, h_{agg} + k_{s2}, 20mm)$ = **25.0** mm

PASS - Actual bar spacing exceeds minimum allowable

Zone 2 (375 mm - 1125 mm) Positive moment - section 6.1

Design bending moment $M = \mathrm{abs}(M_{m1\_s1\_z2\_max\_red})$ = **42.0** kNm

Effective depth of tension reinforcement d = **240** mm

Redistribution ratio $\delta = \min(M_{pos\_red\_z2} / M_{pos\_z2}, 1)$ = **1.000**

$K = M / (b \times d^2 \times f_{ck})$ = **0.054**

$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2))$ = **0.207**

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d)$ = **228** mm

Depth of neutral axis $x = 2 \times (d - z) / \lambda$ = **30** mm

Area of tension reinforcement required $A_{s,req} = M / (f_{yd} \times z)$ = **424** mm^2

Tension reinforcement provided 3 × 20ϕ

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

Minimum area of reinforcement - exp.9.1N $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d$ = **142** mm^2

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{s,max} = 0.04 \times b \times h$ = **3600** mm^2

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

Crack control - Section 7.3

Maximum crack width w_k = **0.3** mm

Design value modulus of elasticity reinf – 3.2.7(4) E_s = **200000** N/mm^2

Mean value of concrete tensile strength $f_{ct,eff} = f_{ctm}$ = **3.8** 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 $s_{bar} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2\_b\_L1} \times N_{m1\_s1\_z2\_b\_L1} + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / ((N_{m1\_s1\_z2\_b\_L1} + N_{m1\_s1\_z1\_b\_L1}) - 1) + \phi_{m1\_s1\_z2\_b\_L1}$ = **90** mm

Maximum stress permitted - Table 7.3N σ_s = **328** N/mm^2

Steel to concrete modulus of elast. ratio $\alpha_{cr} = E_s / E_{cm}$ = **5.51**

Distance of the Elastic NA from bottom of beam $y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1))$ = **146** mm

Area of concrete in the tensile zone $A_{ct} = b \times y$ = **43782** mm^2

Minimum area of reinforcement required - exp.7.1 $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s$ = **203** mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent moment $M_{QP} = \text{abs}(M_{m1\_s1\_z2\_pos\_quasi})$ = **24.3**kNm

Permanent load ratio $R_{PL} = M_{QP} / M$ = **0.58**

Service stress in reinforcement $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL}$ = **113** N/mm^2

Maximum bar spacing - Tables 7.3N $s_{bar,max}$ = **300** mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Deflection control - Section 7.4

Reference reinforcement ratio $\rho_{m0} = (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / 1000$ = **0.00671**

Required tension reinforcement ratio $\rho_m = A_{s,req} / (b \times d)$ = **0.00588**

Required compression reinforcement ratio $\rho'_m = A_{s2,req} / (b \times d)$ = **0.00000**

Structural system factor - Table 7.4N K_b = **1.0**

Basic allowable span to depth ratio $\text{span\_to\_depth}_{basic} = K_b \times [11 + 1.5 \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} \times \rho_{m0} / \rho_m + 3.2 \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} \times (\rho_{m0} / \rho_m - 1)^{1.5}]$ = **23.597**

Reinforcement factor - exp.7.17 $K_s = \min(A_{s,prov} / A_{s,req} \times 500 \text{ N/mm}^2 / f_{yk}, 1.5)$ = **1.500**

Flange width factor F1 = 1 = **1.000**

Long span supporting brittle partition factor F2 = 1 = **1.000**

Allowable span to depth ratio $\text{span\_to\_depth}_{allow} = \min(\text{span\_to\_depth}_{basic} \times K_s \times F1 \times F2, 40 \times K_b)$ = **35.396**

Actual span to depth ratio $\text{span\_to\_depth}_{actual} = L_{m1\_s1} / d$ = **6.250**

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

Minimum bar spacing (Section 8.2)

Top bar spacing $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2\_t\_L1} \times N_{m1\_s1\_z2\_t\_L1})) / (N_{m1\_s1\_z2\_t\_L1} - 1)$ = **70.0** mm

Minimum allowable top bar spacing $s_{top,min} = \max(\phi_{m1\_s1\_z2\_t\_L1} \times k_{s1}, h_{agg} + k_{s2}, 20\text{mm})$ = **25.0** mm

PASS - Actual bar spacing exceeds minimum allowable

Bottom bar spacing $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2\_b\_L1} \times N_{m1\_s1\_z2\_b\_L1} + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / ((N_{m1\_s1\_z2\_b\_L1} + N_{m1\_s1\_z1\_b\_L1}) - 1)$ = **70.0** mm

Minimum allowable bottom bar spacing $s_{bot,min} = \max(\phi_{m1\_s1\_z2\_b\_L1} \times k_{s1}, h_{agg} + k_{s2}, 20\text{mm})$ = **25.0** mm

PASS - Actual bar spacing exceeds minimum allowable

Zone 3 (1125 mm - 1500 mm) Positive moment - section 6.1

Design bending moment $M = \text{abs}(M_{m1\_s1\_z3\_max\_red})$ = **1.5** kNm

Effective depth of tension reinforcement d = **240** mm

Redistribution ratio $\delta = \min(M_{pos\_red\_z3} / M_{pos\_z3}, 1)$ = **1.000**

	$K = M / (b \times d^2 \times f_{ck}) = \mathbf{0.002}$
	$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d) = \mathbf{228}$ mm
Depth of neutral axis	$x = 2 \times (d - z) / \lambda = \mathbf{30}$ mm
Area of tension reinforcement required	$A_{s,req} = M / (f_{yd} \times z) = \mathbf{16}$ mm^2
Tension reinforcement provided	$3 \times 20\phi$
Area of tension reinforcement provided	$A_{s,prov} = \mathbf{942}$ mm^2
Minimum area of reinforcement - exp.9.1N	$A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d = \mathbf{142}$ mm^2
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{s,max} = 0.04 \times b \times h = \mathbf{3600}$ mm^2

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

Crack control - Section 7.3

Maximum crack width	$w_k = \mathbf{0.3}$ mm
Design value modulus of elasticity reinf – 3.2.7(4)	$E_s = \mathbf{200000}$ N/mm^2
Mean value of concrete tensile strength	$f_{ct,eff} = f_{ctm} = \mathbf{3.8}$ N/mm^2
Stress distribution coefficient	$k_c = \mathbf{0.4}$
Non-uniform self-equilibrating stress coefficient	$k = \min(\max(1 + (300 \text{ mm} - \min(h, b)) \times 0.35 / 500 \text{ mm}, 0.65), 1) = \mathbf{1.00}$
Actual tension bar spacing	$s_{bar} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_b\_L1} \times N_{m1\_s1\_z3\_b\_L1})) / (N_{m1\_s1\_z3\_b\_L1} - 1) + \phi_{m1\_s1\_z3\_b\_L1} = \mathbf{90}$ mm
Maximum stress permitted - Table 7.3N	$\sigma_s = \mathbf{328}$ N/mm^2
Steel to concrete modulus of elast. ratio	$\alpha_{cr} = E_s / E_{cm} = \mathbf{5.51}$
Distance of the Elastic NA from bottom of beam	$y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1)) = \mathbf{146}$ mm
Area of concrete in the tensile zone	$A_{ct} = b \times y = \mathbf{43782}$ mm^2
Minimum area of reinforcement required - exp.7.1	$A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s = \mathbf{203}$ mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent moment	$M_{QP} = \max(\beta_1 \times \text{abs}(M_{m1_s1_z2_neg_quasi}), \text{abs}(M_{m1_s1_z3_pos_quasi})) = \mathbf{0.9}$kNm
Permanent load ratio	$R_{PL} = M_{QP} / M = \mathbf{0.58}$
Service stress in reinforcement	$\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL} = \mathbf{4}$ N/mm^2
Maximum bar spacing - Tables 7.3N	$s_{bar,max} = \mathbf{300}$ mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Zone 3 (1125 mm - 1500 mm) Negative moment - section 6.1

Design bending moment	$M = \max(\beta_1 \times \text{abs}(M_{m1\_s1\_max\_red}), \text{abs}(M_{m1\_s1\_z3\_min\_red})) = \mathbf{71.2}$ kNm
Effective depth of tension reinforcement	$d = \mathbf{240}$ mm
Redistribution ratio	$\delta = \min(M_{neg\_red\_z3} / M_{neg\_z3}, 1) = \mathbf{1.000}$
	$K = M / (b \times d^2 \times f_{ck}) = \mathbf{0.092}$
	$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d) = \mathbf{219}$ mm
Depth of neutral axis	$x = 2 \times (d - z) / \lambda = \mathbf{53}$ mm
Area of tension reinforcement required	$A_{s,req} = M / (f_{yd} \times z) = \mathbf{749}$ mm^2
Tension reinforcement provided	$3 \times 20\phi$

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

Minimum area of reinforcement - exp.9.1N $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d =$ **142** mm^2

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{s,max} = 0.04 \times b \times h =$ **3600** mm^2

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

Crack control - Section 7.3

Maximum crack width w_k = **0.3** mm

Design value modulus of elasticity reinf – 3.2.7(4) E_s = **200000** N/mm^2

Mean value of concrete tensile strength $f_{ct,eff} = f_{ctm}$ = **3.8** 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 $s_{bar} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_t\_L1} \times N_{m1\_s1\_z3\_t\_L1})) / (N_{m1\_s1\_z3\_t\_L1} - 1) + \phi_{m1\_s1\_z3\_t\_L1} =$ **90** mm

Maximum stress permitted - Table 7.3N σ_s = **328** N/mm^2

Steel to concrete modulus of elast. ratio $\alpha_{cr} = E_s / E_{cm} =$ **5.51**

Distance of the Elastic NA from bottom of beam $y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1)) =$ **146** mm

Area of concrete in the tensile zone $A_{ct} = b \times y =$ **43782** mm^2

Minimum area of reinforcement required - exp.7.1 $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s =$ **203** mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent moment $M_{QP} = \max(\beta_1 \times \text{abs}(M_{m1\_s1\_z2\_pos\_quasi}), \text{abs}(M_{m1\_s1\_z3\_neg\_quasi})) =$ **41.2**kNm

Permanent load ratio $R_{PL} = M_{QP} / M =$ **0.58**

Service stress in reinforcement $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL} =$ **200** N/mm^2

Maximum bar spacing - Tables 7.3N $s_{bar,max}$ = **250** mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Minimum bar spacing (Section 8.2)

Top bar spacing $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_t\_L1} \times N_{m1\_s1\_z3\_t\_L1})) / (N_{m1\_s1\_z3\_t\_L1} - 1) =$ **70.0** mm

Minimum allowable top bar spacing $s_{top,min} = \max(\phi_{m1\_s1\_z3\_t\_L1} \times k_{s1}, h_{agg} + k_{s2}, 20\text{mm}) =$ **25.0** mm

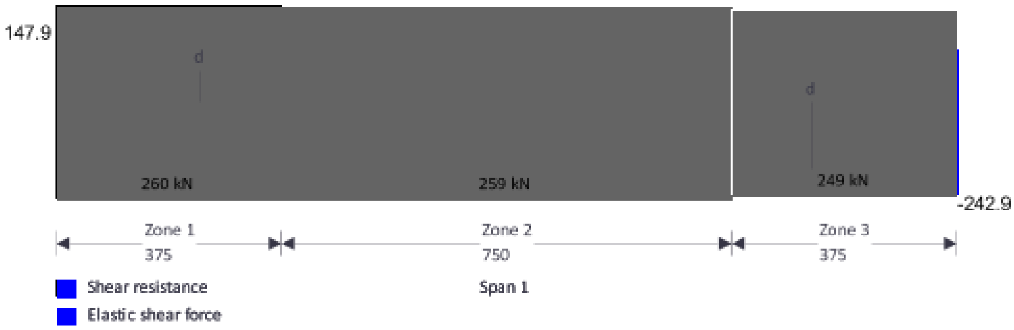
PASS - Actual bar spacing exceeds minimum allowable

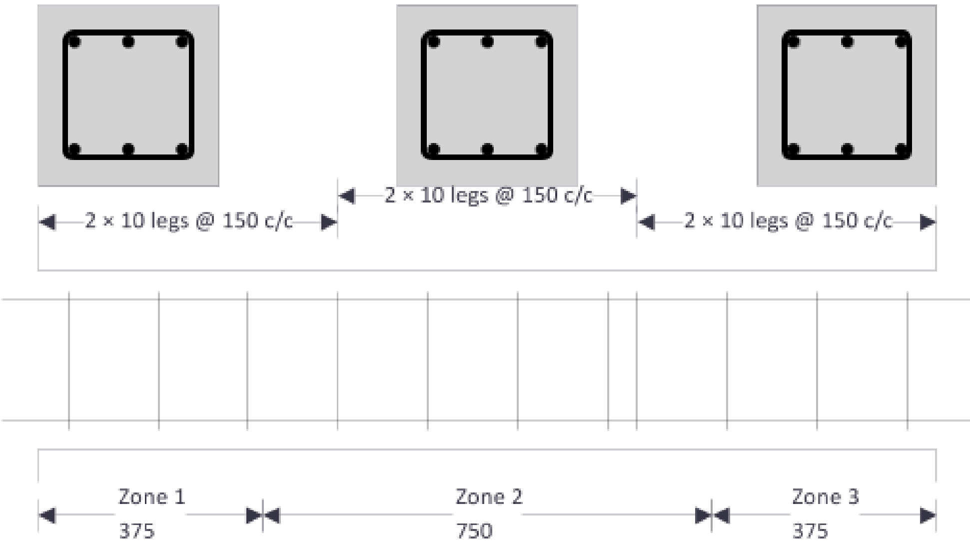
Bottom bar spacing $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_b\_L1} \times N_{m1\_s1\_z3\_b\_L1})) / (N_{m1\_s1\_z3\_b\_L1} - 1) =$ **70.0** mm

Minimum allowable bottom bar spacing $s_{bot,min} = \max(\phi_{m1\_s1\_z3\_b\_L1} \times k_{s1}, h_{agg} + k_{s2}, 20\text{mm}) =$ **25.0** mm

PASS - Actual bar spacing exceeds minimum allowable

Shear design





Angle of comp. shear strut for maximum shear — θ_{max} = 45 deg

Strength reduction factor - cl.6.2.3(3) — $v_1 = 0.6 \times (1 - f_{ck} / 250\ N/mm^2)$ = **0.492**

Compression chord coefficient - cl.6.2.3(3) — α_{cw} = **1.00**

Minimum area of shear reinforcement - exp.9.5N — $A_{sv,min} = 0.08\ N/mm^2 \times b \times (f_{ck} / 1\ N/mm^2)^{0.5} / f_{yk}$ = **322** mm²/m

Zone 1 (0 mm - 375 mm) shear - section 6.2

Design shear force at support — $V_{Ed,max} = max(abs(V_{z1\_max}), abs(V_{z1\_red\_max}))$ = **148** kN

Min lever arm in shear zone — z = **228** mm

Maximum design shear resistance - exp.6.9 — $V_{Rd,max} = \alpha_{cw} \times b \times z \times v_1 \times f_{cwd} / (cot(\theta_{max}) + tan(\theta_{max}))$ = **505** kN

PASS - Design shear force at support is less than maximum design shear resistance

Design shear force at 240mm from support — V_{Ed} = **85** kN

Design shear stress — $v_{Ed} = V_{Ed} / (b \times z)$ = **1.248** N/mm²

Angle of concrete compression strut - cl.6.2.3 — $\theta = min(max(0.5 \times Asin(min(2 \times v_{Ed} / (\alpha_{cw} \times f_{cwd} \times v_1),1)), 21.8\ deg), 45deg)$ = **21.8** deg

Area of shear reinforcement required - exp.6.8 — $A_{sv,des} = v_{Ed} \times b / (f_{yd} \times cot(\theta))$ = **345** mm²/m

Area of shear reinforcement required — $A_{sv,req} = max(A_{sv,min}, A_{sv,des})$ = **345** mm²/m

Shear reinforcement provided — 2 × 10 legs @ 150 c/c

Area of shear reinforcement provided — $A_{sv,prov}$ = **1047** mm²/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N — $s_{vl,max} = 0.75 \times d$ = **180** mm

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

Zone 2 (375 mm - 1125 mm) shear - section 6.2

Design shear force at support — $V_{Ed,max} = max(abs(V_{z2\_max}), abs(V_{z2\_red\_max}))$ = **145** kN

Min lever arm in shear zone — z = **228** mm

Maximum design shear resistance - exp.6.9 — $V_{Rd,max} = \alpha_{cw} \times b \times z \times v_1 \times f_{cwd} / (cot(\theta_{max}) + tan(\theta_{max}))$ = **505** kN

PASS - Design shear force at support is less than maximum design shear resistance

Design shear force within zone — V_{Ed} = **145** kN

Design shear stress — $v_{Ed} = V_{Ed} / (b \times z)$ = **2.123** N/mm²

Angle of concrete compression strut - cl.6.2.3 — $\theta = min(max(0.5 \times Asin(min(2 \times v_{Ed} / (\alpha_{cw} \times f_{cwd} \times v_1),1)), 21.8\ deg), 45deg)$ = **21.8** deg

Area of shear reinforcement required - exp.6.8 — $A_{sv,des} = v_{Ed} \times b / (f_{yd} \times cot(\theta))$ = **586** mm²/m

Area of shear reinforcement required $A_{sv,req} = \max(A_{sv,min}, A_{sv,des}) = \mathbf{586}$ mm^2/m

Shear reinforcement provided 2 × 10 legs @ 150 c/c

Area of shear reinforcement provided $A_{sv,prov} = \mathbf{1047}$ mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N $s_{vl,max} = 0.75 \times d = \mathbf{180}$ mm

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

Zone 3 (1125 mm - 1500 mm) shear - section 6.2

Design shear force at support $V_{Ed,max} = \max(\text{abs}(V_{z3\_max}), \text{abs}(V_{z3\_red\_max})) = \mathbf{243}$ kN

Min lever arm in shear zone $z = \mathbf{219}$ mm

Maximum design shear resistance - exp.6.9 $V_{Rd,max} = \alpha_{cw} \times b \times z \times v_1 \times f_{cwd} / (\cot(\theta_{max}) + \tan(\theta_{max})) = \mathbf{484}$ kN

PASS - Design shear force at support is less than maximum design shear resistance

Design shear force at 240mm from support $V_{Ed} = \mathbf{180}$ kN

Design shear stress $v_{Ed} = V_{Ed} / (b \times z) = \mathbf{2.749}$ N/mm^2

Angle of concrete compression strut - cl.6.2.3 $\theta = \min(\max(0.5 \times \text{Asin}(\min(2 \times v_{Ed} / (\alpha_{cw} \times f_{cwd} \times v_1),1)), 21.8 \text{ deg}), 45\text{deg}) = \mathbf{21.8}$ deg

Area of shear reinforcement required - exp.6.8 $A_{sv,des} = v_{Ed} \times b / (f_{yd} \times \cot(\theta)) = \mathbf{759}$ mm^2/m

Area of shear reinforcement required $A_{sv,req} = \max(A_{sv,min}, A_{sv,des}) = \mathbf{759}$ mm^2/m

Shear reinforcement provided 2 × 10 legs @ 150 c/c

Area of shear reinforcement provided $A_{sv,prov} = \mathbf{1047}$ mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

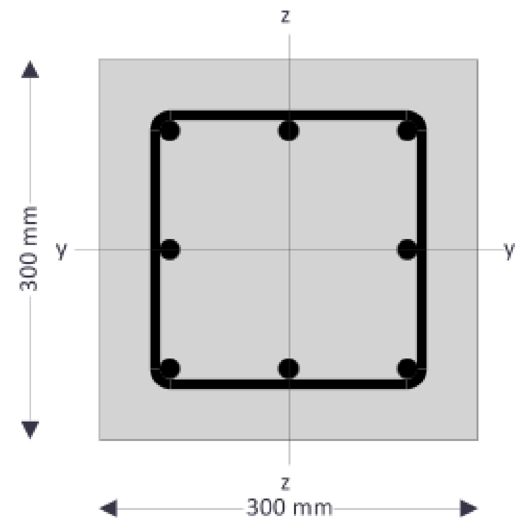
Maximum longitudinal spacing - exp.9.6N $s_{vl,max} = 0.75 \times d = \mathbf{180}$ mm

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

RC COLUMN DESIGN

In accordance with EN1992-1-1:2004 incorporating Corrigendum January 2008 and the UK national annex

Tedds calculation version 1.3.03



8 no. 16 mm diameter longitudinal bars
Max link spacing 300 mm generally, 180 mm for 300 mm above and below slab/beam and at laps

Column geometry

Overall depth (perp y)	h= **300** mm	Overall breadth (perp z)	b = **300** mm
Clear ht bet restr about y axis	l_y = **2400** mm	Clear ht bet restr about z axis	l_z = **2400** mm
Stability in the z direction	**Braced**	Stability in the y direction	**Braced**

Concrete details

Cylinder strength of concrete	f_{ck} = **45** MPa	Safety factor for concrete	γ_C = **1.50**
Coefficient α_{cc}	α_{cc} = **0.85**		
Maximum aggregate size	d_g = **20** mm		

Reinforcement details

Nominal cover to links	c_{nom} = **40** mm	Longitudinal bar diameter	ϕ = **16** mm
Link diameter	ϕ_v = **8** mm	Total no. of longitudinal bars	N = **8**
No.bars per face parallel y axis	N_y = **3**	No.bars per face parallel z axis	N_z = **3**
Area of longitudinal reinft	A_s = **1608** mm^2	Safety factor for reinforcement	γ_S = **1.15**
Modulus of elasticity of reinft	E_s = **200000** MPa		

Fire resistance details

Fire resistance period	R = **60** min	Exposure to fire	**More than one side**
Ratio of fire design axial load to design resistance		μ_{fi} = **0.70**	

Axial load and bending moments from frame analysis

Design axial load	N_{Ed} = **386.0** kN		
Moment about y axis at top	M_{topy} = **30.0** kNm	Moment about y axis at btm	M_{btmy} = **30.0** kNm
Moment about z axis at top	M_{topz} = **0.0** kNm	Moment about z axis at btm	M_{btmz} = **0.0** kNm

Column end restraints

End restraints buckling abt y **Braced, no rotational restraint both ends** End restraints buckling abt z **Braced, no rotational restraint both ends**

Check nominal cover for fire and bond requirements

Min cover to links for bond	$c_{min,b}$ = **8** mm	Min axis distance for fire	a_{fi} = **46** mm

Allowance for deviations Δc_{dev} = **10** mm   Min allowable nominal cover c_{nom_min} = **30.0** mm

PASS - the nominal cover is greater than the minimum required

Column slenderness

Slend. ratio buckling abt y λ_y = **27.7**   Slend. ratio buckling abt z λ_z = **27.7**

Slend. limit about y λ_{limy} = **30.3**   Slend. limit about z λ_{limz} = **30.3**

Design bending moments

Design moment about y axis M_{Edy} = **40.3** kNm   Design moment about z axis M_{Edz} = **10.3** kNm

Moment of resistances

Mt of resistance about y axis M_{Rdy} = **108.2** kNm   Mt of resistance about z axis M_{Rdz} = **108.2** kNm

PASS - The moment capacity about the y axis exceeds the design bending moment

PASS - The moment capacity about the z axis exceeds the design bending moment

ratio$_e$ > 0.2 - Biaxial bending check is required

Biaxial bending

Exponent a   a = **1.02**

Biaxial bending utilisation   UF = $(M_{Edy} / M_{Rdy})^a + (M_{Edz} / M_{Rdz})^a$ = **0.453**

PASS - The biaxial bending capacity is adequate

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CALCULATION SHEET

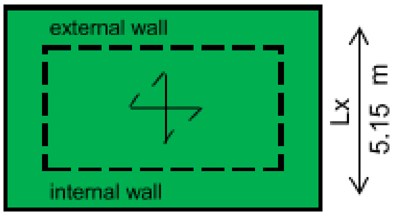
Project No: 18-3470	**Sheet No:**	**Rev:**
Project: Henthorn Road, Clitheroe.		
Calcs By: SWH		
Checked By: WG		**Date:** **Sep-18**

Base Slab Design (One way Spanning)

Design assuming Tank is full of water and also has traffic loading applied. Base will span between external wall and internal wall. Consider 1m width.

Loading (Also refer to Cover Slab Design & Floatation check.)

Lx = 5.15 m



Dead:- Ground Cover = 191 kN

Cover Slab = 80 kN

External Wall = 18 kN

Base Slab = 0 kN

Water if Full = 240 kN

t_{base} = 300 mm

Ground over Toe = 0 kN

50% Internal Wall = 9 kN

Total SLS Dead Load = 538 kN

Imposed:- SLS Traffic Load= 0 kN Surcharge = 5152 kN Total SLS Imposed Load = 5152 kN

Total SLS Load = 5690 kN

Total ULS Design Load = 726 kN
$(1.35G_k + 1.5Q_k)$

SLS Dead Pressure = 51 kN/m^2

SLS Imposed Pressure = 11 kN/m^2

Total SLS Pressure = 62.0 kN/m^2 ULS Pressure = 85.4 kN/m^2

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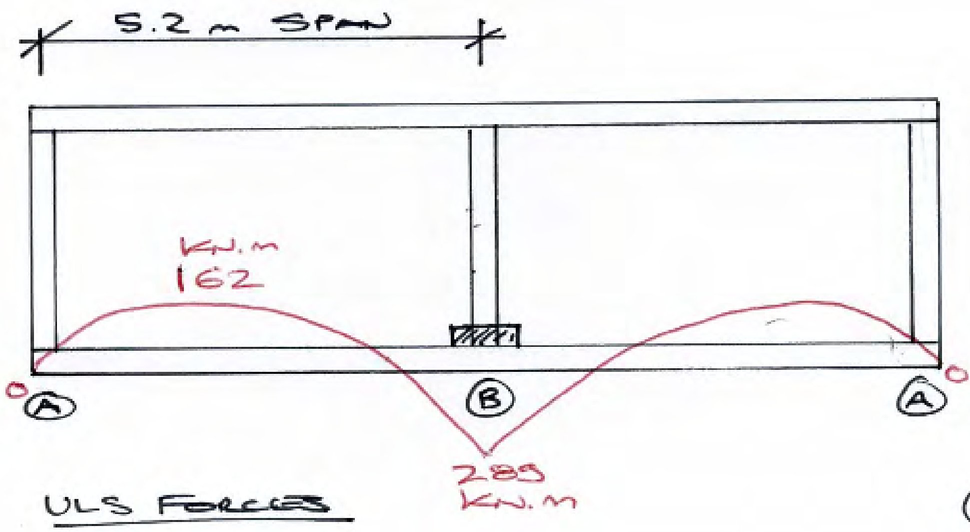
CALCULATION SHEET

Project No: 18-3470	Sheet No:	Rev:
Project: HAWTHORN RD, CLITHEROE.		
Calcs By: SWH		
Checked By:		Date: Sep-18



ULS FORCES

(ULS) kN.m

SPAN Bm = $0.070 \times 85.4 \times 5.2^2$ = 162

Bm (HOGGING) @ INT. WALL = $0.125 \times 85.4 \times 5.2^2$ = 289

REACTION @ A (ULS) = $0.375 \times 85.4 \times 5.2$ = 167 kN

REACTION @ B = $1.250 \times 85.4 \times 5.2$ = 555 kN

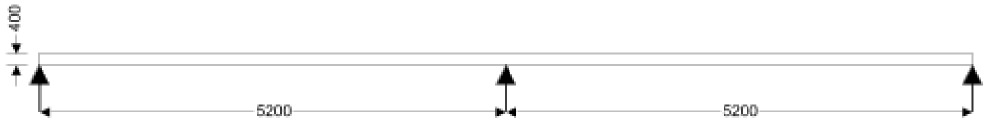
SHEAR AT FACE OF PRECAST WALL BASE = $\dfrac{2.6 - 0.45}{2.6} \times \dfrac{555}{2}$ = 230 kN

GHW Consulting Engineers Ltd 3 Emmanuel Court 20-22 Reddicroft Sutton Colfield, B73 6BN	Project Henthorn Road, Clitheroe.				Job no. 18-3470	
	Calcs for Base Slab				Start page no./Revision 1	
	Calcs by SWH	Calcs date 18/09/2018	Checked by WG	Checked date	Approved by	Approved date

RC SLAB DESIGN

In accordance with EN1992-1-1:2004 incorporating corrigendum January 2008 and the UK national annex

Tedds calculation version 1.0.17



Slab definition

Slab reference name	**Base**
Overall slab depth	h = **400** mm
Number of spans	N_{spans} = **2**
First support	**Simple**
Last support	**Simple**
Nominal cover to top reinforcement	c_{nom_t} = **40** mm
Nominal cover to bottom reinforcement	c_{nom_b} = **40** mm

Loading

Ratio of quasi-permanent to ultimate load	r_q = **0.300**

Concrete properties

Concrete strength class	C45/55
Characteristic cylinder strength	f_{ck} = **45** N/mm^2
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} = **25.5** N/mm^2
Mean axial tensile strength (Table 3.1)	$f_{ctm} = 0.30\ \text{N/mm}^2 \times (f_{ck} / 1\ \text{N/mm}^2)^{2/3}$ = **3.8** 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)	γ_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 top reinforcement	c_{nom_t} = **40** mm
Nominal cover to bottom reinforcement	c_{nom_b} = **40** mm
Fire resistance period to top of slab	R_{top} = **60** min
Fire resistance period to bottom of slab	R_{btm} = **60** min
Axis distance to top reinft (Table 5.8)	a_{fi_t} = **20** mm
Axis distance to bottom reinft (Table 5.8)	a_{fi_b} = **20** mm
Max bar diameter in top	ϕ_{max_t} = **25** mm
Max bar diameter in bottom	ϕ_{max_b} = **20** mm
Min. top cover requirement with regard to bond	$c_{min,b\_t} = \phi_{max\_t}$ = **25** mm
Min. btm cover requirement with regard to bond	$c_{min,b\_b} = \phi_{max\_b}$ = **20** mm
Reinforcement fabrication	**Not subject to QA system**
Cover allowance for deviation	Δc_{dev} = **10** mm
Min. required nominal cover to top reinft	$c_{nom\_t\_min} = \max(a_{fi\_t} - \phi_{max\_t} / 2, c_{min,b\_t} + \Delta c_{dev})$ = **35.0** mm
Min. required nominal cover to bottom reinft	$c_{nom\_b\_min} = \max(a_{fi\_b} - \phi_{max\_b} / 2, c_{min,b\_b} + \Delta c_{dev})$ = **30.0** mm

PASS - There is sufficient cover to the top reinforcement

PASS - There is sufficient cover to the bottom reinforcement

Bending design checks

Redistribution ratio δ = **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 = **5200** mm

Design bending moment M_{p1} = **162.0** kNm/m

Reinforcement provided **20 mm dia. bars at 125 mm centres**

Area provided A_{sp1} = **2513** mm^2/m

Effective depth to tension reinforcement $d_{p1} = h - c_{nom\_b} - \phi_{p1} / 2$ = **350.0** mm

K factor $K = M_{p1} / (b \times d_{p1}^2 \times f_{ck})$ = **0.029**

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 = **332.5** mm

Area of reinforcement required for bending $A_{sp1\_m} = M_{p1} / (f_{yd} \times z)$ = **1121** mm^2/m

Minimum area required $A_{sp1\_min} = \max(0.26 \times (f_{ctm}/f_{yk}), 0.0013) \times b \times d_{p1}$ = **691** mm^2/m

Area of reinforcement required $A_{sp1\_req} = \max(A_{sp1\_m}, A_{sp1\_min})$ = **1121** mm^2/m

PASS - Area of tension reinforcement provided is adequate (0.446)

Check reinforcement spacing

Reinforcement service stress $\sigma_s = (f_{yk} / \gamma_S) \times \min((A_{sp1\_m}/A_{sp1}), 1.0) \times r_q$ = **58.2** N/mm^2

Maximum allowable spacing (Table 7.3N) s_{max_p1} = **300** mm

Actual bar spacing s_{p1} = **125** mm

PASS - The reinforcement spacing is acceptable

Reinforcement design at midspan of span 2 (cl.6.1)

Length of span 2 l_2 = **5200** mm

Design bending moment M_{p2} = **162.0** kNm/m

Reinforcement provided **20 mm dia. bars at 125 mm centres**

Area provided A_{sp2} = **2513** mm^2/m

Effective depth to tension reinforcement $d_{p2} = h - c_{nom\_b} - \phi_{p2} / 2$ = **350.0** mm

K factor $K = M_{p2} / (b \times d_{p2}^2 \times f_{ck})$ = **0.029**

K < K' - Compression reinforcement is not required

Lever arm $z = \min(0.95 \times d_{p2}, d_{p2} / 2 \times (1 + \sqrt{(1 - 3.53 \times K)}))$

z = **332.5** mm

Area of reinforcement required for bending $A_{sp2\_m} = M_{p2} / (f_{yd} \times z)$ = **1121** mm^2/m

Minimum area required $A_{sp2\_min} = \max(0.26 \times (f_{ctm}/f_{yk}), 0.0013) \times b \times d_{p2}$ = **691** mm^2/m

Area of reinforcement required $A_{sp2\_req} = \max(A_{sp2\_m}, A_{sp2\_min})$ = **1121** mm^2/m

PASS - Area of tension reinforcement provided is adequate (0.446)

Check reinforcement spacing

Reinforcement service stress $\sigma_s = (f_{yk} / \gamma_S) \times \min((A_{sp2\_m}/A_{sp2}), 1.0) \times r_q$ = **58.2** N/mm^2

Maximum allowable spacing (Table 7.3N) s_{max_p2} = **300** mm

Actual bar spacing s_{p2} = **125** mm

PASS - The reinforcement spacing is acceptable

Reinforcement design at support 2 (cl.6.1)

Design bending moment M_{n2} = **289.0** kNm/m

Reinforcement provided — **25 mm dia. bars at 125 mm centres**

Area provided — A_{sn2} = **3927** mm^2/m

Effective depth to tension reinforcement — $d_{n2} = h - c_{nom\_t} - \phi_{n2} / 2$ = **347.5** mm

K factor — $K = M_{n2} / (b \times d_{n2}^2 \times f_{ck})$ = **0.053**

K < K' - Compression reinforcement is not required

Lever arm — $z = \min(0.95 \times d_{n2}, d_{n2} / 2 \times (1 + \sqrt{(1 - 3.53 \times K)}))$

z = **330.1** mm

Area of reinforcement required for bending — $A_{sn2\_m} = M_{n2} / (f_{yd} \times z)$ = **2013** mm^2/m

Minimum area required — $A_{sn2\_min} = \max(0.26 \times (f_{ctm}/f_{yk}), 0.0013) \times b \times d_{n2}$ = **686** mm^2/m

Area of reinforcement required — $A_{sn2\_req} = \max(A_{sn2\_m}, A_{sn2\_min})$ = **2013** mm^2/m

PASS - Area of tension reinforcement provided is adequate (0.513)

Check reinforcement spacing

Reinforcement service stress — $\sigma_s = (f_{yk} / \gamma_S) \times \min((A_{sn2\_m}/A_{sn2}), 1.0) \times r_q$ = **66.9** N/mm^2

Maximum allowable spacing (Table 7.3N) — s_{max_n2} = **300** mm

Actual bar spacing — s_{n2} = **125** mm

PASS - The reinforcement spacing is acceptable

Shear design checks

Shear resistance constant (cl. 6.2.2) — $C_{Rd,c}$ = 0.18 N/mm^2 / γ_C = **0.12** N/mm^2

Shear capacity check at support 1

Shear force — V_1 = **167.0** kN/m

Reinforcement provided — **20 mm dia. bars at 125 mm centres**

Area provided — A_{sd1} = **2513** mm^2/m

Effective depth — $d_{d1} = h - c_{nom\_b} - \phi_{d1} / 2$ = **350.0** mm

Effective depth factor (cl. 6.2.2) — $k = \min(2.0, 1 + (200 \text{ mm} / d_{d1})^{0.5})$ = **1.756**

Reinforcement ratio — $\rho_l = \min(0.02, A_{sd1} / (b \times d_{d1}))$ = **0.0072**

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} = **191.2** kN/m

Shear resistance (Exp. 6.2a) — $V_{Rd,c1} = \max(V_{Rd,c\_min}, C_{Rd,c} \times k \times (100 \times \rho_l \times (f_{ck}/1 \text{ N/mm}^2))^{0.333} \times b \times d_{d1})$

$V_{Rd,c1}$ = **234.6** kN/m

PASS - Shear capacity is adequate (0.712)

Shear capacity check at support 2

Shear force — V_2 = **230.0** kN/m

Effective depth factor (cl. 6.2.2) — $k = \min(2.0, 1 + (200 \text{ mm} / d_{n2})^{0.5})$ = **1.759**

Reinforcement ratio — $\rho_l = \min(0.02, A_{sn2} / (b \times d_{n2}))$ = **0.0113**

Minimum shear resistance (Exp. 6.3N) — $V_{Rd,c\_min} = 0.035 \text{ N/mm}^2 \times k^{1.5} \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} \times b \times d_{n2}$

V_{Rd,c_min} = **190.3** kN/m

Shear resistance (Exp. 6.2a) — $V_{Rd,c2} = \max(V_{Rd,c\_min}, C_{Rd,c} \times k \times (100 \times \rho_l \times (f_{ck}/1 \text{ N/mm}^2))^{0.333} \times b \times d_{n2})$

$V_{Rd,c2}$ = **271.3** kN/m

PASS - Shear capacity is adequate (0.848)

Shear capacity check at support 3

Shear force — V_3 = **167.0** kN/m

Reinforcement provided — **20 mm dia. bars at 125 mm centres**

Area provided — A_{sd3} = **2513** mm^2/m

Effective depth — $d_{d3} = h - c_{nom\_b} - \phi_{d3} / 2 = \mathbf{350.0}$ mm

Effective depth factor (cl. 6.2.2) — $k = \min(2.0, 1 + (200 \text{ mm} / d_{d3})^{0.5}) = \mathbf{1.756}$

Reinforcement ratio — $\rho_l = \min(0.02, A_{sd3} / (b \times d_{d3})) = \mathbf{0.0072}$

Minimum shear resistance (Exp. 6.3N) — $V_{Rd,c\_min} = 0.035 \text{ N/mm}^2 \times k^{1.5} \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} \times b \times d_{d3}$

$V_{Rd,c\_min} = \mathbf{191.2}$ kN/m

Shear resistance (Exp. 6.2a) — $V_{Rd,c3} = \max(V_{Rd,c\_min}, C_{Rd,c} \times k \times (100 \times \rho_l \times (f_{ck}/1 \text{ N/mm}^2))^{0.333} \times b \times d_{d3})$

$V_{Rd,c3} = \mathbf{234.6}$ kN/m

PASS - Shear capacity is adequate (0.712)

Deflection checks

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

Reference reinforcement ratio — $\rho_0 = (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / 1000 = \mathbf{0.0067}$

Required tension reinforcement ratio — $\rho = \max(0.0035, A_{sp1\_m} / (b \times d_{p1})) = \mathbf{0.0035}$

Required compression reinforcement ratio — $\rho' = A_{scp1\_req} / (b \times d_{p1}) = \mathbf{0.0000}$

Structural system factor (Table 7.4N) — $K_\delta = \mathbf{1.3}$

Basic span-to-depth ratio limit $ratio_{lim1\_bas} = K_\delta \times [11 + 1.5 \times (f_{ck}/1 \text{ N/mm}^2)^{0.5} \times \rho_0/\rho + 3.2 \times (f_{ck}/1 \text{ N/mm}^2)^{0.5} \times (\rho_0/\rho - 1)^{1.5}]$

(Exp. 7.16a) — $ratio_{lim1\_bas} = \mathbf{63.86}$

Modified span-to-depth ratio limit

$ratio_{lim1} = \min(40 \times K_\delta, \min(1.5, (500 \text{ N/mm}^2 / f_{yk}) \times (A_{sp1} / A_{sp1\_m})) \times ratio_{lim1\_bas}) = \mathbf{52.00}$

Actual span-to-depth ratio — $ratio_{act1} = l_1 / d_{p1} = \mathbf{14.86}$

PASS - Span-to-depth ratio is acceptable (0.286)

Basic span-to-depth ratio deflection check span 2 (cl. 7.4.2)

Reference reinforcement ratio — $\rho_0 = (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / 1000 = \mathbf{0.0067}$

Required tension reinforcement ratio — $\rho = \max(0.0035, A_{sp2\_m} / (b \times d_{p2})) = \mathbf{0.0035}$

Required compression reinforcement ratio — $\rho' = A_{scp2\_req} / (b \times d_{p2}) = \mathbf{0.0000}$

Structural system factor (Table 7.4N) — $K_\delta = \mathbf{1.3}$

Basic span-to-depth ratio limit $ratio_{lim2\_bas} = K_\delta \times [11 + 1.5 \times (f_{ck}/1 \text{ N/mm}^2)^{0.5} \times \rho_0/\rho + 3.2 \times (f_{ck}/1 \text{ N/mm}^2)^{0.5} \times (\rho_0/\rho - 1)^{1.5}]$

(Exp. 7.16a) — $ratio_{lim2\_bas} = \mathbf{63.86}$

Modified span-to-depth ratio limit

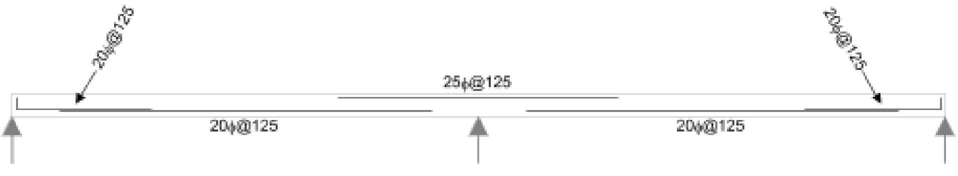
$ratio_{lim2} = \min(40 \times K_\delta, \min(1.5, (500 \text{ N/mm}^2 / f_{yk}) \times (A_{sp2} / A_{sp2\_m})) \times ratio_{lim2\_bas}) = \mathbf{52.00}$

Actual span-to-depth ratio — $ratio_{act2} = l_2 / d_{p2} = \mathbf{14.86}$

PASS - Span-to-depth ratio is acceptable (0.286)

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.



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CALCULATION SHEET

Project No: 18-3470	**Sheet No:**	**Rev:**
Project: Henthorn Road, Clitheroe.		
Calcs By: SWH		
Checked By: WG		**Date: Sep-18**

Base Bottom Reinforcement

ULS Bm from wall = 36.8 kNm

Assumed Outer Face Tension Bar Dia = 16 mm

d = 252 mm

$k = M/bd^2.f_{ck}$ = 0.011

L. A. factor z/d = [1-√1-3.53k] = 0.95

z = 239 mm

$A_s = M/0.87f_yz$ = 354 mm^2

USE H 16 @ 125 mm c/c in each face + H8 @ 150mm c/c distribution.

(A_{sprov} = 1608 mm^2)

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<u>Crack Width Summary</u>

Element	Thickness	h_D	W_{k1}	h_D/h	Actual Crack width	
Cover Slab	300	0	0.200	0	0.177	Pass
Walls	300	2400	0.185	8	0.150	Pass
Internal Wall	300	2400	0.185	8	1.153	Pass
Base	300	3600	0.165	12	0.160	Pass