Appendix B

Basement Design Calculations for the Permanent Structure

Contents

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Job No: 180709

Date	Version	Notes/Amendments/Purpose of issue

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Calculation for: 24-26 Redington Gardens

Job No. 180709

Date: Dec 18

Calculation by: KSC

Checked by:

Sheet No. LD

LOADING

Roof.	DL	=	1.50 Say	
	LL	=	0.75 KN/m²	0.75KN/m²
2nd	DL	=	9.5 KN/m²	
	LL	=	2.5 KN/m²	2.5 KN/m²
1st	DL	=	9.5 KN/m²	
	LL	=	2.5 KN/m²	2.5KN/m²
UGF	DL	=	9.5 KN/m²	
	LL	=	2.5 KN/m²	2.5KN/m²
LGF	DL	=	9.5 KN/m²	
	LL	=	2.5 KN/m²	2.5KN/m²
Basement	DL	=	14.4 KN/m²	
	LL	=	2.5 KN/m²	2.5KN/m²

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Calculation for: Redington

Job No. 180709

Date: Dec 18

Calculation by: KSC

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Sheet No. L1

Spine wall – FRONT

200 RC wall –	0.2×24×14 =	67.2	
300 capping	0.2×24×3	14.40	
L1–L4 slab	9.5×8.25×4	313.5	
L1–L4 Live	3.5×.0×8.25×4		115.5
Roof	1.25×8.25	10.31	6.18
Live	0.75×8.25		
Basement	9.6×4.25	40.8	
Live	3.5×4.25		15
		446.21	136.68

Total DL+LL = 582.89 =

Spine wall Rear

200 RC wall	0.2×24×7.1	34.08	
L1 & L2 Slab	9.5×8.25×2	157	
L1 – L2 Live	3.5×8.25×2		58
Basement	9.6×4.25	40.8	
Live	3.5×4.25		15
		231.88	73

Total DL+LL = 305 kN/m
Factored load on column = 422.5×4.5 = 1901 kN

Spine wall Podium

200 RC wall –	0.2×24×4	19.2	
L1 Slab	9.5×8.25	78.3	
L1 Live	2.5×8.25		20.6
L1 finishes	0.5×18×8.25	75.	
Basement	9.6×4.25	40.8	
Live	3.5×4.25		15
		213.3	35.6

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Calculation for:	Job No.
Redington Gdns	180709
	Date: Dec 18

Calculation by:	Checked by:	Sheet No.
KSC		L2

Perimeter Load.

Roof -	DL =	1.5 × 3 =	4.5	
	LL =	0.75 × 3		2.25
2nd	DL =	9.5 × 4 =	38	
	LL =	2.5 × 4 =		10.0
1st	DL =		38	
	LL			10
UG	DL		38	
	LL			10
LG	DL		38	
	LL			10
Base	DL =	14.4 × 4	57.6	
	LL			10
			214.1	52.5

Center Loads.

Roof	DL =	1.5 × 8	12	
	LL	0.75 × 8	~~6~~	6
2nd	DL	9.5 × 8	76	
	LL			20
1st	DL		76	
	LL			20
UG	DL		76	
	LL			20
LG	DL		76	
	LL			20
Base	DL		115.2	
				20
			431	106

Wall DL = 0.2 × 24 × 12.525 = 60 kN/m

@ LGF DL = 336 kN/m LL = 80 kN/m = 416 kN 573 kN/m

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Column Load on Spine wall (Excluding Basement Loading)

DL = 415.3 kN/m × 4.2 = 1746

LL = 86 kN/m × 4.2 = 361 kN

Wall = 0.2×24×12.5×4.2 = 252

DL = 1998 kN

LL = 361 kN

Factored Load = 3238 kN

Foundation Load.

DL = 431 × 4.2 = 1810 kN

LL = 106 × 4.2 = 445 kN

Wall = 0.2×24×17.5×4.2 = 352

DL = 2168 kN

LL = 445 kN

2607 kN

Pile Load = 651 kN

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Calculation for:		Job No. 180709
Redington Gardens		Date: Dec 18
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Rear Section Central Loading

Loading

		DL	DL	LL
Lower GF				
Slab DL		9.6 × 6	57.6	
Soil		24.3 × 6	145.8	
Live		2.5 × 6		15
Basement Pool				
Slab		12 × 3	36	
Pool slab		4.8 × 3	14.4	
Water		20 × 3	60	
Live		1.5 × 3		4.5
			313.8	19.5	kN/m

Pile Load

DL = 313.8 × 5 = 1569 kN
LL = 19.5 × 5 = 97.5 kN
1665 kN

∴ Load/pile = 555 kN

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Calculation for: 24-26 Redington Gdns.		Job No. 180709
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Rear Section perimeter

LGF	Slab	9.6 x 3	=	28.8	
	Soil	24.3 x 3	=	72.9	
	Live	2.5 x 3			7.5
Basement	Slab	12 x 1.5		18	
		4.8 x 1.5		7.2	
		20 x 1.5		30	
	Live	1.5 x 1.5			2.25
				156.9	9.75
Wall				78.0	
				235kN	9.75kN

Load on piled wall

DL = 235 x 0.65 = 152kN

LL = 9.75 x 0.65 = 6.0 kN/m

Tension Load = 70 x 1.5 x 0.65 = 68.25kN (Tension)

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Calculation for: 24-26 Redington Gardens

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Sheet No. L6

Load on Col on GL D

Roof DL $1.5 \times 2.5 \times 8 =$ 30
LL $0.75 \times 2.5 \times 8 =$ 15

2nd $9.5 \times 2.75 \times 8 =$ 209
LL $2.5 \times 2.75 \times 8 =$ 55

1st $9.5 \times 2.75 \times 8 =$ 209
LL $2.5 \times 2.75 \times 8 =$ 55

UGF $9.5 \times 5 \times 8 =$ 380
LL $2.5 \times 5 \times 8 =$ 100

UGF $9.5 \times 5 \times 8 =$ 380
$2.5 \times 5 \times 8$ 100

LGF $14.4 \times 5 \times 8 =$ 576
$2.5 \times 5 \times 8$ 100

Wall $0.2 \times 24 \times 5 \times 9 =$ 216
$0.2 \times 24 \times 2.5 \times 9 =$ 108

2108 425

RC wall: 240

Additional load on Basement:

B/B floor = $4 \times 1.5 \times 8 =$ 48 kN
Pool $27 \times 2 \times \frac{1}{5} \times 8 =$ 86 kN

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DL = 2[illegible] kN

LL = 425 kN

LC 485

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Calculation for: 24-26 Redrington Gardens	Job No. 180709
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Gable wall

Wall	0.35 × 24 × 10.5	88.2	
L1-4	9.5 × 4.1 × 4	155.8	
Live	3.5 × 4.1 × 4		57.4
Roof	1.25 × 2.05	5.06	
	0.75 × 2.05		3.03
		250	60

DL + LL = 310 kN/m

Colm React = 310 × 8 = 2480 kN

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Calculation for:		Job No. 180709
24-26 Redington Gardens		Date: Dec 18
Calculation by: KSC	Checked by:	Sheet No. L8

Rear wall on GL D

Cavity wall -	$0.2 \times 24 \times 6.6 \times \frac{15.95}{2}$ = [illegible]		
	$\times 0.75$	175 kN	
Roof	$1.25 \times 2 \times \frac{15.95}{2}$ =	20	
Live	$0.75 \times 2 \times \frac{15.95}{2}$ =		12
		195	12

Total Load = 207 kN

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Calculation for:	Job No.
PILE LOADS 25 – 26 REDINGTON GARDENS	180709
	Date: JAN '19
Calculation by: JE	Checked by: KSC	Sheet No. 49

P19, P20 + P21 → PILE GROUP

LOWER G.F.		D.L. [kN/m]	I.L. [kN/m]
SLAB D.L.	9.6 × 6	57.6	
SOIL	24.3 × 6	145.8	
LIVE	2.5 × 6		15

BASEMENT			
SLAB D.L.	12 × 3	36	
POOL SLAB	4.8 × 3	14.4	
WATER	20 × 3	60	
LIVE	1.5 × 3		4.5
		313.8	19.5

PILE LOAD

D.L. 313.8 × 5 = 1569 ∴ 550 PER PILE

I.L. 19.5 × 5 = 97.5 ∴ 50 PER PILE

TENSION

45 × 15 m² = 675 ∴ 225 kN PER PILE

P6 – BASEMENT

SLAB D.L. 14.4 × 14 ∴ 250 kN

LIVE 2.5 × 14 ∴ 50 kN

TENSION 60 × 14 ∴ 840 kN

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Calculation for:		Job No. 180709
25-26 REDINGTON GARDENS		Date: JAN '19
Calculation by: JE	Checked by: KSC	Sheet No. L10

P1

BASEMENT		D.L. [kN/m]	I.L. [kN/m]
SLAB D.L.	12 x 13	156	
POOL SLAB	4.8 x 13	62.4	
WATER	20 x 13	260	
LIVE	1.5 x 13		19.5
TOTAL		478.4	19.5

DL ∴ 500 kN ; IL = 50 kN

TENSION : 60 x 13 ∴ 780 kN

P40

BASEMENT		D.L. [kN/m]	I.L. [kN/m]
SLAB D.L.	12 x 18	216	
LIVE	2.5 x 18		45

∴ D.L. = 250 kN ; I.L = 50 kN

TENSION : 60 x 18 ∴ 1080 kN

PILE GROUP (P28, P29, P30 + P31)

LOWER GF / + UPPER FLOORS

SLAB D.L.	(9.6 x 8) x 4 + 1.5	308.7	
LIVE	(2.5 x 8) x 4 + 0.75		80.7

BASEMENT

SLAB D.L.	12 x 4.5	54	
LIVE	2.5 x 4.5		11.25

PILE LOAD PER PILE ∴ D.L. 362.7 x 5 / 4 = 450 kN
∴ I.L. 91.95 x 5 / 4 = 125 kN

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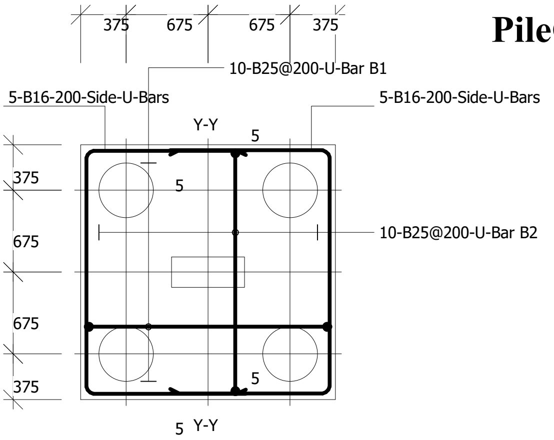
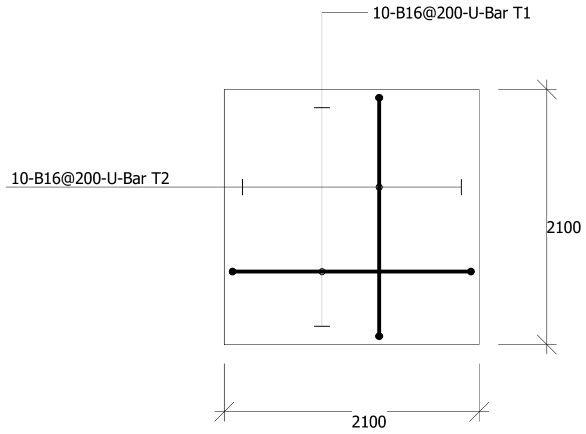
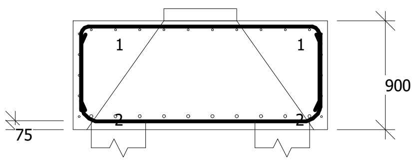
MasterRC - Pile Cap Design
PileCap Type 1







Summary of Design Data

Design to	EC 2: 2004 - Using UK values
Working vertical load per pile	750kN
Pile dia., cap o/a depth, overhang:	450mm, 900mm, 150mm
Pile spacing, centre to centre:	3.00 • dia. (1350mm)
Column size:	600mm. (x-x) x 250mm. (y-y)
Concrete grade:	C32/40
Concrete cover to reinforcement:	Bottom: 75mm., sides and top: 50mm.
Overall plan dimensions:	Width (x-x): 2100mm. Length (y-y): 2100mm.
Column Offsets:	ex = 0 mm. ey = 0 mm.

Forces

Axial Loads (kN)	Dead 1900, Imposed 950
Moments Mxx (kN.m)	Dead 0, Imposed 0 +ve causes compression on top
Moments Myy (kN.m)	Dead 0, Imposed 0 +ve causes compression on left

Calculations

No. of piles	2944/750	4	
Pile loads - Axial	Uniformly distributed from Column Axial Load		
Load per pile (service)	(1900 + 950 + 0 + 95)/4	736 kN	OK
Load per pile (Ultimate)	((1900+95.3)•1.35 + 950•1.50 + 0•1.05)/4 = 4120/4	1030 kN	
Load per pile (Ultimate Net)	(1900•1.35 + 950•1.50 + 0•1.05)/4 = 3992/4	998 kN	

Pile Cap Design Moments

$M_{xx}2Fp(L-b/2)$	2•1030.0(0.675 - 0.250/2)	1133.0 kN.m
$M_{xx}sw = 1.35•24•B•D•la^2/2$	$1.35•24•2.100•0.900•0.925^2/2$	26.0 kN.m
$M_{xx}res= M_{xx}- M_{xx}sw$	1133.0 - 26.0	1107.0 kN.m
$M_{yy}= 2Fp(L-a/2)$	2•1030.0(0.675 - 0.600/2)	772.0 kN.m
$M_{yy}sw = 1.35•24•B•D•la^2/2$	$1.35•24•2.100•0.900•0.925^2/2$	26.0 kN.m
$M_{yy}res= M_{yy}- M_{yy}sw$	772.0 - 26.0	746.0 kN.m

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Y-Y Axis: Layer B1

Reinf% = 100As/BD	As = 10B25@200 = 4910 => 100•4910/(2100 • 900)	0.26%	OK
Strut & Tie Analogy			
Truss Ften = 2•Fult/(24•ld)•($3l^2$-a^2)	2 • 4120/(24 • 1350 • 813) • (3 • 1350^2 - 600^2)	1599 kN	
As = F_{ten}/(0.87•f_y)	1598.7/(0.87 • 500)	3675 mm²	OK
Beam Bending Theory			
Asreq = fn(M, B ,d, f_{cd}, fy)	746, 2100, 813, 18, 500	2144 mm²	
Deep Beam Bending Theory			
Ref:	Reynolds & Steedman 10th edition. Table 148		
a1/h <0.23	150/900 = 0.17 < 0.23	Caution	
As =1.9•M/(f_y•min(L,h))	1.9 • 746 / (500 • min(1350,900))	3150 mm²	
Deep Beam Shear Theory			
V_{1c}= K_1•(h-0.35•a_1)•f_t•b	0.70 • (900 - 0.35 • 150) •3.16 • 2100.00	3937 kN	
V_1= V_{1C}+ K_2• A_{sprov}• d • $\sin^2(\phi)$/h	3937 + 225 • 4910 • 813 • Sin²(80.5) / 900	4907 kN	
V_{cap}= V_1+ K_2• A_{sv}•$\sin^2(\phi)$/2	4907 + 225 • 2011 • Sin²(80.5) / 2	5127 kN	
Beam Shear on Y-Y plane			
	Fv = 1030 x 2 piles	2060 kN	OK
Reduced V_{app}= V_{app}•$\beta_{enhance}$	2060.0 • 0.250	515.0	6.2.2.6
V_{app}/ max(V_{Rd},c.a, V_{Rd},c.b)	515.0 / Max(641.9, 617.9)	0.802	OK
V_{Rd},max = 0.5 • B_w• d • v • f_{cd}	0.5•2100•812.0•0.523•18	8088.9 kN	OK

X-X Axis: Layer B2

Reinf% = 100As/BD	As = 10B25@200 = 4910 => 100•4910/(2100 • 900)	0.26%	OK
Strut & Tie Analogy			
F_{ten} = F_{ult}•2/(24•ld)•($3l^2$-b^2)	2 • 4120/(24 • 1350 • 788) • (3 • 1350^2 - 250^2)	1746 kN	
As = Ften/(0.87•fy)	1745.5 / (0.87 • 500)	4013 mm²	
Beam Bending Theory			
Asreq = fn(M, B ,d, f_{cd}, fy)	1107, 2100, 788, 18, 500	3313 mm²	
Deep Beam Bending Theory			
Ref:	Reynolds & Steedman 10th edition. Table 148		
As =1.9•M/(f_y•min(L,h))	1.9 • 1107 / (500 • min(1350,900))	4674 mm²	OK
Deep Beam Shear Theory			
V_{1c}= K_1•(h-0.35•a_1)•f_t•b	0.70 • (900 - 0.35 • 325) •3.16 • 2100.00	3652 kN	
V_1= V_{1C}+ K_2• A_{sprov}• d • $\sin^2(\phi)$/h	3652 + 225 • 4910 • 788 • Sin²(70.1) / 900	4507 kN	
V_{cap}= V_1+ K_2• A_{sv}•$\sin^2(\phi)$/2	4507 + 225 • 2011 • Sin²(70.1) / 2	4708 kN	
Beam Shear on X-X plane			
	Fv = 1030 x 2 piles	2060 kN	OK
Reduced V_{app}= V_{app}•$\beta_{enhance}$	2060.0 • 0.263	542.449	6.2.2.6
V_{app}/ max(V_{Rd},c.a, V_{Rd},c.b)	542.4 / Max(632.3, 604.2)	0.858	OK
V_{Rd},max = 0.5 • B_w• d • v • f_{cd}	0.5•2100•788.0•0.523•18	7849.8 kN	OK
Punching Shear			
Column Head	Fv = 1030 x 4 piles - (1.35 x 95.26) selfweight	3991 kN	
V_{Rd},max = 0.5 • B_w• d • v • f_{cd}	0.5•1700•812.0•0.523•18	6548.2 kN	OK
Shear perimeter at 2 d_{eff}	Perimeter Crosses Piles. Perimeter limit to 1/5 int piles		
	Fv = 1030 x 4 piles	4120 kN	
Reduced V_{app}= V_{app}•$\beta_{enhance}$	4120.0 • 0.259	1068.625	6.2.2.6
V_{app}/ max(V_{Rd},c.a, V_{Rd},c.b)	1068.6 / Max(1268.9, 1257.1)	0.842	OK
Truss Concrete Compression			
Fc, Vertical Compression Check	Using maximum ultimate pile load	1030.0 kN	
Fcap = pi•dia²/4 •0.85•v'•f_{cd}	159043•0.85•0.52•18.1	1282.6 kN	OK
Anti-Crack Steel			
Top Nominal Steel	16@ 200 mm = 1005 mm²/m	0.112%	
Spacing [3.12.5.4]	16² • 500 / Min(900,500)	256	OK
Side Steel	16@ 200 mm = 1005 mm²/m	0.048%	
Spacing [3.12.5.4]	16² • 500 / Min(2100,500)	256	OK
Friction Pile Spacing			
Circular piles Spacing >= 3ϕ	3•450 = 1350 mm	actual = 1350 mm	OK
	see BS 8004 Clause 7.3.4.2		

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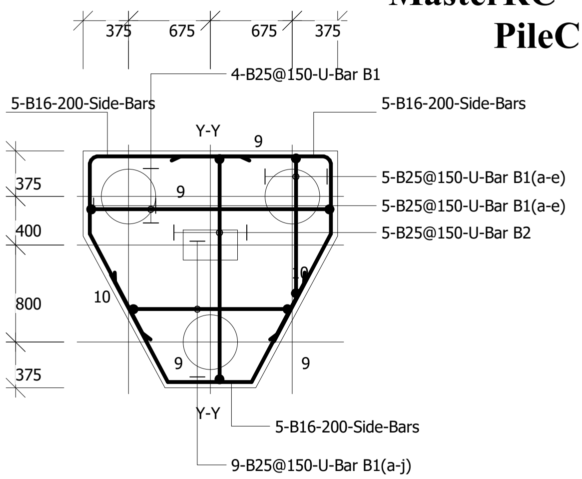
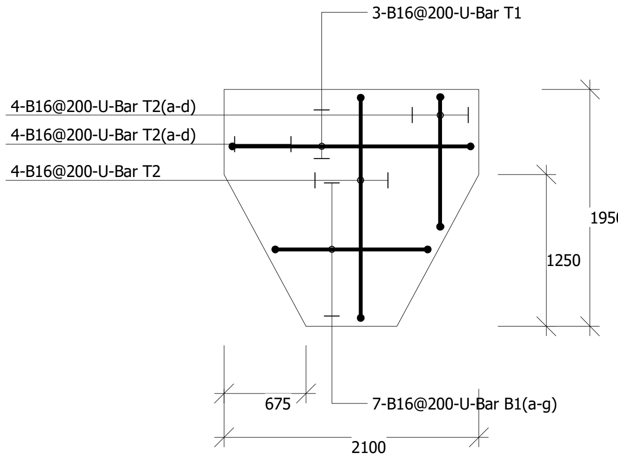
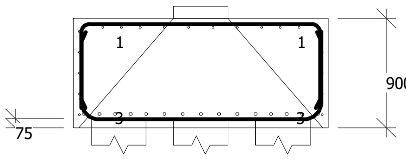
MasterRC - Pile Cap Design
PileCap Type 2







Summary of Design Data

Design to	EC 2: 2004 - Using UK values
Working vertical load per pile	750kN
Pile dia., cap o/a depth, overhang:	450mm, 900mm, 150mm
Pile spacing, centre to centre:	3.00 • dia. (1350mm)
Column size:	450mm. (x-x) x 250mm. (y-y)
Concrete grade:	C32/40
Concrete cover to reinforcement:	Bottom: 75mm., sides and top: 50mm.
Overall plan dimensions:	Width (x-x): 2100mm. Length (y-y): 1950mm.
Column Offsets:	ex = 0 mm. ey = 0 mm.

Forces

Axial Loads (kN)	Dead 1570, Imposed 150
Moments Mxx (kN.m)	Dead 0, Imposed 0 +ve causes compression on top
Moments Myy (kN.m)	Dead 0, Imposed 0 +ve causes compression on left

Calculations

No. of piles	1791/750	3	
Pile loads - Axial	Uniformly distributed from Column Axial Load		
Load per pile (service)	(1570 + 150 + 0 + 70)/3	597 kN	OK
Load per pile (Ultimate)	((1570+70.2)•1.35 + 150•1.50 + 0•1.05)/3 = 2439/3	813 kN	
Load per pile (Ultimate Net)	(1570•1.35 + 150•1.50 + 0•1.05)/3 = 2346/3	782 kN	

Pile Cap Design Moments

M_{xx}= Fp(L-b/2)	813.0(0.779 - 0.250/2)	532.0 kN.m
M_{xx}sw = 1.35•24•B•D•la²/2	1.35•24•0.750•0.900•1.029²/2	12.0 kN.m
M_{xx}res= M_{xx}- M_{xx}sw	532.0 - 12.0	520.0 kN.m
M_{yy}= Fp(L-a/2)	813.0(0.675 - 0.450/2)	366.0 kN.m
M_{yy}sw = 1.35•24•B•D•la²/2	1.35•24•0.750•0.900•0.925²/2	9.0 kN.m
M_{yy}res= M_{yy}- M_{yy}sw	366.0 - 9.0	357.0 kN.m

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Date : 04 January 2019 / Ver. 2018.08
Checked : KSC
Approved :

Y-Y Axis: Layer B1

Reinf% = 100As/BD	As = 4B25@150 = 1964 => 100•1964/(700 • 900)	0.31%	OK

Strut & Tie Analogy

$F_{ten} = F_{ult}/(36•ld)•(4l^2+b^2-3a^2)$	$2439/(36 • 1350 • 813) • (4 • 1350^2 + 250^2 - 3 • 450^2)$	417 kN	
$As = F_{ten}/(0.87•f_y)$	416.6/(0.87 • 500)	958 mm²	

Beam Bending Theory

$Asreq = fn(M, B ,d, f_{cd}, fy)$	357, 700, 813, 18, 500	1033 mm²	

Deep Beam Bending Theory

Ref:	Reynolds & Steedman 10th edition. Table 148		
$As = 1.9•M/(f_y•min(L,h))$	1.9 • 357 / (500 • min(1350,900))	1507 mm²	OK

Deep Beam Shear Theory

$V_{1c} = K_1•(h-0.35•a_1)•f_t•b$	0.70 • (900 - 0.35 • 225) •3.16 • 700.00	1272 kN	
$V_1 = V_{1C} + K_2• A_{sprov}• d • \sin^2(\phi)/h$	1272 + 225 • 1964 • 813 • Sin²(76.0) / 900	1647 kN	
$V_{cap} = V_1 + K_2• A_{sv}•\sin^2(\phi)/2$	1647 + 225 • 2011 • Sin²(76.0) / 2	1860 kN	

Beam Shear on Y-Y plane

	Fv = 813 x 1 piles	813 kN	OK
Reduced $V_{app} = V_{app}•\beta_{enhance}$	813.0 • 0.285	231.898	6.2.2.6
$V_{app}/ \max(V_{Rd},c.a, V_{Rd},c.b)$	231.9 / Max(345.4, 338.1)	0.671	OK
$V_{Rd},max = 0.5 • B_w• d • v • f_{cd}$	0.5•1162•800.0•0.523•18	4409.7 kN	OK

X-X Axis: Layer B2

Reinf% = 100As/BD	As = 5B25@150 = 2455 => 100•2455/(750 • 900)	0.36%	OK

Strut & Tie Analogy

$F_{ten} = F_{ult}/(18•ld)•(2l^2-b^2)$	$2439/(18 • 1350 • 788) • (2 • 1350^2 - 250^2)$	457 kN	
As = Ften/(0.87•fy)	456.6 / (0.87 • 500)	1050 mm²	

Beam Bending Theory

$Asreq = fn(M, B ,d, f_{cd}, fy)$	520, 750, 788, 18, 500	1569 mm²	

Deep Beam Bending Theory

Ref:	Reynolds & Steedman 10th edition. Table 148		
$As = 1.9•M/(f_y•min(L,h))$	1.9 • 520 / (500 • min(1558,900))	2196 mm²	OK

Deep Beam Shear Theory

$V_{1c} = K_1•(h-0.35•a_1)•f_t•b$	0.70 • (900 - 0.35 • 429) •3.16 • 750.00	1244 kN	
$V_1 = V_{1C} + K_2• A_{sprov}• d • \sin^2(\phi)/h$	1244 + 225 • 2455 • 788 • Sin²(64.5) / 900	1638 kN	
$V_{cap} = V_1 + K_2• A_{sv}•\sin^2(\phi)/2$	1638 + 225 • 2011 • Sin²(64.5) / 2	1822 kN	

Beam Shear on X-X plane

	Fv = 813 x 1 piles	813 kN	OK
Reduced $V_{app} = V_{app}•\beta_{enhance}$	813.0 • 0.343	278.566	6.2.2.6
$V_{app}/ \max(V_{Rd},c.a, V_{Rd},c.b)$	278.6 / Max(303.6, 374.3)	0.744	OK
$V_{Rd},max = 0.5 • B_w• d • v • f_{cd}$	0.5•1301•788.0•0.523•18	4863.2 kN	OK

Punching Shear

Column Head	Fv = 813 x 3 piles - (1.35 x 70.23) selfweight	2344 kN	
$V_{Rd},max = 0.5 • B_w• d • v • f_{cd}$	0.5•1400•812.0•0.523•18	5392.6 kN	OK

Truss Concrete Compression

Fc, Vertical Compression Check	Using maximum ultimate pile load	813.0 kN	
$Fcap = pi•dia^2/4 •0.85•v'•f_{cd}$	159043•0.85•0.52•18.1	1282.6 kN	OK

Anti-Crack Steel

Top Nominal Steel	16@ 200 mm = 1005 mm²/m	0.112%	
Spacing [3.12.5.4]	16² • 500 / Min(900,500)	256	OK
Side Steel	16@ 200 mm = 1005 mm²/m	0.052%	
Spacing [3.12.5.4]	16² • 500 / Min(1950,500)	256	OK

Friction Pile Spacing

Circular piles Spacing >= 3ϕ	3•450 = 1350 mm	actual = 1350 mm	OK
	see BS 8004 Clause 7.3.4.2		

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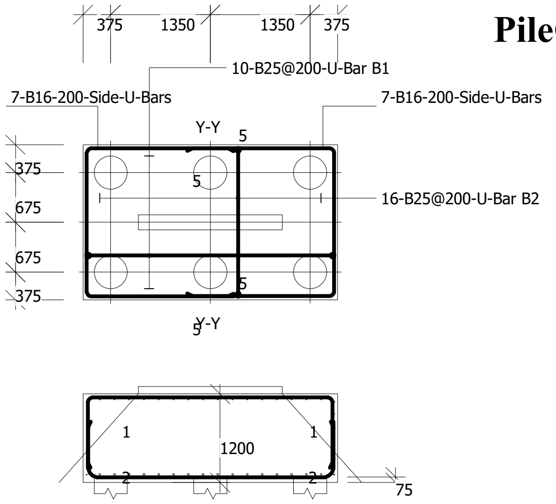
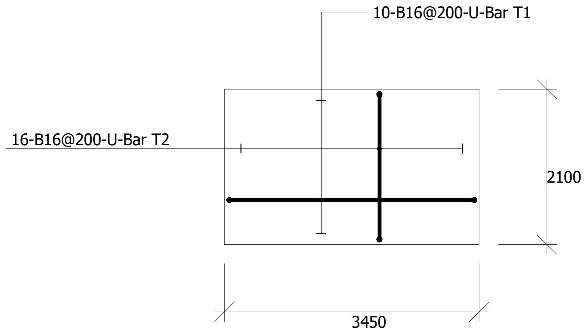
25676

Job Ref	**: 180709**
Sheet	**: /PC05**
Made by	**: JE**
Date	**: 04 January 2019 / Ver. 2018.08**
Checked	**: KSC**
Approved	**:**

MasterRC - Pile Cap Design
PileCap Type 3





Summary of Design Data

Design to	EC 2: 2004 - Using UK values
Working vertical load per pile	750kN
Pile dia., cap o/a depth, overhang:	450mm, 1200mm, 150mm
Pile spacing, centre to centre:	3.00 • dia. (1350mm)
Column size:	1950mm. (x-x) x 200mm. (y-y)
Concrete grade:	C32/40
Concrete cover to reinforcement:	Bottom: 75mm., sides and top: 50mm.
Overall plan dimensions:	Width (x-x): 3450mm. Length (y-y): 2100mm.
Column Offsets:	ex = 0 mm. ey = 0 mm.

Forces

Axial Loads (kN)	Dead 2580, Imposed 625
Moments Mxx (kN.m)	Dead 0, Imposed 0 +ve causes compression on top
Moments Myy (kN.m)	Dead 0, Imposed 0 +ve causes compression on left

Calculations

No. of piles	3414/750 = 5 (Overridden by preferred minimum!)	6	
Pile loads - Axial	Uniformly distributed from Column Axial Load		
Load per pile (service)	(2580 + 625 + 0 + 209)/6	569 kN	OK
Load per pile (Ultimate)	((2580+208.7)•1.35 + 625•1.50 + 0•1.05)/6 = 4704/6	784 kN	
Load per pile (Ultimate Net)	(2580•1.35 + 625•1.50 + 0•1.05)/6 = 4422/6	737 kN	

Pile Cap Design Moments

M_{xx}= 3Fp(L-b/2)	3•784.0(0.675 - 0.200/2)	1352.0 kN.m	
M_{xx}sw = 1.35•24•B•D•la²/2	1.35•24•3.450•1.200•0.950²/2	61.0 kN.m	
M_{xx}res= M_{xx}- M_{xx}sw	1352.0 - 61.0	1291.0 kN.m	
M_{yy}= 2Fp(L-a/2)	2•784.0(1.350 - 1.950/2)	588.0 kN.m	
M_{yy}sw = 1.35•24•B•D•la²/2	1.35•24•2.100•1.200•1.625²/2	108.0 kN.m	
M_{yy}res= M_{yy}- M_{yy}sw	588.0 - 108.0	480.0 kN.m	

Y-Y Axis: Layer B1

Reinf% = 100As/BD	As = 10B25@200 = 4910 => 100•4910/(2100 • 1200)	0.19%	OK

Strut & Tie Analogy

F_{ten} = F_{ult}•l/(3•d)	4704 • 1350 / (3 • 1113)	1903 kN	
As = F_{ten}/(0.87•f_y)	1902.7/(0.87 • 500)	4374 mm²	OK

Beam Bending Theory

Asreq = fn(M, B ,d, f_{cd}, fy)	480, 2100, 1113, 18, 500	997 mm²	

Beam Shear on Y-Y plane

	Fv = 784 x 2 piles	1568 kN	
Reduced V_{app}= V_{app}•$\beta_{enhance}$	1568.0 • 0.250	392.0	6.2.2.6

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Made by	**: JE**
Date	**: 04 January 2019 / Ver. 2018.08**
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Approved	**:**

V_{app}/ max(V_{Rd},c.a, V_{Rd},c.b)	392.0 / Max(753.4, 785.7)	0.499	OK
V_{Rd},max = 0.5 • B_w• d • v • f_{cd}	0.5•2100•1112.0•0.523•18	11077.4 kN	OK

X-X Axis: Layer B2

Reinf% = 100As/BD	As = 16B25@200 = 7856 => 100•7856/(3450 • 1200)	0.19%	OK

Strut & Tie Analogy

F_{ten} = F_{ult}•l/(4•d)	4704 • 1350 / (4 • 1088)	1460 kN	
As = Ften/(0.87•fy)	1459.9 / (0.87 • 500)	3356 mm²	

Beam Bending Theory

Asreq = fn(M, B ,d, f_{cd}, fy)	1291, 3450, 1088, 18, 500	2755 mm²	

Deep Beam Bending Theory

Ref:	Reynolds & Steedman 10th edition. Table 148		
As =1.9•M/(f_y•min(L,h))	1.9 • 1291 / (500 • min(1350,1200))	4088 mm²	OK

Deep Beam Shear Theory

V_{1c}= K_1•(h-0.35•a_1)•f_t•b	0.70 • (1200 - 0.35 • 350) •3.16 • 3450.00	8223 kN	
V_1= V_{1C}+ K_2• A_{sprov}• d • sin²(ϕ)/h	8223 + 225 • 7856 • 1088 • Sin²(73.7) / 1200	9699 kN	
V_{cap}= V_1+ K_2• A_{sv}•sin²(ϕ)/2	9699 + 225 • 2815 • Sin²(73.7) / 2	9991 kN	

Beam Shear on X-X plane

	Fv = 784 x 3 piles	2352 kN	OK
Reduced V_{app}= V_{app}•$\beta_{enhance}$	2352.0 • 0.250	588.0	6.2.2.6
V_{app}/ max(V_{Rd},c.a, V_{Rd},c.b)	588.0 / Max(1213.1, 1269.2)	0.463	OK
V_{Rd},max = 0.5 • B_w• d • v • f_{cd}	0.5•3450•1088.0•0.523•18	17805.9 kN	OK

Punching Shear

Column Head	Fv = 784 x 6 piles - (1.35 x 208.66) selfweight	4422 kN	
V_{Rd},max = 0.5 • B_w• d • v • f_{cd}	0.5•4300•1112.0•0.523•18	22682.4 kN	OK
Shear perimeter at 2 d_{eff}	Perimeter Crosses Piles. Perimeter limit to 1/5 int piles		
	Fv = 784 x 6 piles	4704 kN	
Reduced V_{app}= V_{app}•$\beta_{enhance}$	4704.0 • 0.250	1176.0	6.2.2.6
V_{app}/ max(V_{Rd},c.a, V_{Rd},c.b)	1176.0 / Max(2263.0, 2604.6)	0.452	OK

Truss Concrete Compression

Fc, Vertical Compression Check	Using maximum ultimate pile load	784.0 kN	
Fcap = pi•dia²/4 •0.85•v'•f_{cd}	159043•0.85•0.52•18.1	1282.6 kN	OK

Anti-Crack Steel

Top Nominal Steel	16@ 200 mm = 1005 mm²/m	0.084%	
Spacing [3.12.5.4]	16² • 500 / Min(1200,500)	256	OK
Side Steel	16@ 200 mm = 1005 mm²/m	0.048%	
Spacing [3.12.5.4]	16² • 500 / Min(2100,500)	256	OK

Friction Pile Spacing

Circular piles Spacing >= 3ϕ	3•450 = 1350 mm	actual = 1350 mm	OK
	see BS 8004 Clause 7.3.4.2		

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Calculation for: 2A-26 Rodeyton Gardens

Job No. 180709

Date: Dec 18

Calculation by: KSC

Checked by:

Sheet No. 51

Floor Slab - Check as propped Cantilever with Continuity over Spine wall

$DL = 9.5 kN/m^2$

$LL = 2.5 kN/m^2$

Factored Load = $9.5 \times 1.35 + 2.5 \times 1.5 = 16.5 kN/m^2$

Moment = $16.5 \times 8^2/8 = 132.5 kNm$

Span moment = $\frac{16.5 \times 8^2}{14.2} = 74.36 kNm$

Refer to output

275 mm THK Slab Satisfactory.

400 slab to front

DL	400 SLAB =	$9.6 kN/m^2$
	Soil =	$27 kN/m^2$
		$33.6 kN/m^2$
	LIVE =	$5.0 kN/m^2$

Factored Load = $33.6 \times 1.35 + 5.0 \times 1.5 = 52.86 kN/m^2$

Moment = $52.87 \times 7^2/8 = 323 kNm$

Refer to output

400 mm THK Slab Satisfactory.

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Check Basement Slab for uplift

Load.

Gravity = $0.3 \times 24 = 7.2\ kN/m^2$

Uplift = $60\ kN/m^2$

Refer to MPA Spreadsheet

Project	Spreadsheets to EC2				The Concrete Centre		
Client	MY Construction				Made by	Date	Page
Location	Basement Floor Uplift	from grids A to D			rmw	04-Jan-19	S 3
	FLAT SLAB ANALYSIS & DESIGN to EN 1992-1 : 2004				Checked	Revision	Job No
	Originated from TCC33.xlsm v 4.11 on CD	© 2003-2019 TCC			chg	-	

mpa The Concrete Centre

MATERIALS

fck	32	N/mm²			
fyk	500	N/mm²	dg	20	mm
fywk	500	N/mm²	γ_s	1.15	steel
$\varphi_{(t\text{-}t0)}$	2		γ_c	1.50	concrete
Steel class	A		Wk	0.4	mm top
				0.3	mm btm

COVERS

	mm	TO LAYER
Top cover	30	1
Btm cover	30	1
$\Delta_{c,dev}$	10	mm

SPANS

	L (m)
SPAN 1	4.000
SPAN 2	4.000
SPAN 3	
SPAN 4	
SPAN 5	
SPAN 6	

GEOMETRY

Bay type	INTERNAL	
Slab depth, h	300	mm
Int Panel width, b	3500	mm
End distance	325	from supt 1
End distance	325	from supt 3

PERIMETER LOADS *characteristic*

1.00 *kN/m outside supports 1 & 3*

LOADING PATTERN

	min	max
DEAD	1.35	1.35
IMPOSED		1.50

BS EN 1990: (6.10)

SUPPORTS

	ABOVE (m)	H (mm)	B (mm)	End Cond	BELOW (m)	H (mm)	B (mm)	End Cond
Support 1					3.75	450	450	F
Support 2					3.75	450	450	F
Support 3					3.75	450	450	F
Support 4								
Support 5								
Support 6								
Support 7								

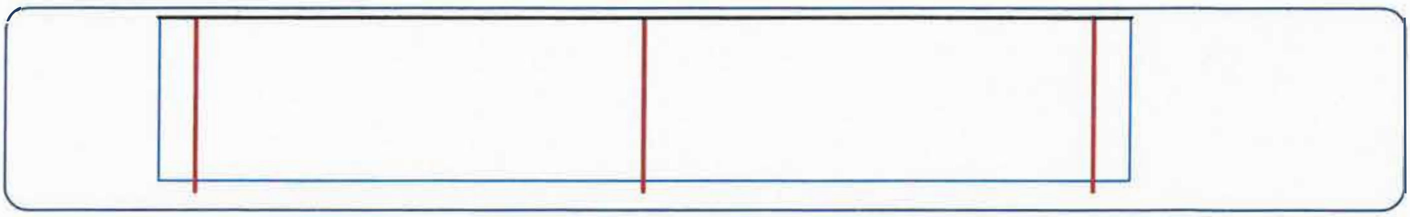
Usage: Dwelling *With brittle partitions*

LOADING

UDLs (kN/m²) PLs (kN/m) Position (m)

	Dead Load	Imposed Load	Position from left	Loaded Length		Dead Load	Imposed Load	Position from left	Loaded Length
Span 1					**Span 4**				
UDL	7.20	-60.00	~~~~~	~~~~~	UDL			~~~~~	~~~~~
PL 1				~~~~~	PL 1				~~~~~
PL 2				~~~~~	PL 2				~~~~~
Part UDL					Part UDL				
Span 2					**Span 5**				
UDL	7.20	-60.00	~~~~~	~~~~~	UDL			~~~~~	~~~~~
PL 1				~~~~~	PL 1				~~~~~
PL 2				~~~~~	PL 2				~~~~~
Part UDL					Part UDL				
Span 3					**Span 6**				
UDL			~~~~~	~~~~~	UDL			~~~~~	~~~~~
PL 1				~~~~~	PL 1				~~~~~
PL 2				~~~~~	PL 2				~~~~~
Part UDL					Part UDL				

LOADING DIAGRAM



Project	Spreadsheets to EC2		**The Concrete Centre**	
Client	MY Construction		Made by: rm	Date: 04-Jan-19	Page: S4
Location	Basement Floor Uplift, from grids A to D			
	FLAT SLAB ANALYSIS & DESIGN to EN 1992-1 : 2004		Checked:	Revision: -	Job NO: 180709
	Originated from TCC33.xlsmv 4.11 on CD	© 2003-2019 TCC		

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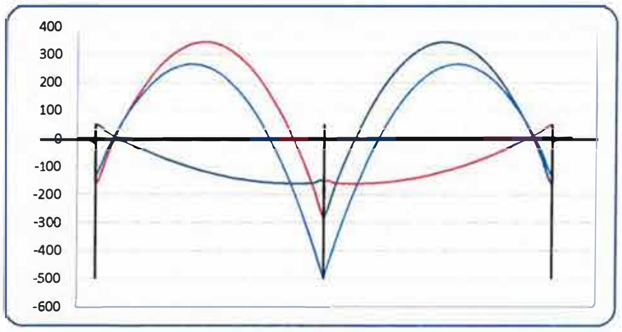
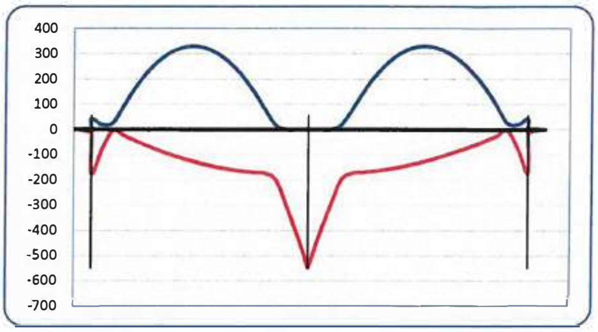
BENDING MOMENT DIAGRAMS (kNm)



Elastic Moments

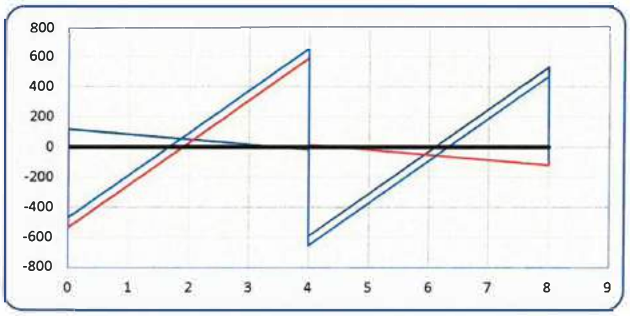


Redistributed Envelope

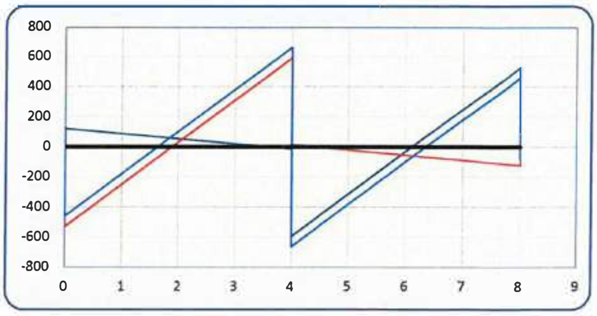
SUPPORT No	1	2	3				
Elastic M	47.6		47.6	~	~	~	~
Redistributed M	42.8		42.8	~	~	~	~
δ	0.900	1.000	0.900	~	~	~	~
Redistribution	10.0%	10.0%	10.0%				
End support reinf. Ø mm	12		12				

SPAN No	1	2				
Elastic M	161.3	498.1	~	~	~	~
Redistributed M	175.8	548.0	~	~	~	~
δ	1.090	1.100	~	~	~	~

SHEAR FORCE DIAGRAMS (kN/m)



Elastic Shears



Redistributed Shears

SPAN No	1		2			
Elastic V	531.3	654.6	654.6	531.3	~	~
Redistributed V	528.2	663.9	663.9	528.2	~	~

SPAN No						
Elastic V	~	~	~	~	~	~
Redistributed V	~	~	~	~	~	~

REACTIONS (kN/m)

	SUPPORT	1	2	3	
ALL SPANS LOADED		-546.7	-1327.7	-546.7	
MAXIMUM		35.4		35.4	
Characteristic Dead		65.2	94.6	65.2	
Characteristic Imposed					
For punching	ex /by				(6.43)
	u_1 / u_1^*	1.2578		1.2578	(6.44)

COLUMN MOMENTS (kNm)

		1	2	3
ALL SPANS	Above			
LOADED	Below	-114.3		114.3
WITH MAX	Above			
REACTION	Below	60.9		-60.9

SPAN 1

			LEFT	CENTRE	RIGHT
ACTIONS	δ		1.000	1.090	1.000
	Be		1000		1750
	Total M	kNm	27.8	175.8	0.0
	Mt max	kNm	379.1		653.5
MIDDLE STRIP	Width	mm	2500	1750	1750
	M	kNm	-9.5	79.1	
	d	mm	264.0	264.0	264.0
	As	mm²/m	-35	415	
	As deflection	mm²/m		79	
			Provide H12 @ 275 T1	*Provide H12 @ 250 B1*	*Provide H12 @ 275 T1*
	As prov	mm²/m	411	452	411
	Top steel			*Provide H12 @ 275 T1*	
	Deflection		L/d = 4,000 /264 = 15.15 < 40K = 48.00		OK
COLUMN STRIP	Width	mm	1000	1750	1750
	M	kNm	27.8	96.7	0.0
	d	mm	264.0	264.0	262.0
	As	mm²/m	255	507	0
	As deflection	mm²/m		142	
			Provide H12 @ 275 T1	*Provide H12 @ 200 B1*	*Provide H16 @ 350:700 T1*
	As prov	mm²/m	411	565	431
	Top steel			*Provide H16 @ 250 T1*	
	Deflection		L/d = 4,000 /264 = 15.15 < 40K = 48.00		OK
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	σs		ok		ok
	max S		ok	ok	ok

SPAN 2

			LEFT	CENTRE	RIGHT
ACTIONS	δ		1.000	1.100	1.000
	Be		1750		1000
	Total M	kNm	0.0	548.0	27.8
	Mt max	kNm	653.5		379.1
MIDDLE STRIP	Width	mm	1750	1750	2500
	M	kNm		246.6	-2.7
	d	mm	264.0	262.0	264.0
	As	mm²/m		1316	-10
	As deflection	mm²/m		520	
			Provide H12 @ 275 T1	*Provide H16 @ 150 B1*	*Provide H12 @ 275 T1*
	As prov	mm²/m	411	1340	411
	Top steel			*Provide H12 @ 275 T1*	
	Deflection		L/d = 4,000 /262 = 15.27 < 25.64 x 1.00 x 1.475 = 37.82		OK
COLUMN STRIP	Width	mm	1750	1750	1000
	M	kNm	0.0	301.4	27.8
	d	mm	262.0	262.0	264.0
	As	mm²/m	0	1634	255
	As deflection	mm²/m		759	
			Provide H16 @ 350:700 T1	*Provide H16 @ 100 B1*	*Provide H12 @ 275 T1*
	As prov	mm²/m	431	2011	411
	Top steel			*Provide H16 @ 250 T1*	
	Deflection		L/d = 4,000 /262 = 15.27 < 22.44 x 1.00 x 1.500 = 33.65		OK
CHECKS	% As		ok	ok	ok
	Singly reinforced		ok	ok	ok
	σs		ok	ok	ok
	max S		ok	ok	ok

Project	Spreadsheets to EC2	**The Concrete Centre**	
Client	MY Construction	Made by: rmw	Date: 04-Jan-19	Page: S6
Location	Basement Floor Uplift, from grids A to D	Checked: chg	Revision: -	Job No: 180709

FLAT SLAB ANALYSIS & DESIGN to EN 1992-1 : 2004

Originated from TCC33.xlsmv 4.11 on CD

WEIGHT of REINFORCEMENT

		No Mid Strip	No Col Strip	Type	Dia	Length	Unit wt	Weight
TOP STEEL	Support 1	10		T	12	1925	0.888	17.1
			5	T	12	1925	0.888	8.5
	Span 1	7		T	12	3350	0.888	20.8
			7	T	16	3800	1.578	42.0
	Support 2	7		T	12	2000	0.888	12.4
			4	T	16	2000	1.578	12.6
	Span 2	7		T	12	3350	0.888	20.8
			7	T	16	3800	1.578	42.0
	Support 3	10		T	12	1925	0.888	17.1
			5	T	12	1925	0.888	8.5
BTM STEEL	Span 1	7		T	12	3825	0.888	23.8
			9	T	12	4450	0.888	35.6
	Span 2	12		T	16	3825	1.578	72.4
			18	T	16	4450	1.578	126.4

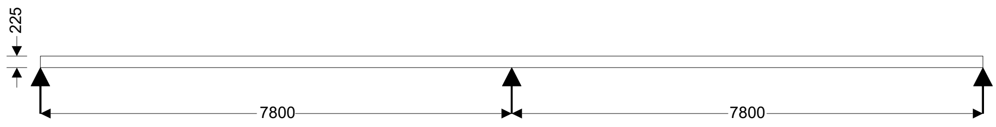
SUMMARY *Rebar for single direction only. All figures approximate - see User Guide.*

TOTAL REINFORCEMENT IN BAY (kg) 460 REINFORCEMENT DENSITY (kg/m^3) 50.7

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	**LGF Floor Slab**
Overall slab depth	h = **225** mm
Number of spans	N_{spans} = **2**
First support	**Simple**
Last support	**Simple**
Nominal cover to top reinforcement	c_{nom_t} = **25** mm
Nominal cover to bottom reinforcement	c_{nom_b} = **25** mm

Loading

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

Concrete properties

Concrete strength class	C32/40
Characteristic cylinder strength	f_{ck} = **32** 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} = **18.1** N/mm^2
Mean axial tensile strength (Table 3.1)	$f_{ctm} = 0.30\ \text{N/mm}^2 \times (f_{ck} / 1\ \text{N/mm}^2)^{2/3}$ = **3.0** 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} = **25** mm
Nominal cover to bottom reinforcement	c_{nom_b} = **25** 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} = **20** mm
Max bar diameter in bottom	ϕ_{max_b} = **20** mm
Min. top cover requirement with regard to bond	$c_{min,b\_t} = \phi_{max\_t}$ = **20** mm
Min. btm cover requirement with regard to bond	$c_{min,b\_b} = \phi_{max\_b}$ = **20** mm
Reinforcement fabrication	**Subject to QA system**
Cover allowance for deviation	Δc_{dev} = **5** mm
Min. required nominal cover to top reinft	$c_{nom\_t\_min} = \max(a_{fi\_t} - \phi_{max\_t} / 2, c_{min,b\_t} + \Delta c_{dev})$ = **25.0** mm
Min. required nominal cover to bottom reinft	$c_{nom\_b\_min} = \max(a_{fi\_b} - \phi_{max\_b} / 2, c_{min,b\_b} + \Delta c_{dev})$ = **25.0** 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 = **7800** mm

Design bending moment M_{p1} = **75.0** 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 $d_{p1} = h - c_{nom\_b} - \phi_{p1} / 2$ = **190.0** mm

K factor $K = M_{p1} / (b \times d_{p1}^2 \times f_{ck})$ = **0.065**

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 = **178.4** mm

Area of reinforcement required for bending $A_{sp1\_m} = M_{p1} / (f_{yd} \times z)$ = **967** mm^2/m

Minimum area required $A_{sp1\_min} = \max(0.26 \times (f_{ctm}/f_{yk}), 0.0013) \times b \times d_{p1}$ = **299** mm^2/m

Area of reinforcement required $A_{sp1\_req} = \max(A_{sp1\_m}, A_{sp1\_min})$ = **967** mm^2/m

PASS - Area of tension reinforcement provided is adequate (0.462)

Check reinforcement spacing

Reinforcement service stress $\sigma_s = (f_{yk} / \gamma_S) \times \min((A_{sp1\_m}/A_{sp1}), 1.0) \times r_q$ = **60.2** 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

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

Length of span 2 l_2 = **7800** mm

Design bending moment M_{p2} = **75.0** kNm/m

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

Area provided A_{sp2} = **2094** mm^2/m

Effective depth to tension reinforcement $d_{p2} = h - c_{nom\_b} - \phi_{p2} / 2$ = **190.0** mm

K factor $K = M_{p2} / (b \times d_{p2}^2 \times f_{ck})$ = **0.065**

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 = **178.4** mm

Area of reinforcement required for bending $A_{sp2\_m} = M_{p2} / (f_{yd} \times z)$ = **967** mm^2/m

Minimum area required $A_{sp2\_min} = \max(0.26 \times (f_{ctm}/f_{yk}), 0.0013) \times b \times d_{p2}$ = **299** mm^2/m

Area of reinforcement required $A_{sp2\_req} = \max(A_{sp2\_m}, A_{sp2\_min})$ = **967** mm^2/m

PASS - Area of tension reinforcement provided is adequate (0.462)

Check reinforcement spacing

Reinforcement service stress $\sigma_s = (f_{yk} / \gamma_S) \times \min((A_{sp2\_m}/A_{sp2}), 1.0) \times r_q$ = **60.2** N/mm^2

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

Actual bar spacing s_{p2} = **150** mm

PASS - The reinforcement spacing is acceptable

Reinforcement design at support 2 (cl.6.1)

Design bending moment M_{n2} = **132.5** kNm/m

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

Area provided — A_{sn2} = **2094** mm^2/m

Effective depth to tension reinforcement — $d_{n2} = h - c_{nom\_t} - \phi_{n2} / 2$ = **190.0** mm

K factor — $K = M_{n2} / (b \times d_{n2}^2 \times f_{ck})$ = **0.115**

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 = **168.3** mm

Area of reinforcement required for bending — $A_{sn2\_m} = M_{n2} / (f_{yd} \times z)$ = **1811** mm^2/m

Minimum area required — $A_{sn2\_min} = \max(0.26 \times (f_{ctm}/f_{yk}), 0.0013) \times b \times d_{n2}$ = **299** mm^2/m

Area of reinforcement required — $A_{sn2\_req} = \max(A_{sn2\_m}, A_{sn2\_min})$ = **1811** mm^2/m

PASS - Area of tension reinforcement provided is adequate (0.865)

Check reinforcement spacing

Reinforcement service stress — $\sigma_s = (f_{yk} / \gamma_S) \times \min((A_{sn2\_m}/A_{sn2}), 1.0) \times r_q$ = **112.8** N/mm^2

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

Actual bar spacing — s_{n2} = **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 = **66.0** kN/m

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

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

Effective depth — $d_{d1} = h - c_{nom\_b} - \phi_{d1} / 2$ = **190.0** mm

Effective depth factor (cl. 6.2.2) — $k = \min(2.0, 1 + (200 \text{ mm} / d_{d1})^{0.5})$ = **2.000**

Reinforcement ratio — $\rho_l = \min(0.02, A_{sd1} / (b \times d_{d1}))$ = **0.0110**

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} = **106.4** kN/m

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

$V_{Rd,c1}$ = **149.4** kN/m

PASS - Shear capacity is adequate (0.442)

Shear capacity check at support 2

Shear force — V_2 = **66.0** kN/m

Effective depth factor (cl. 6.2.2) — $k = \min(2.0, 1 + (200 \text{ mm} / d_{n2})^{0.5})$ = **2.000**

Reinforcement ratio — $\rho_l = \min(0.02, A_{sn2} / (b \times d_{n2}))$ = **0.0110**

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} = **106.4** kN/m

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

$V_{Rd,c2}$ = **149.4** kN/m

PASS - Shear capacity is adequate (0.442)

Shear capacity check at support 3

Shear force — V_3 = **66.0** kN/m

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

Area provided — A_{sd3} = **2094** mm^2/m

Effective depth $d_{d3} = h - c_{nom\_b} - \phi_{d3} / 2 =$ **190.0** mm

Effective depth factor (cl. 6.2.2) $k = \min(2.0, 1 + (200 \text{ mm} / d_{d3})^{0.5}) =$ **2.000**

Reinforcement ratio $\rho_l = \min(0.02, A_{sd3} / (b \times d_{d3})) =$ **0.0110**

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

$V_{Rd,c\_min} =$ **106.4** kN/m

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

$V_{Rd,c3} =$ **149.4** kN/m

PASS - Shear capacity is adequate (0.442)

Deflection checks

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

Reference reinforcement ratio $\rho_0 = (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / 1000 =$ **0.0057**

Required tension reinforcement ratio $\rho = \max(0.0035, A_{sp1\_m} / (b \times d_{p1})) =$ **0.0051**

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

Structural system factor (Table 7.4N) $K_\delta =$ **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) $ratio_{lim1\_bas} =$ **27.44**

Modified span-to-depth ratio limit

$ratio_{lim1} = \min(40 \times K_\delta, \min(1.5, (500 \text{ N/mm}^2/ f_{yk}) \times (A_{sp1} / A_{sp1\_m})) \times ratio_{lim1\_bas}) =$ **41.16**

Actual span-to-depth ratio $ratio_{act1} = l_1 / d_{p1} =$ **41.05**

PASS - Span-to-depth ratio is acceptable (0.997)

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

Reference reinforcement ratio $\rho_0 = (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / 1000 =$ **0.0057**

Required tension reinforcement ratio $\rho = \max(0.0035, A_{sp2\_m} / (b \times d_{p2})) =$ **0.0051**

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

Structural system factor (Table 7.4N) $K_\delta =$ **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) $ratio_{lim2\_bas} =$ **27.44**

Modified span-to-depth ratio limit

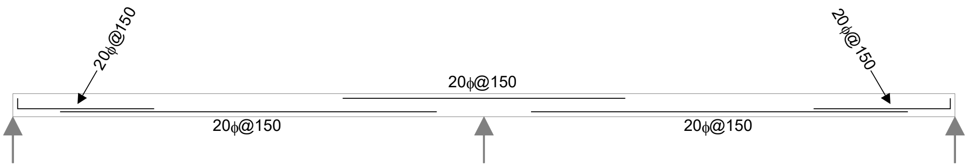
$ratio_{lim2} = \min(40 \times K_\delta, \min(1.5, (500 \text{ N/mm}^2/ f_{yk}) \times (A_{sp2} / A_{sp2\_m})) \times ratio_{lim2\_bas}) =$ **41.16**

Actual span-to-depth ratio $ratio_{act2} = l_2 / d_{p2} =$ **41.05**

PASS - Span-to-depth ratio is acceptable (0.997)

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.



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

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

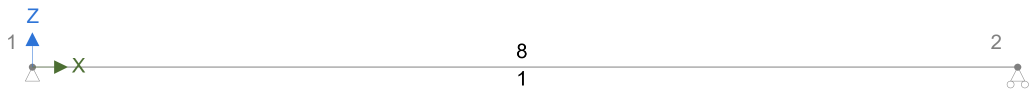
Tedds calculation version 3.0.13

ANALYSIS

Tedds calculation version 1.0.23

Geometry

Geometry (m) - Concrete (C32 2500 Quartzite) - R 1200x650



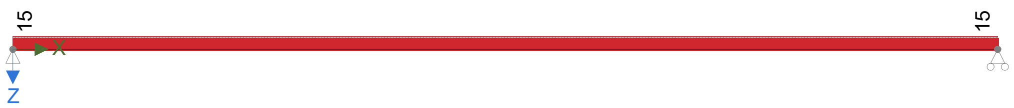
Span	Length (m)	Section	Start Support	End Support
1	8	R 1200x650	Pinned	Roller Pin X
R 1200x650: A = 7800 cm^2, I_y = 2746250 cm^4, I_z = 9360000 cm^4, A_y = 6500 cm^2, A_z = 6500 cm^2				
Concrete (C32 2500 Quartzite): Density 2500 kg/m^3, Youngs 33.3457645 kN/mm^2, Shear 13.8940685 kN/mm^2, Thermal 0.00001 °C^{-1}				

Loading

Self weight included

Permanent - Loading (kN/m)

Imposed - Loading (kN/m)



Load combination factors

Load combination	Self Weight	Permanent	Imposed
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.35	1.50
1.0G + 1.0Q + 1.0RQ (Service)	1.00	1.00	1.00
1.0G + 1.0ψ_2Q (Quasi)	1.00	1.00	0.30
1.35G + 1.5Q + 1.5ψ_0S (Strength)	1.35	1.35	1.50
1.0G + 1.0Q + 0.5S (Service)	1.00	1.00	1.00
1.35G + 1.5ψ_0Q + 1.5S (Strength)	1.35	1.35	1.05

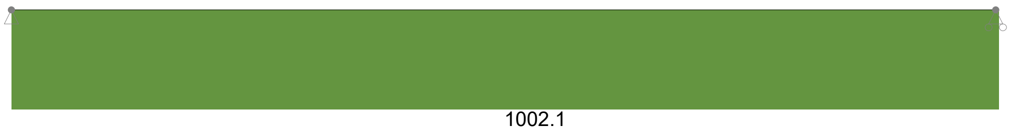
Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Permanent	UDL	GlobalZ	57 kN/m
Beam	Imposed	UDL	GlobalZ	15 kN/m

Results

Forces

Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)

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

Concrete strength class	C32/40
Aggregate type	Quartzite
Aggregate adjustment factor - cl.3.1.3(2)	AAF = **1.0**
Characteristic compressive cylinder strength	f_{ck} = **32** N/mm^2
Mean value of compressive cylinder strength	f_{cm} = f_{ck} + 8 N/mm^2 = **40** N/mm^2
Mean value of axial tensile strength	f_{ctm} = 0.3 N/mm^2 $\times$ (f_{ck}/ 1 N/mm^2)$^{2/3}$ = **3.0** 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 = **33346** 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**

Tekla Tedds Cranston Consulting Limited 16 Jubilee Road Newtownards BT23 4YH	Project Redington Gardens			Job no. 180709	
	Calcs for LGF Beam on GL D			Start page no./Revision B3	
	Calcs by S	Calcs date 09/05/2019	Checked by	Checked date	Approved by	Approved date

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 — $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **18.1** N/mm^2

Compressive strength coefficient - cl.3.1.6(1) — α_{ccw} = **1.00**

Design compressive concrete strength - exp.3.15 — $f_{cwd} = \alpha_{ccw} \times f_{ck} / \gamma_C$ = **21.3** 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} = **35** mm

Nominal cover to bottom reinforcement — c_{nom_b} = **35** mm

Nominal cover to side reinforcement — c_{nom_s} = **35** 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

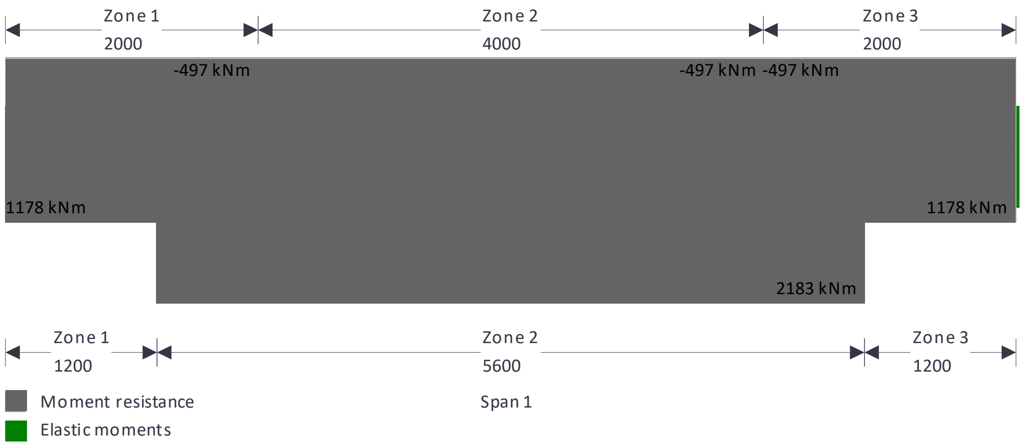
Rectangular section details

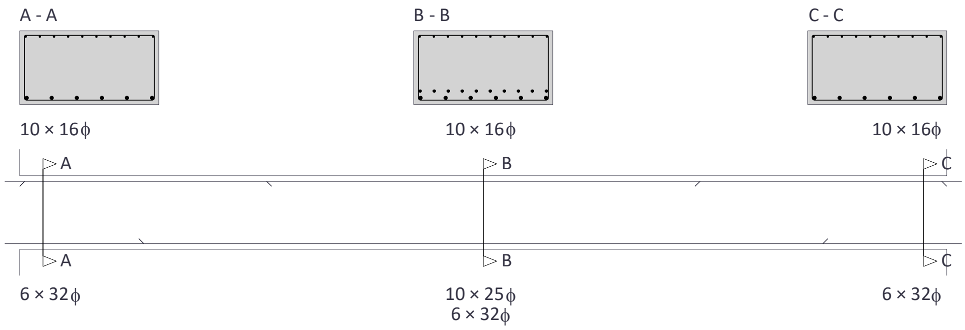
Section width — b = **1200** mm

Section depth — h = **650** mm

PASS - Minimum dimensions for fire resistance met

Moment design





Zone 1 (0 mm - 1200 mm) Positive moment - section 6.1

Design bending moment	$M = abs(M_{m1\_s1\_z1\_max\_red})$ = **511.1** kNm
Effective depth of tension reinforcement	d = **591** mm
Redistribution ratio	$\delta = min(M_{pos\_red\_z1} / M_{pos\_z1}, 1)$ = **1.000**
	$K = M / (b \times d^2 \times f_{ck})$ = **0.038**
	$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2))$ = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d)$ = **561** mm
Depth of neutral axis	$x = 2 \times (d - z) / \lambda$ = **74** mm
Area of tension reinforcement required	$A_{s,req} = M / (f_{yd} \times z)$ = **2094** mm²
Tension reinforcement provided	6 × 32ϕ
Area of tension reinforcement provided	$A_{s,prov}$ = **4825** mm²
Minimum area of reinforcement - exp.9.1N	$A_{s,min} = max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d$ = **1115** mm²
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{s,max} = 0.04 \times b \times h$ = **31200** 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/mm²
Mean value of concrete tensile strength	$f_{ct,eff} = f_{ctm}$ = **3.0** N/mm²
Stress distribution coefficient	k_c = **0.4**
Non-uniform self-equilibrating stress coefficient	$k = min(max(1 + (300\text{ mm} - min(h, b)) \times 0.35 / 500\text{ mm}, 0.65), 1)$ = **0.76**
Actual tension bar spacing	$s_{bar} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / (N_{m1\_s1\_z1\_b\_L1} - 1) + \phi_{m1\_s1\_z1\_b\_L1}$ = **216.4** mm
Maximum stress permitted - Table 7.3N	σ_s = **227** N/mm²
Steel to concrete modulus of elast. ratio	$\alpha_{cr} = E_s / E_{cm}$ = **6.00**
Distance of the Elastic NA from bottom of beam	$y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1))$ = **317** mm
Area of concrete in the tensile zone	$A_{ct} = b \times y$ = **380427** mm²
Minimum area of reinforcement required - exp.7.1	$A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s$ = **1531** mm²

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

Quasi-permanent moment	$M_{QP} = max(\beta_1 \times abs(M_{m1\_s1\_z2\_neg\_quasi}), abs(M_{m1\_s1\_z1\_pos\_quasi}))$ = **328.9**kNm
Permanent load ratio	$R_{PL} = M_{QP} / M$ = **0.64**
Service stress in reinforcement	$\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL}$ = **121** N/mm²

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 - 2000 mm) Negative moment - section 6.1

Design bending moment $M = \max(\beta_1 \times \text{abs}(M_{m1\_s1\_max\_red}), \text{abs}(M_{m1\_s1\_z1\_min\_red}))$ = **250.5** kNm

Effective depth of tension reinforcement d = **599** mm

Redistribution ratio $\delta = 1$ = **1.000**

$K = M / (b \times d^2 \times f_{ck})$ = **0.018**

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

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d)$ = **569** mm

Depth of neutral axis $x = 2 \times (d - z) / \lambda$ = **75** mm

Area of tension reinforcement required $A_{s,req} = M / (f_{yd} \times z)$ = **1013** mm^2

Tension reinforcement provided 10 × 16ϕ

Area of tension reinforcement provided $A_{s,prov}$ = **2011** mm^2

Minimum area of reinforcement - exp.9.1N $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d$ = **1130** mm^2

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{s,max} = 0.04 \times b \times h$ = **31200** 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 $f_{ct,eff} = f_{ctm}$ = **3.0** N/mm^2

Stress distribution coefficient k_c = **0.4**

Non-uniform self-equilibrating stress coefficient $k = \min(\max(1 + (300 \text{ mm} - \min(h, b)) \times 0.35 / 500 \text{ mm}, 0.65), 1)$ = **0.76**

Actual tension bar spacing $s_{bar} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_t\_L1} \times N_{m1\_s1\_z1\_t\_L1})) / (N_{m1\_s1\_z1\_t\_L1} - 1) + \phi_{m1\_s1\_z1\_t\_L1}$ = **122** mm

Maximum stress permitted - Table 7.3N σ_s = **302** N/mm^2

Steel to concrete modulus of elast. ratio $\alpha_{cr} = E_s / E_{cm}$ = **6.00**

Distance of the Elastic NA from bottom of beam $y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1))$ = **322** mm

Area of concrete in the tensile zone $A_{ct} = b \times y$ = **385818** mm^2

Minimum area of reinforcement required - exp.7.1 $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s$ = **1165** mm^2

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

Quasi-permanent moment $M_{QP} = \max(\beta_1 \times \text{abs}(M_{m1\_s1\_z2\_pos\_quasi}), \text{abs}(M_{m1\_s1\_z1\_neg\_quasi}))$ = **161.2**kNm

Permanent load ratio $R_{PL} = M_{QP} / M$ = **0.64**

Service stress in reinforcement $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL}$ = **141** 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 $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_t\_L1} \times N_{m1\_s1\_z1\_t\_L1})) / (N_{m1\_s1\_z1\_t\_L1} - 1)$ = **106.0** mm

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

PASS - Actual bar spacing exceeds minimum allowable

Bottom bar spacing $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / (N_{m1\_s1\_z1\_b\_L1} - 1) = \mathbf{184.4}$ mm

Minimum allowable bottom bar spacing $s_{bot,min} = \max(\phi_{m1\_s1\_z1\_b\_L1} \times k_{s1}, h_{agg} + k_{s2}, 20mm) = \mathbf{32.0}$ mm

PASS - Actual bar spacing exceeds minimum allowable

Zone 2 (1200 mm - 6800 mm) Positive moment - section 6.1

Design bending moment $M = abs(M_{m1\_s1\_z2\_max\_red}) = \mathbf{1002.1}$ kNm

Effective depth of tension reinforcement $d = \mathbf{560}$ mm

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

$K = M / (b \times d^2 \times f_{ck}) = \mathbf{0.083}$

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

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d) = \mathbf{516}$ mm

Depth of neutral axis $x = 2 \times (d - z) / \lambda = \mathbf{112}$ mm

Area of tension reinforcement required $A_{s,req} = M / (f_{yd} \times z) = \mathbf{4468}$ mm^2

Tension reinforcement provided Layer 1 - 6 × 32ϕ, Spacing - 32mm, Layer 2 - 10 × 25ϕ

Area of tension reinforcement provided $A_{s,prov} = \mathbf{9734}$ mm^2

Minimum area of reinforcement - exp.9.1N $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d = \mathbf{1058}$ mm^2

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{s,max} = 0.04 \times b \times h = \mathbf{31200}$ 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 $f_{ct,eff} = f_{ctm} = \mathbf{3.0}$ N/mm^2

Stress distribution coefficient $k_c = \mathbf{0.4}$

Non-uniform self-equilibrating stress coefficient $k = \min(\max(1 + (300 mm - \min(h, b)) \times 0.35 / 500 mm, 0.65), 1) = \mathbf{0.76}$

Actual tension bar spacing $s_{bar} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2\_b\_L1} \times N_{m1\_s1\_z2\_b\_L1} + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / ((N_{m1\_s1\_z2\_b\_L1} + N_{m1\_s1\_z1\_b\_L1}) - 1) + \phi_{m1\_s1\_z2\_b\_L1} = \mathbf{216.4}$ mm

Maximum stress permitted - Table 7.3N $\sigma_s = \mathbf{227}$ N/mm^2

Steel to concrete modulus of elast. ratio $\alpha_{cr} = E_s / E_{cm} = \mathbf{6.00}$

Distance of the Elastic NA from bottom of beam $y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1)) = \mathbf{311}$ mm

Area of concrete in the tensile zone $A_{ct} = b \times y = \mathbf{373409}$ mm^2

Minimum area of reinforcement required - exp.7.1 $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s = \mathbf{1503}$ mm^2

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

Quasi-permanent moment $M_{QP} = abs(M_{m1_s1_z2_pos_quasi}) = \mathbf{645.0}$kNm

Permanent load ratio $R_{PL} = M_{QP} / M = \mathbf{0.64}$

Service stress in reinforcement $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL} = \mathbf{128}$ 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

Deflection control - Section 7.4

Reference reinforcement ratio $\rho_{m0} = (f_{ck} / 1 N/mm^2)^{0.5} / 1000 = \mathbf{0.00566}$

Required tension reinforcement ratio $\rho_m = A_{s,req} / (b \times d) = \mathbf{0.00664}$

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 $span\_to\_depth_{basic} = K_b \times [11 + 1.5 \times (f_{ck} / 1\ N/mm^2)^{0.5} \times \rho_{m0} / (\rho_m - \rho'_m) + (f_{ck} / 1\ N/mm^2)^{0.5} \times (\rho'_m / \rho_{m0})^{0.5} / 12] =$ **18.225**

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

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

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

Allowable span to depth ratio $span\_to\_depth_{allow} = \min(span\_to\_depth_{basic} \times K_s \times F1 \times F2, 40 \times K_b) =$ **27.338**

Actual span to depth ratio $span\_to\_depth_{actual} = L_{m1\_s1} / d =$ **14.273**

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

Minimum bar spacing (Section 8.2)

Top bar spacing $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2\_t\_L1} \times N_{m1\_s1\_z2\_t\_L1})) / (N_{m1\_s1\_z2\_t\_L1} - 1) =$ **106.0** mm

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

PASS - Actual bar spacing exceeds minimum allowable

Bottom bar spacing $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2\_b\_L1} \times N_{m1\_s1\_z2\_b\_L1} + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / ((N_{m1\_s1\_z2\_b\_L1} + N_{m1\_s1\_z1\_b\_L1}) - 1) =$ **184.4** mm

Minimum allowable bottom bar spacing $s_{bot,min} = \max(\phi_{m1\_s1\_z2\_b\_L1} \times k_{s1}, h_{agg} + k_{s2}, 20mm) =$ **32.0** mm

PASS - Actual bar spacing exceeds minimum allowable

Zone 3 (6800 mm - 8000 mm) Positive moment - section 6.1

Design bending moment $M = abs(M_{m1\_s1\_z3\_max\_red}) =$ **511.1** kNm

Effective depth of tension reinforcement $d =$ **591** mm

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

$K = M / (b \times d^2 \times f_{ck}) =$ **0.038**

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

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d) =$ **561** mm

Depth of neutral axis $x = 2 \times (d - z) / \lambda =$ **74** mm

Area of tension reinforcement required $A_{s,req} = M / (f_{yd} \times z) =$ **2094** mm²

Tension reinforcement provided $6 \times 32\phi$

Area of tension reinforcement provided $A_{s,prov} =$ **4825** mm²

Minimum area of reinforcement - exp.9.1N $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d =$ **1115** mm²

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{s,max} = 0.04 \times b \times h =$ **31200** 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/mm²

Mean value of concrete tensile strength $f_{ct,eff} = f_{ctm} =$ **3.0** N/mm²

Stress distribution coefficient $k_c =$ **0.4**

Non-uniform self-equilibrating stress coefficient $k = \min(\max(1 + (300\ mm - \min(h, b)) \times 0.35 / 500\ mm, 0.65), 1) =$ **0.76**

Actual tension bar spacing — $s_{bar} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_b\_L1} \times N_{m1\_s1\_z3\_b\_L1})) / (N_{m1\_s1\_z3\_b\_L1} - 1) + \phi_{m1\_s1\_z3\_b\_L1}$ = **216.4** mm

Maximum stress permitted - Table 7.3N — σ_s = **227** N/mm^2

Steel to concrete modulus of elast. ratio — $\alpha_{cr} = E_s / E_{cm}$ = **6.00**

Distance of the Elastic NA from bottom of beam — $y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1))$ = **317** mm

Area of concrete in the tensile zone — $A_{ct} = b \times y$ = **380427** mm^2

Minimum area of reinforcement required - exp.7.1 — $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s$ = **1531** mm^2

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

Quasi-permanent moment — $M_{QP} = \max(\beta_1 \times \text{abs}(M_{m1\_s1\_z2\_neg\_quasi}), \text{abs}(M_{m1\_s1\_z3\_pos\_quasi}))$ = **328.9**kNm

Permanent load ratio — $R_{PL} = M_{QP} / M$ = **0.64**

Service stress in reinforcement — $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL}$ = **121** 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 3 (6000 mm - 8000 mm) Negative moment - section 6.1

Design bending moment — $M = \max(\beta_1 \times \text{abs}(M_{m1\_s1\_max\_red}), \text{abs}(M_{m1\_s1\_z3\_min\_red}))$ = **250.5** kNm

Effective depth of tension reinforcement — d = **599** mm

Redistribution ratio — δ = 1 = **1.000**

$K = M / (b \times d^2 \times f_{ck})$ = **0.018**

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

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d)$ = **569** mm

Depth of neutral axis — $x = 2 \times (d - z) / \lambda$ = **75** mm

Area of tension reinforcement required — $A_{s,req} = M / (f_{yd} \times z)$ = **1013** mm^2

Tension reinforcement provided — 10 × 16ϕ

Area of tension reinforcement provided — $A_{s,prov}$ = **2011** mm^2

Minimum area of reinforcement - exp.9.1N — $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d$ = **1130** mm^2

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{s,max} = 0.04 \times b \times h$ = **31200** 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 — $f_{ct,eff} = f_{ctm}$ = **3.0** 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) = **0.76**

Actual tension bar spacing — $s_{bar} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_t\_L1} \times N_{m1\_s1\_z3\_t\_L1})) / (N_{m1\_s1\_z3\_t\_L1} - 1) + \phi_{m1\_s1\_z3\_t\_L1}$ = **122** mm

Maximum stress permitted - Table 7.3N — σ_s = **302** N/mm^2

Steel to concrete modulus of elast. ratio — $\alpha_{cr} = E_s / E_{cm}$ = **6.00**

Distance of the Elastic NA from bottom of beam — $y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1))$ = **322** mm

Area of concrete in the tensile zone — $A_{ct} = b \times y$ = **385818** mm^2

Minimum area of reinforcement required - exp.7.1 $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s$ = **1165** mm^2

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

Quasi-permanent moment $M_{QP} = \max(\beta_1 \times \mathrm{abs}(M_{m1\_s1\_z2\_pos\_quasi}), \mathrm{abs}(M_{m1\_s1\_z3\_neg\_quasi}))$ = **161.2**kNm

Permanent load ratio $R_{PL} = M_{QP} / M$ = **0.64**

Service stress in reinforcement $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL}$ = **141** 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 $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_t\_L1} \times N_{m1\_s1\_z3\_t\_L1})) / (N_{m1\_s1\_z3\_t\_L1} - 1)$ = **106.0** 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

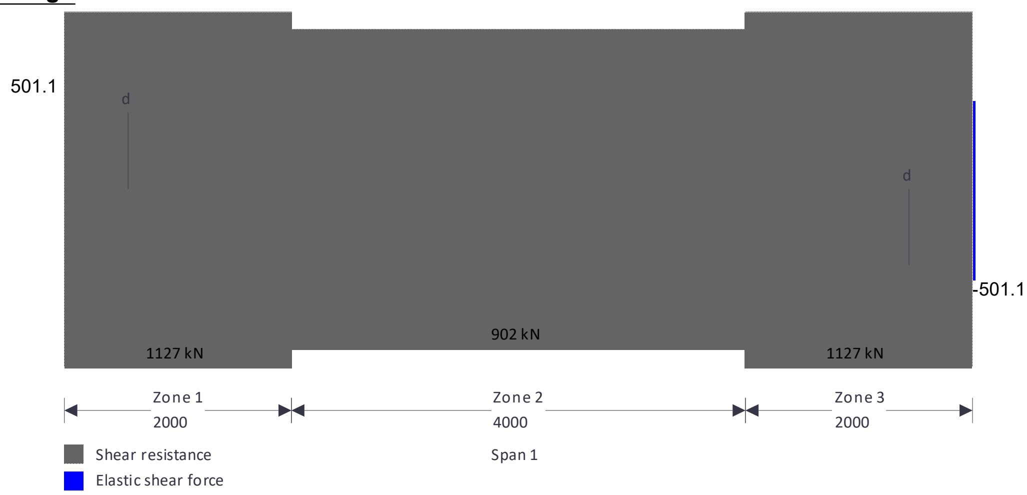
PASS - Actual bar spacing exceeds minimum allowable

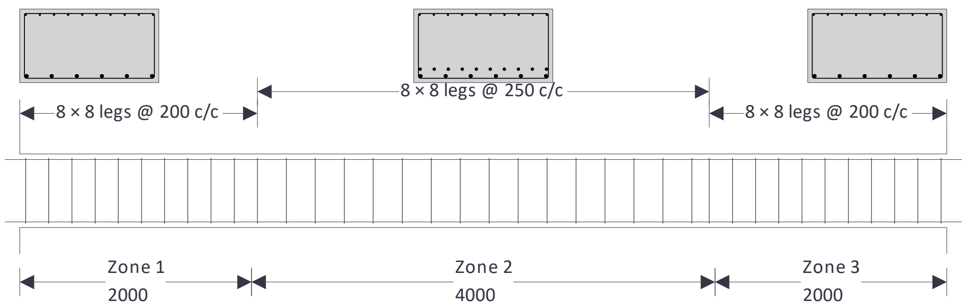
Bottom bar spacing $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_b\_L1} \times N_{m1\_s1\_z3\_b\_L1})) / (N_{m1\_s1\_z3\_b\_L1} - 1)$ = **184.4** mm

Minimum allowable bottom bar spacing $s_{bot,min} = \max(\phi_{m1\_s1\_z3\_b\_L1} \times k_{s1}, h_{agg} + k_{s2}, 20\text{mm})$ = **32.0** 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) $v_1 = 0.6 \times (1 - f_{ck} / 250 \text{ N/mm}^2)$ = **0.523**

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

Minimum area of shear reinforcement - exp.9.5N $A_{sv,min} = 0.08 \text{ N/mm}^2 \times b \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / f_{yk}$ = **1086** mm^2/m

Zone 1 (0 mm - 2000 mm) shear - section 6.2

Design shear force at support $V_{Ed,max}$ = max(abs(V_{z1_max}), abs($V_{z1\_red\_max}$)) = **501** kN

Min lever arm in shear zone z = **516** mm

Maximum design shear resistance - exp.6.9 $V_{Rd,max} = \alpha_{cw} \times b \times z \times v_1 \times f_{cwd} / (\cot(\theta_{max}) + \tan(\theta_{max}))$ = **3455** kN

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

Design shear force at 560mm from support V_{Ed} = **431** kN

Design shear stress $v_{Ed} = V_{Ed} / (b \times z)$ = **0.696** N/mm^2

Angle of concrete compression strut - cl.6.2.3 θ = min(max(0.5 × Asin(min(2 × 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 $A_{sv,des} = v_{Ed} \times b / (f_{yd} \times \cot(\theta))$ = **768** mm^2/m

Area of shear reinforcement required $A_{sv,req}$ = max($A_{sv,min}$, $A_{sv,des}$) = **1086** mm^2/m

Shear reinforcement provided 8 × 8 legs @ 200 c/c

Area of shear reinforcement provided $A_{sv,prov}$ = **2011** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N $s_{vl,max} = 0.75 \times d$ = **420** mm

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

Zone 2 (2000 mm - 6000 mm) shear - section 6.2

Design shear force at support $V_{Ed,max}$ = max(abs(V_{z2_max}), abs($V_{z2\_red\_max}$)) = **251** kN

Min lever arm in shear zone z = **516** mm

Maximum design shear resistance - exp.6.9 $V_{Rd,max} = \alpha_{cw} \times b \times z \times v_1 \times f_{cwd} / (\cot(\theta_{max}) + \tan(\theta_{max}))$ = **3455** kN

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

Design shear force within zone V_{Ed} = **251** kN

Design shear stress $v_{Ed} = V_{Ed} / (b \times z)$ = **0.405** N/mm^2

Angle of concrete compression strut - cl.6.2.3 θ = min(max(0.5 × Asin(min(2 × 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 $A_{sv,des} = v_{Ed} \times b / (f_{yd} \times \cot(\theta))$ = **447** mm^2/m

Area of shear reinforcement required $A_{sv,req}$ = max($A_{sv,min}$, $A_{sv,des}$) = **1086** mm^2/m

Shear reinforcement provided 8 × 8 legs @ 250 c/c

Area of shear reinforcement provided $A_{sv,prov}$ = **1608** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N $s_{vl,max} = 0.75 \times d$ = **420** mm

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

Zone 3 (6000 mm - 8000 mm) shear - section 6.2

Design shear force at support $V_{Ed,max}$ = max(abs(V_{z3_max}), abs($V_{z3\_red\_max}$)) = **501** kN

Min lever arm in shear zone z = **516** mm

Maximum design shear resistance - exp.6.9 $V_{Rd,max} = \alpha_{cw} \times b \times z \times v_1 \times f_{cwd} / (\cot(\theta_{max}) + \tan(\theta_{max}))$ = **3455** kN

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

Design shear force at 560mm from support V_{Ed} = **431** kN

Design shear stress $v_{Ed} = V_{Ed} / (b \times z)$ = **0.696** N/mm^2

Angle of concrete compression strut - cl.6.2.3 θ = min(max(0.5 × Asin(min(2 × 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 $A_{sv,des} = v_{Ed} \times b / (f_{yd} \times \cot(\theta))$ = **768** mm^2/m

Area of shear reinforcement required $A_{sv,req}$ = max($A_{sv,min}$, $A_{sv,des}$) = **1086** mm^2/m

Shear reinforcement provided 8 × 8 legs @ 200 c/c

Area of shear reinforcement provided $A_{sv,prov}$ = **2011** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N $s_{vl,max} = 0.75 \times d =$ **420** mm

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

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

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

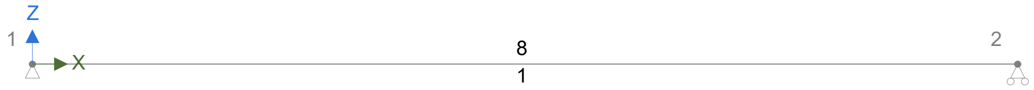
Tedds calculation version 3.0.13

ANALYSIS

Tedds calculation version 1.0.23

Geometry

Geometry (m) - Concrete (C32 2500 Quartzite) - Custom R 650x650



Span	Length (m)	Section	Start Support	End Support
1	8	Custom R 650x650	Pinned	Roller Pin X
Custom R 650x650: A = 6525 cm^2, I_y = 1101094 cm^4, I_z = 1.×10^7 cm^4, A_y = 5438 cm^2, A_z = 5438 cm^2				
Concrete (C32 2500 Quartzite): Density 2500 kg/m^3, Youngs 33.3457645 kN/mm^2, Shear 13.8940685 kN/mm^2, Thermal 0.00001 °C^{-1}				

Loading

Self weight included

Permanent - Loading (kN/m)

Imposed - Loading (kN/m)

Load combination factors

Load combination	Self Weight	Permanent	Imposed
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.35	1.50
1.0G + 1.0Q + 1.0RQ (Service)	1.00	1.00	1.00
1.0G + 1.0ψ_2Q (Quasi)	1.00	1.00	0.30
1.35G + 1.5Q + 1.5ψ_0S (Strength)	1.35	1.35	1.50
1.0G + 1.0Q + 0.5S (Service)	1.00	1.00	1.00
1.35G + 1.5ψ_0Q + 1.5S (Strength)	1.35	1.35	1.05

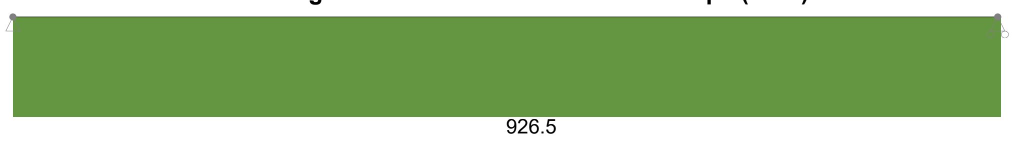
Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Permanent	UDL	GlobalZ	47.5 kN/m
Beam	Permanent	UDL	GlobalZ	8.4 kN/m
Beam	Imposed	UDL	GlobalZ	12.5 kN/m

Results

Forces

Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)

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

Concrete strength class	C32/40
Aggregate type	Quartzite
Aggregate adjustment factor - cl.3.1.3(2)	AAF = **1.0**
Characteristic compressive cylinder strength	f_{ck} = **32** N/mm^2
Mean value of compressive cylinder strength	f_{cm} = f_{ck} + 8 N/mm^2 = **40** N/mm^2
Mean value of axial tensile strength	f_{ctm} = 0.3 N/mm^2 $\times$ (f_{ck}/ 1 N/mm^2)$^{2/3}$ = **3.0** 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 = **33346** 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	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **18.1** N/mm^2
Compressive strength coefficient - cl.3.1.6(1)	α_{ccw} = **1.00**
Design compressive concrete strength - exp.3.15	$f_{cwd} = \alpha_{ccw} \times f_{ck} / \gamma_C$ = **21.3** 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} = **35** mm
Nominal cover to bottom reinforcement	c_{nom_b} = **35** mm
Nominal cover to side reinforcement	c_{nom_s} = **35** 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

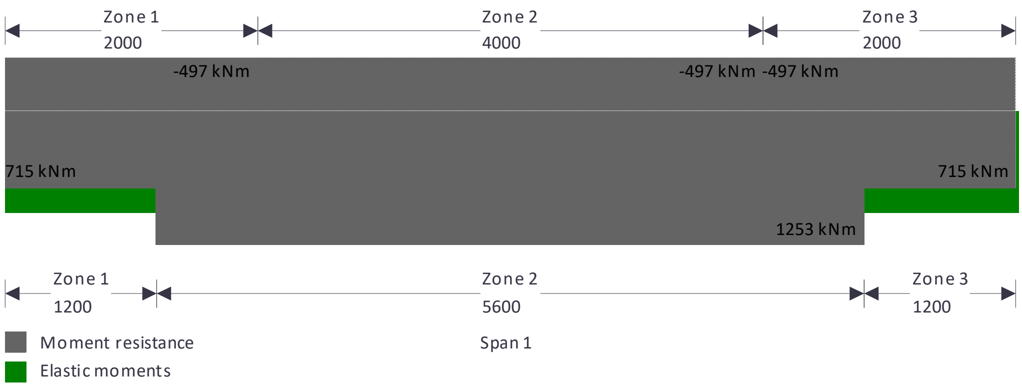
Beam - Span 1

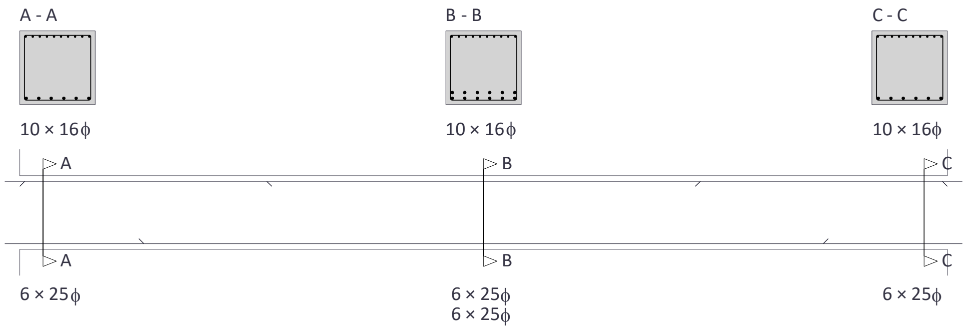
Rectangular section details

Section width	b = **650** mm
Section depth	h = **650** mm

PASS - Minimum dimensions for fire resistance met

Moment design





Zone 1 (0 mm - 1200 mm) Positive moment - section 6.1

Design bending moment — $M = abs(M_{m1\_s1\_z1\_max\_red})$ = **472.5** kNm

Effective depth of tension reinforcement — d = **595** mm

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

$K = M / (b \times d^2 \times f_{ck})$ = **0.064**

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

K' > K - No compression reinforcement is required

Lever arm — $z = min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d)$ = **559** mm

Depth of neutral axis — $x = 2 \times (d - z) / \lambda$ = **90** mm

Area of tension reinforcement required — $A_{s,req} = M / (f_{yd} \times z)$ = **1945** mm^2

Tension reinforcement provided — 6 × 25ϕ

Area of tension reinforcement provided — $A_{s,prov}$ = **2945** mm^2

Minimum area of reinforcement - exp.9.1N — $A_{s,min} = max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d$ = **608** mm^2

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{s,max} = 0.04 \times b \times h$ = **16900** 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 — $f_{ct,eff} = f_{ctm}$ = **3.0** N/mm^2

Stress distribution coefficient — k_c = **0.4**

Non-uniform self-equilibrating stress coefficient — $k = min(max(1 + (300 mm - min(h, b)) \times 0.35 / 500 mm, 0.65), 1)$ = **0.76**

Actual tension bar spacing — $s_{bar} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / (N_{m1\_s1\_z1\_b\_L1} - 1) + \phi_{m1\_s1\_z1\_b\_L1}$ = **107.8** mm

Maximum stress permitted - Table 7.3N — σ_s = **314** N/mm^2

Steel to concrete modulus of elast. ratio — $\alpha_{cr} = E_s / E_{cm}$ = **6.00**

Distance of the Elastic NA from bottom of beam — $y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1))$ = **316** mm

Area of concrete in the tensile zone — $A_{ct} = b \times y$ = **205352** mm^2

Minimum area of reinforcement required - exp.7.1 — $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s$ = **598** mm^2

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

Quasi-permanent moment — $M_{QP} = max(\beta_1 \times abs(M_{m1\_s1\_z2\_neg\_quasi}), abs(M_{m1\_s1\_z1\_pos\_quasi}))$ = **308.6**kNm

Permanent load ratio — $R_{PL} = M_{QP} / M$ = **0.65**

Service stress in reinforcement — $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL}$ = **188** N/mm^2

Maximum bar spacing - Tables 7.3N $s_{bar,max}$ = **265.5** mm

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

Zone 1 (0 mm - 2000 mm) Negative moment - section 6.1

Design bending moment	$M = \max(\beta_1 \times \text{abs}(M_{m1\_s1\_max\_red}), \text{abs}(M_{m1\_s1\_z1\_min\_red}))$ = **231.6** kNm
Effective depth of tension reinforcement	d = **599** mm
Redistribution ratio	δ = 1 = **1.000**
	$K = M / (b \times d^2 \times f_{ck})$ = **0.031**
	$K' = (2 \times \eta \times \alpha_{cc} / \gamma_C) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2))$ = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d)$ = **569** mm
Depth of neutral axis	$x = 2 \times (d - z) / \lambda$ = **75** mm
Area of tension reinforcement required	$A_{s,req} = M / (f_{yd} \times z)$ = **936** mm^2
Tension reinforcement provided	10 $\times$ 16ϕ
Area of tension reinforcement provided	$A_{s,prov}$ = **2011** mm^2
Minimum area of reinforcement - exp.9.1N	$A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d$ = **612** mm^2
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{s,max} = 0.04 \times b \times h$ = **16900** 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	$f_{ct,eff} = f_{ctm}$ = **3.0** N/mm^2
Stress distribution coefficient	k_c = **0.4**
Non-uniform self-equilibrating stress coefficient	$k = \min(\max(1 + (300 \text{ mm} - \min(h, b)) \times 0.35 / 500 \text{ mm}, 0.65), 1)$ = **0.76**
Actual tension bar spacing	$s_{bar} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_t\_L1} \times N_{m1\_s1\_z1\_t\_L1})) / (N_{m1\_s1\_z1\_t\_L1} - 1) + \phi_{m1\_s1\_z1\_t\_L1}$ = **60.9** mm
Maximum stress permitted - Table 7.3N	σ_s = **351** N/mm^2
Steel to concrete modulus of elast. ratio	$\alpha_{cr} = E_s / E_{cm}$ = **6.00**
Distance of the Elastic NA from bottom of beam	$y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1))$ = **319** mm
Area of concrete in the tensile zone	$A_{ct} = b \times y$ = **207113** mm^2
Minimum area of reinforcement required - exp.7.1	$A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s$ = **538** mm^2

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

Quasi-permanent moment	$M_{QP} = \max(\beta_1 \times \text{abs}(M_{m1\_s1\_z2\_pos\_quasi}), \text{abs}(M_{m1\_s1\_z1\_neg\_quasi}))$ = **151.3**kNm
Permanent load ratio	$R_{PL} = M_{QP} / M$ = **0.65**
Service stress in reinforcement	$\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL}$ = **132** 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	$s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_t\_L1} \times N_{m1\_s1\_z1\_t\_L1})) / (N_{m1\_s1\_z1\_t\_L1} - 1)$ = **44.9** mm
Minimum allowable top bar spacing	$s_{top,min} = \max(\phi_{m1\_s1\_z1\_t\_L1} \times k_{s1}, h_{agg} + k_{s2}, 20\text{mm})$ = **25.0** mm

PASS - Actual bar spacing exceeds minimum allowable

Bottom bar spacing $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z1\_v}) + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / (N_{m1\_s1\_z1\_b\_L1} - 1) = \mathbf{82.8}$ mm

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

PASS - Actual bar spacing exceeds minimum allowable

Zone 2 (1200 mm - 6800 mm) Positive moment - section 6.1

Design bending moment $M = abs(M_{m1\_s1\_z2\_max\_red}) = \mathbf{926.5}$ kNm

Effective depth of tension reinforcement $d = \mathbf{570}$ mm

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

$K = M / (b \times d^2 \times f_{ck}) = \mathbf{0.137}$

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

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d) = \mathbf{489}$ mm

Depth of neutral axis $x = 2 \times (d - z) / \lambda = \mathbf{201}$ mm

Area of tension reinforcement required $A_{s,req} = M / (f_{yd} \times z) = \mathbf{4356}$ mm^2

Tension reinforcement provided Layer 1 - 6 × 25ϕ, Spacing - 25mm, Layer 2 - 6 × 25ϕ

Area of tension reinforcement provided $A_{s,prov} = \mathbf{5890}$ mm^2

Minimum area of reinforcement - exp.9.1N $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d = \mathbf{582}$ mm^2

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{s,max} = 0.04 \times b \times h = \mathbf{16900}$ 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 $f_{ct,eff} = f_{ctm} = \mathbf{3.0}$ N/mm^2

Stress distribution coefficient $k_c = \mathbf{0.4}$

Non-uniform self-equilibrating stress coefficient $k = \min(\max(1 + (300\text{ mm} - \min(h, b)) \times 0.35 / 500\text{ mm}, 0.65), 1) = \mathbf{0.76}$

Actual tension bar spacing $s_{bar} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2\_b\_L1} \times N_{m1\_s1\_z2\_b\_L1} + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / ((N_{m1\_s1\_z2\_b\_L1} + N_{m1\_s1\_z1\_b\_L1}) - 1) + \phi_{m1\_s1\_z2\_b\_L1} = \mathbf{107.8}$ mm

Maximum stress permitted - Table 7.3N $\sigma_s = \mathbf{314}$ N/mm^2

Steel to concrete modulus of elast. ratio $\alpha_{cr} = E_s / E_{cm} = \mathbf{6.00}$

Distance of the Elastic NA from bottom of beam $y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1)) = \mathbf{309}$ mm

Area of concrete in the tensile zone $A_{ct} = b \times y = \mathbf{200898}$ mm^2

Minimum area of reinforcement required - exp.7.1 $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s = \mathbf{585}$ mm^2

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

Quasi-permanent moment $M_{QP} = abs(M_{m1_s1_z2_pos_quasi}) = \mathbf{605.2}$kNm

Permanent load ratio $R_{PL} = M_{QP} / M = \mathbf{0.65}$

Service stress in reinforcement $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL} = \mathbf{210}$ N/mm^2

Maximum bar spacing - Tables 7.3N $s_{bar,max} = \mathbf{237.5}$ mm

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

Deflection control - Section 7.4

Reference reinforcement ratio $\rho_{m0} = (f_{ck} / 1\text{ N/mm}^2)^{0.5} / 1000 = \mathbf{0.00566}$

Required tension reinforcement ratio $\rho_m = A_{s,req} / (b \times d) = \mathbf{0.01177}$

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 — $span\_to\_depth_{basic} = K_b \times [11 + 1.5 \times (f_{ck} / 1\ N/mm^2)^{0.5} \times \rho_{m0} / (\rho_m - \rho'_m) + (f_{ck} / 1\ N/mm^2)^{0.5} \times (\rho'_m / \rho_{m0})^{0.5} / 12] =$ **15.079**

Reinforcement factor - exp.7.17 — $K_s = \min(A_{s,prov} / A_{s,req} \times 500\ N/mm^2 / f_{yk}, 1.5) =$ **1.352**

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

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

Allowable span to depth ratio — $span\_to\_depth_{allow} = \min(span\_to\_depth_{basic} \times K_s \times F1 \times F2, 40 \times K_b) =$ **20.389**

Actual span to depth ratio — $span\_to\_depth_{actual} = L_{m1\_s1} / d =$ **14.047**

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

Minimum bar spacing (Section 8.2)

Top bar spacing — $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2\_t\_L1} \times N_{m1\_s1\_z2\_t\_L1})) / (N_{m1\_s1\_z2\_t\_L1} - 1) =$ **44.9** mm

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

PASS - Actual bar spacing exceeds minimum allowable

Bottom bar spacing — $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z2\_v}) + \phi_{m1\_s1\_z2\_b\_L1} \times N_{m1\_s1\_z2\_b\_L1} + \phi_{m1\_s1\_z1\_b\_L1} \times N_{m1\_s1\_z1\_b\_L1})) / ((N_{m1\_s1\_z2\_b\_L1} + N_{m1\_s1\_z1\_b\_L1}) - 1) =$ **82.8** mm

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

PASS - Actual bar spacing exceeds minimum allowable

Zone 3 (6800 mm - 8000 mm) Positive moment - section 6.1

Design bending moment — $M = abs(M_{m1\_s1\_z3\_max\_red}) =$ **472.5** kNm

Effective depth of tension reinforcement — $d =$ **595** mm

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

$K = M / (b \times d^2 \times f_{ck}) =$ **0.064**

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

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d) =$ **559** mm

Depth of neutral axis — $x = 2 \times (d - z) / \lambda =$ **90** mm

Area of tension reinforcement required — $A_{s,req} = M / (f_{yd} \times z) =$ **1945** mm²

Tension reinforcement provided — $6 \times 25\phi$

Area of tension reinforcement provided — $A_{s,prov} =$ **2945** mm²

Minimum area of reinforcement - exp.9.1N — $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d =$ **608** mm²

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{s,max} = 0.04 \times b \times h =$ **16900** 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/mm²

Mean value of concrete tensile strength — $f_{ct,eff} = f_{ctm} =$ **3.0** N/mm²

Stress distribution coefficient — $k_c =$ **0.4**

Non-uniform self-equilibrating stress coefficient — $k = \min(\max(1 + (300\ mm - \min(h, b)) \times 0.35 / 500\ mm, 0.65), 1) =$ **0.76**

Actual tension bar spacing $s_{bar} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_b\_L1} \times N_{m1\_s1\_z3\_b\_L1})) / (N_{m1\_s1\_z3\_b\_L1} - 1) + \phi_{m1\_s1\_z3\_b\_L1}$ = **107.8** mm

Maximum stress permitted - Table 7.3N σ_s = **314** N/mm^2

Steel to concrete modulus of elast. ratio $\alpha_{cr} = E_s / E_{cm}$ = **6.00**

Distance of the Elastic NA from bottom of beam $y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1))$ = **316** mm

Area of concrete in the tensile zone $A_{ct} = b \times y$ = **205352** mm^2

Minimum area of reinforcement required - exp.7.1 $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s$ = **598** mm^2

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

Quasi-permanent moment $M_{QP} = \max(\beta_1 \times abs(M_{m1\_s1\_z2\_neg\_quasi}), abs(M_{m1\_s1\_z3\_pos\_quasi}))$ = **308.6**kNm

Permanent load ratio $R_{PL} = M_{QP} / M$ = **0.65**

Service stress in reinforcement $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL}$ = **188** N/mm^2

Maximum bar spacing - Tables 7.3N $s_{bar,max}$ = **265.5** mm

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

Zone 3 (6000 mm - 8000 mm) Negative moment - section 6.1

Design bending moment $M = \max(\beta_1 \times abs(M_{m1\_s1\_max\_red}), abs(M_{m1\_s1\_z3\_min\_red}))$ = **231.6** kNm

Effective depth of tension reinforcement d = **599** mm

Redistribution ratio $\delta = 1$ = **1.000**

$K = M / (b \times d^2 \times f_{ck})$ = **0.031**

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

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 \times d \times [1 + (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}], 0.95 \times d)$ = **569** mm

Depth of neutral axis $x = 2 \times (d - z) / \lambda$ = **75** mm

Area of tension reinforcement required $A_{s,req} = M / (f_{yd} \times z)$ = **936** mm^2

Tension reinforcement provided $10 \times 16\phi$

Area of tension reinforcement provided $A_{s,prov}$ = **2011** mm^2

Minimum area of reinforcement - exp.9.1N $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d$ = **612** mm^2

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{s,max} = 0.04 \times b \times h$ = **16900** 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 $f_{ct,eff} = f_{ctm}$ = **3.0** N/mm^2

Stress distribution coefficient k_c = **0.4**

Non-uniform self-equilibrating stress coefficient $k = \min(\max(1 + (300 \text{ mm} - \min(h, b)) \times 0.35 / 500 \text{ mm}, 0.65), 1)$ = **0.76**

Actual tension bar spacing $s_{bar} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_t\_L1} \times N_{m1\_s1\_z3\_t\_L1})) / (N_{m1\_s1\_z3\_t\_L1} - 1) + \phi_{m1\_s1\_z3\_t\_L1}$ = **60.9** mm

Maximum stress permitted - Table 7.3N σ_s = **351** N/mm^2

Steel to concrete modulus of elast. ratio $\alpha_{cr} = E_s / E_{cm}$ = **6.00**

Distance of the Elastic NA from bottom of beam $y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1))$ = **319** mm

Area of concrete in the tensile zone $A_{ct} = b \times y$ = **207113** mm^2

Minimum area of reinforcement required - exp.7.1 $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s$ = **538** mm²

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

Quasi-permanent moment $M_{QP} = \max(\beta_1 \times \text{abs}(M_{m1\_s1\_z2\_pos\_quasi}), \text{abs}(M_{m1\_s1\_z3\_neg\_quasi}))$ = **151.3**kNm

Permanent load ratio $R_{PL} = M_{QP} / M$ = **0.65**

Service stress in reinforcement $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL}$ = **132** N/mm²

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 $s_{top} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_t\_L1} \times N_{m1\_s1\_z3\_t\_L1})) / (N_{m1\_s1\_z3\_t\_L1} - 1)$ = **44.9** 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

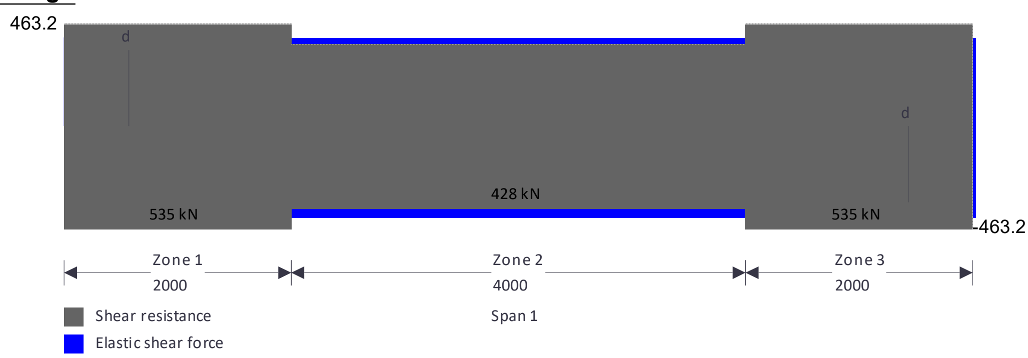
PASS - Actual bar spacing exceeds minimum allowable

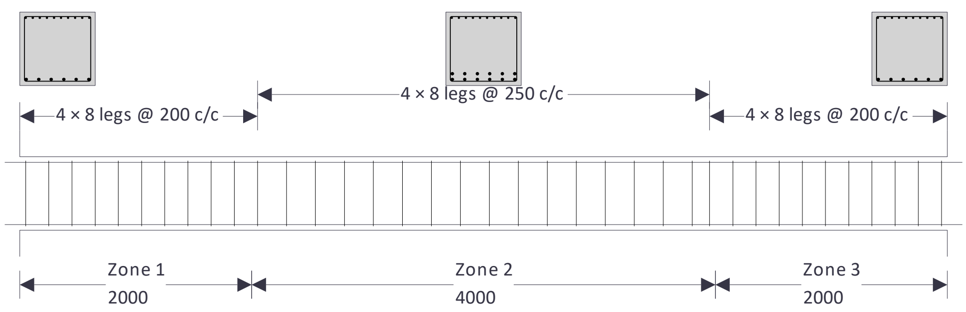
Bottom bar spacing $s_{bot} = (b - (2 \times (c_{nom\_s} + \phi_{m1\_s1\_z3\_v}) + \phi_{m1\_s1\_z3\_b\_L1} \times N_{m1\_s1\_z3\_b\_L1})) / (N_{m1\_s1\_z3\_b\_L1} - 1)$ = **82.8** mm

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) $v_1 = 0.6 \times (1 - f_{ck} / 250 \text{ N/mm}^2)$ = **0.523**

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

Minimum area of shear reinforcement - exp.9.5N $A_{sv,min} = 0.08 \text{ N/mm}^2 \times b \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / f_{yk}$ = **588** mm²/m

Zone 1 (0 mm - 2000 mm) shear - section 6.2

Design shear force at support $V_{Ed,max} = \max(\text{abs}(V_{z1\_max}), \text{abs}(V_{z1\_red\_max}))$ = **463** kN

Min lever arm in shear zone z = **489** mm

Maximum design shear resistance - exp.6.9 $V_{Rd,max} = \alpha_{cw} \times b \times z \times v_1 \times f_{cwd} / (\cot(\theta_{max}) + \tan(\theta_{max}))$ = **1774** kN

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Calculation for:		Job No.
25-26 Redington Gardens		180709
		Date: Jan 19
Calculation by: KSC	Checked by:	Sheet No. B21

Beam along GL 5

Loading

DL = 156.5 kN/m
LL = 42.5 kN/m } Sheet L2 Exc Basement.

Wall DL = 0.24 × 24 × 10.2 = 58.7 kN/m

Factored Load =

Design for DL = 215 kN, LL = 45 kN

Factored Load = 215 × 1.35 + 45 × 1.5 = 357 kN/m

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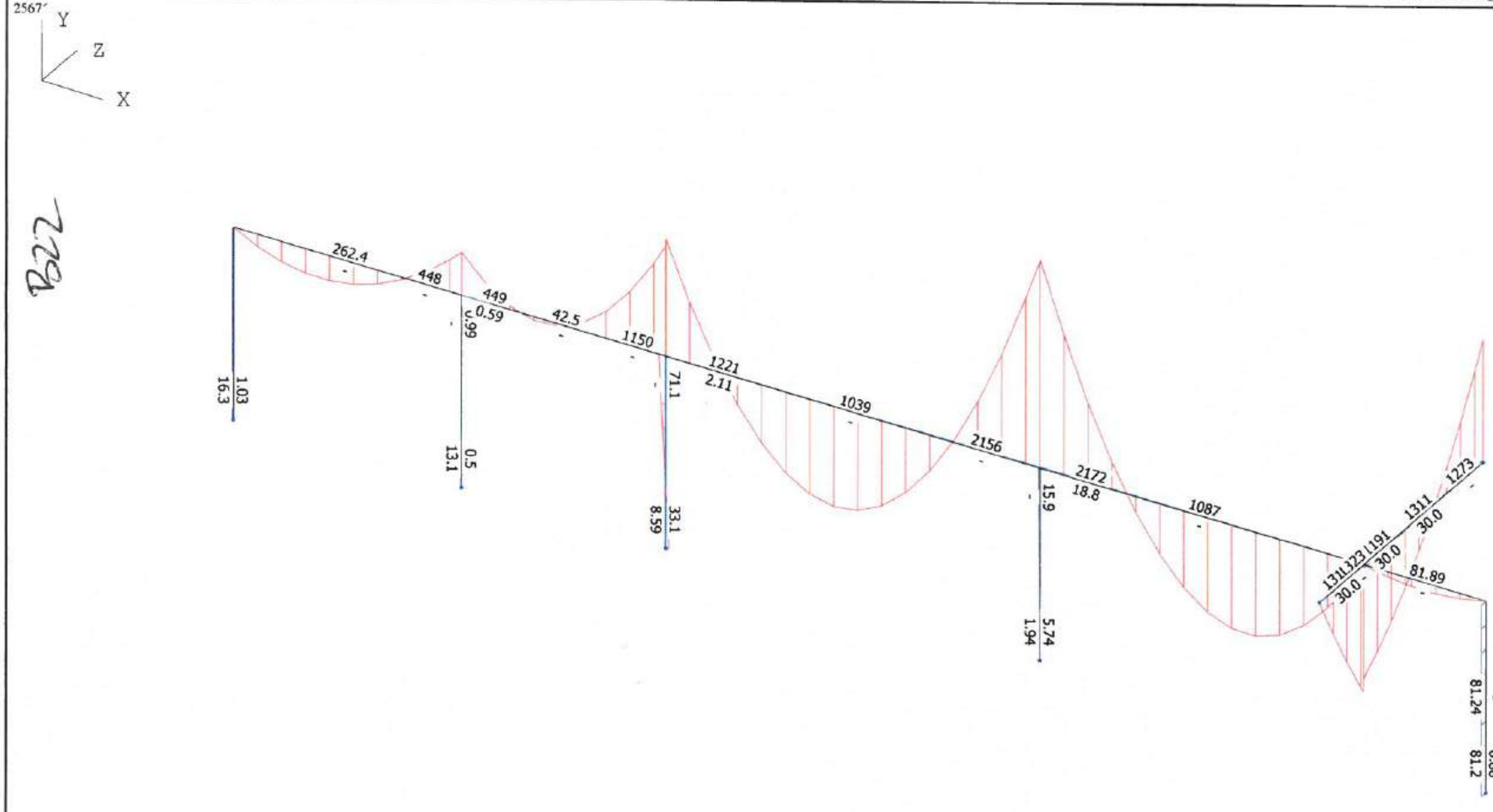
B22



Load Case 001 : Dead plus Live b (Ultimate)
Bending Moment Diagram - (Full Frame) - 3D Front View
Bending Moment Values (kN.m)

Not to Scale

500 kN.m = 1m

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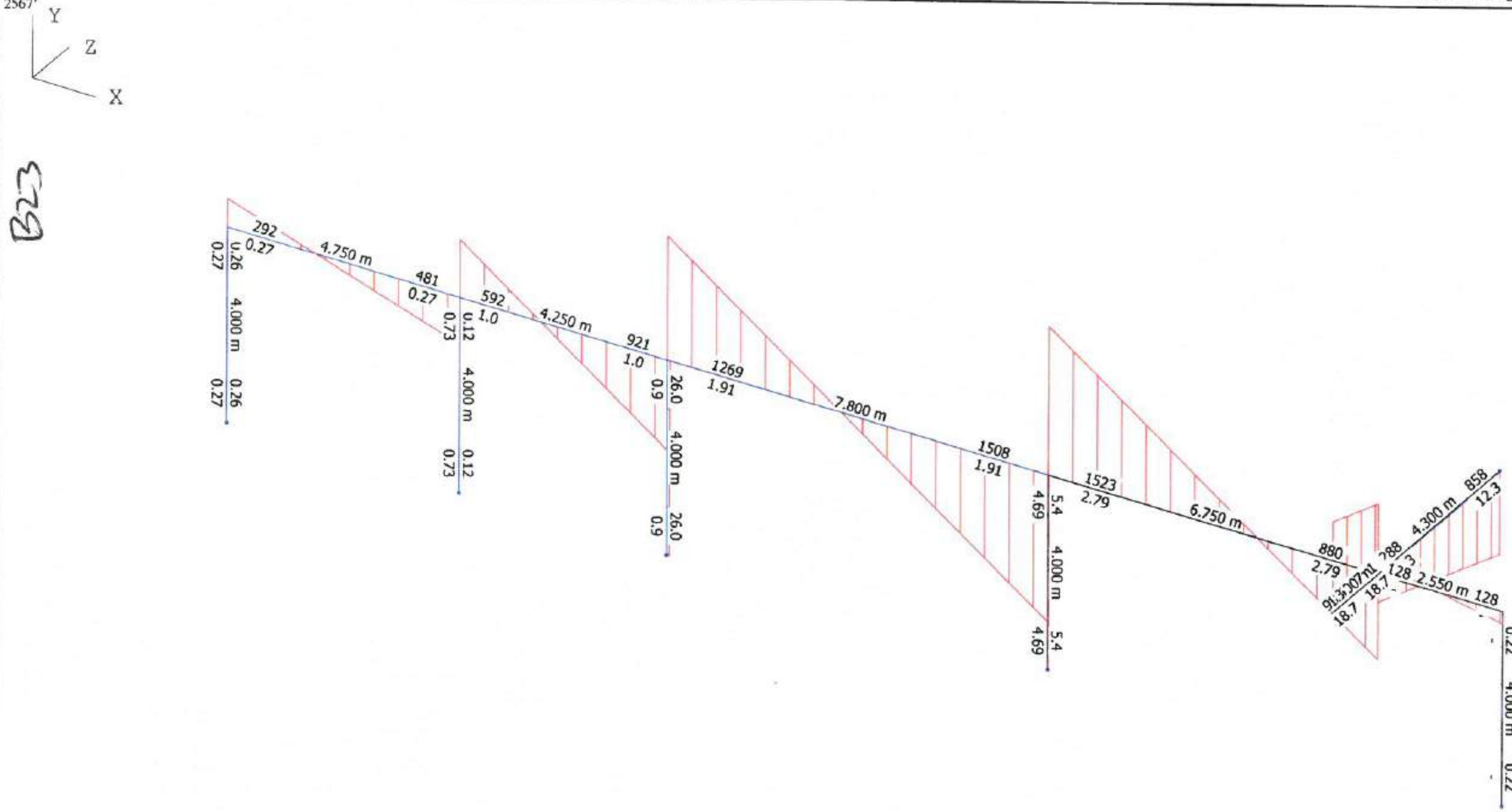
B23



Not to Scale

Load Case 001 : Dead plus Live b (Ultimate)
Shear Force Diagram - (Full Frame) - 3D Front View
Shear Force Values (kN)

500 kN = 1m

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B24

Load Diagram - Load Case 001 - All Groups - Y Loads - Including Automatic Selfweight (Density)
Frame Geometry - (Full Frame) - 3D Front View

Not to Scale

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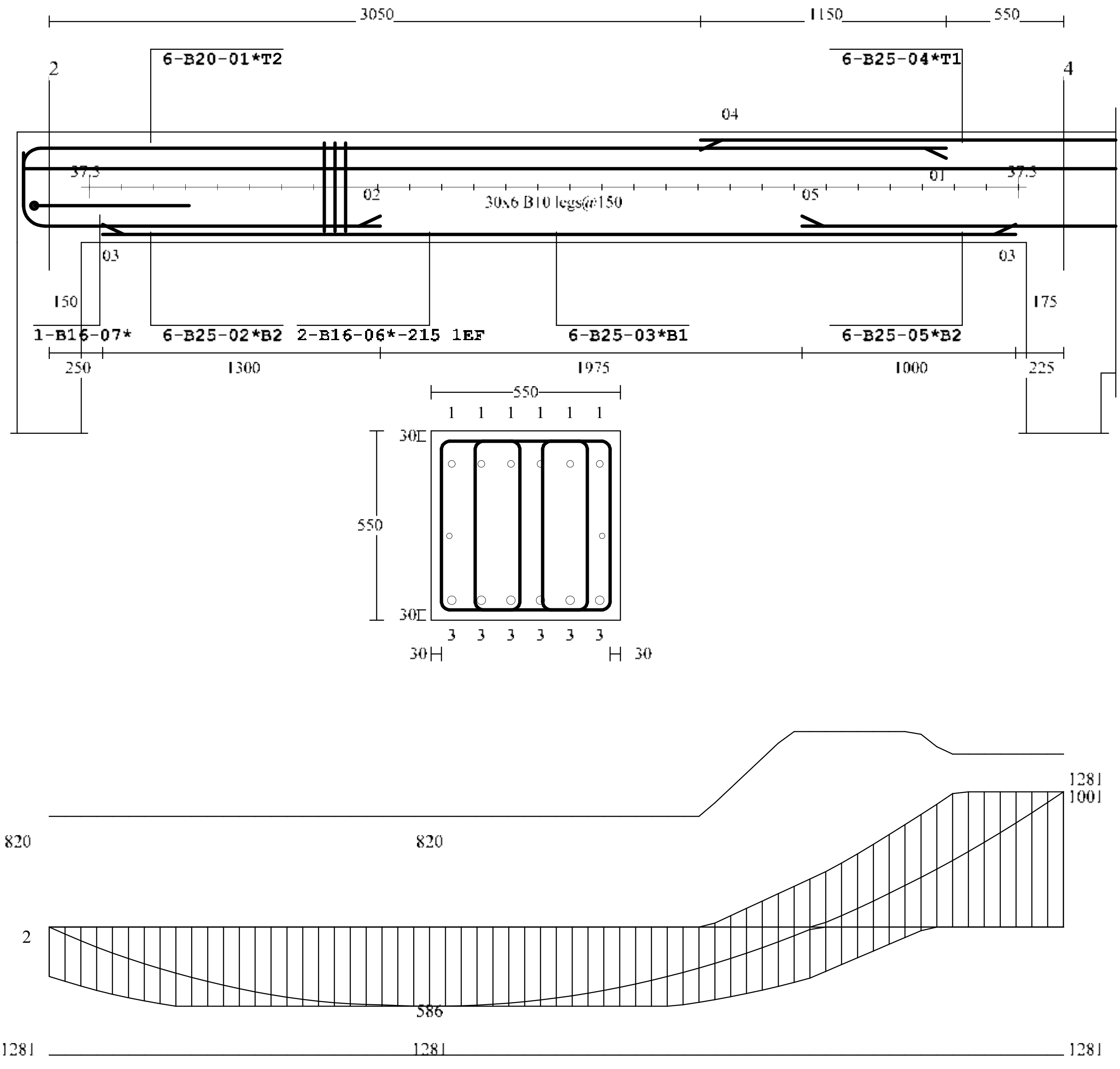
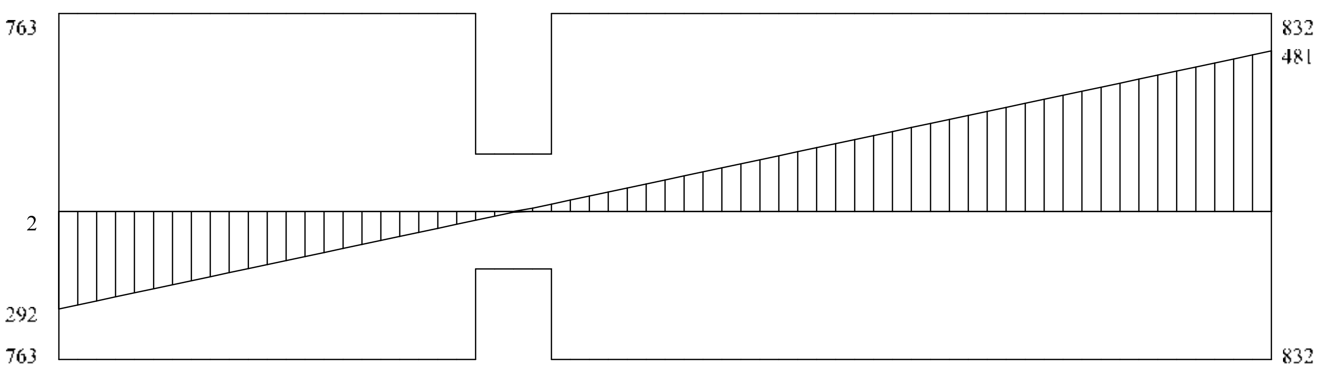


Tension Force Envelope (EC2 Fig 9.2)



Shear Force Envelope

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Member CBL1Id 1

Basic Data

Design to	EC 2: 2004 - Using UK values		
f_{ck}, f_{yk}, γ_c, γ_s, η, λ	C32/40, 500, 1.5, 1.15, 1.0, 0.8		
f_{ykv}, $v crit	500, 0.1		

Torsional Design

Ak = Fn(A, u, B, H, Cvr_{max}, t)	302500, 2200, 550, 550, 53, 105.00	198025 mm²	
$T_{Rd\text{-}c}$= f_{ctd}• t • 2 • A_k	1.41 • 105.00 • 2 • 198025	58.7 kN.m	
$T_{Rd\text{-}max}$= 2•v•α_{cw}•f_{cd}•A_k•t•Sinθ•Cosθ	2•0.52•1.00•18.13•198025•105.0•0.89•0.45	157.8 kN.m	
Unity T_{Ed}/ T_{Rd}.c + V_{Ed}/ $V_{Rd\text{-}c}$	15.18/58.68 + 481.10/172.21	3.05	Design Torsion
A_{sl}= T_{Ed}/ A_k /2 • Cotθ • u_k / (F_{ywk}/γ_s)	15.18 / 198025 / 2 • 2.00 • 1780/(500 / 1.15)	313.8 mm²	
$A_{s\ Per_n}$ = A_{sl}/ ($No._{Hor}$+ $No._{Ver}$) /2	313.8/ (3 + 1) /2	39.2 mm²	
A_{sr} deduction T+B = A_s• $No._{Hor}$	39.2 • 3	117.7 mm²	
q = T / A_k / 2	15.2 / 198025/2	38.3 N/mm	
A_s/$S_v$reduction = (q •γ_s)/(Cotθ •F_{ywk}) • 2	(38.3 • 1.15) / (2.00 • 500) • 2	0.088 mm	
$A_s$prov	min(1885, 2945, 2945, 2945, 2945)	1885	OK

Bending Moments

In-Span Steel @ 1781 mm. Sagging

A_sRequired to EC2 Cl 3.1.7, Fig 3.5	Assuming d = 496, d' = 54	top 118, bottom 1408 mm²

Right Support Steel Hogging

A_sRequired to EC2 Cl 3.1.7, Fig 3.5	Assuming d = 496, d' = 54	top 2432, bottom 118 mm²

Shear

Max Shear

V_{Rd}.max = 0.5 • B_w• d • v • f_{cd}	0.5•550•456.5•0.523•18	1191.0 kN	
V_{Ed}.max		444.8 kN	OK

Nominal Shear Zone at 151 mm

V_{Rd}.c.a=Fn(C_{rdc},K, A_{sl},f_{ck},K_1,σ_{cp},B_w,d)	0.12, 1.662, 2828, 32, 0.15, 0.0, 550, 456.5	165.4 kN	(6.2.a)
V_{Rd}.c.b=(V_{min}+ K_1.σ_{cp}) • B_w• d	(0.42 + 0.15 • 0.0) • 550.0 • 456.5)	106.5 kN	(6.2.b)
V_{app}/ max(V_{Rd}.c.a, V_{Rd}.c.b)	267.7 / Max(165.4, 106.5)	1.618	Links Req
V_{Rd}.s = A_{sw}/ S • Z • 0.8 • f_{ywk}• Cotθ	458 • 150.0 • 410.85 • 0.8 • 500.0 • 2.5	1253.9 kN	(6.8)
V_{Rd}.max =α_{cw}• b_w•z•v_1•f_{cd}/(cotθ +tanθ)	1 • 550 • 410.85 • 0.54 • 18.1 / (2.5 + 0.4)	763.0 kN	(6.9)
V_{app}/ max(V_{Rd}.s, V_{Rd}.max)	267.7 / Min(1253.9, 763.0)	0.351	OK

Nominal Shear Zone at 4574 mm

V_{Rd}.c.a=Fn(C_{rdc},K, A_{sl},f_{ck},K_1,σ_{cp},B_w,d)	0.12, 1.634, 2828, 32, 0.15, 0.0, 550, 497.5	172.2 kN	(6.2.a)
V_{Rd}.c.b=(V_{min}+ K_1.σ_{cp}) • B_w• d	(0.41 + 0.15 • 0.0) • 550.0 • 497.5)	113.2 kN	(6.2.b)
V_{app}/ max(V_{Rd}.c.a, V_{Rd}.c.b)	452.5 / Max(172.2, 113.2)	2.628	Links Req
V_{Rd}.s = A_{sw}/ S • Z • 0.8 • f_{ywk}• Cotθ	458 • 150.0 • 447.75 • 0.8 • 500.0 • 2.5	1366.5 kN	(6.8)
V_{Rd}.max =α_{cw}• b_w•z•v_1•f_{cd}/(cotθ +tanθ)	1 • 550 • 447.75 • 0.54 • 18.1 / (2.5 + 0.4)	831.5 kN	(6.9)
V_{app}/ max(V_{Rd}.s, V_{Rd}.max)	452.5 / Min(1366.5, 831.5)	0.544	OK

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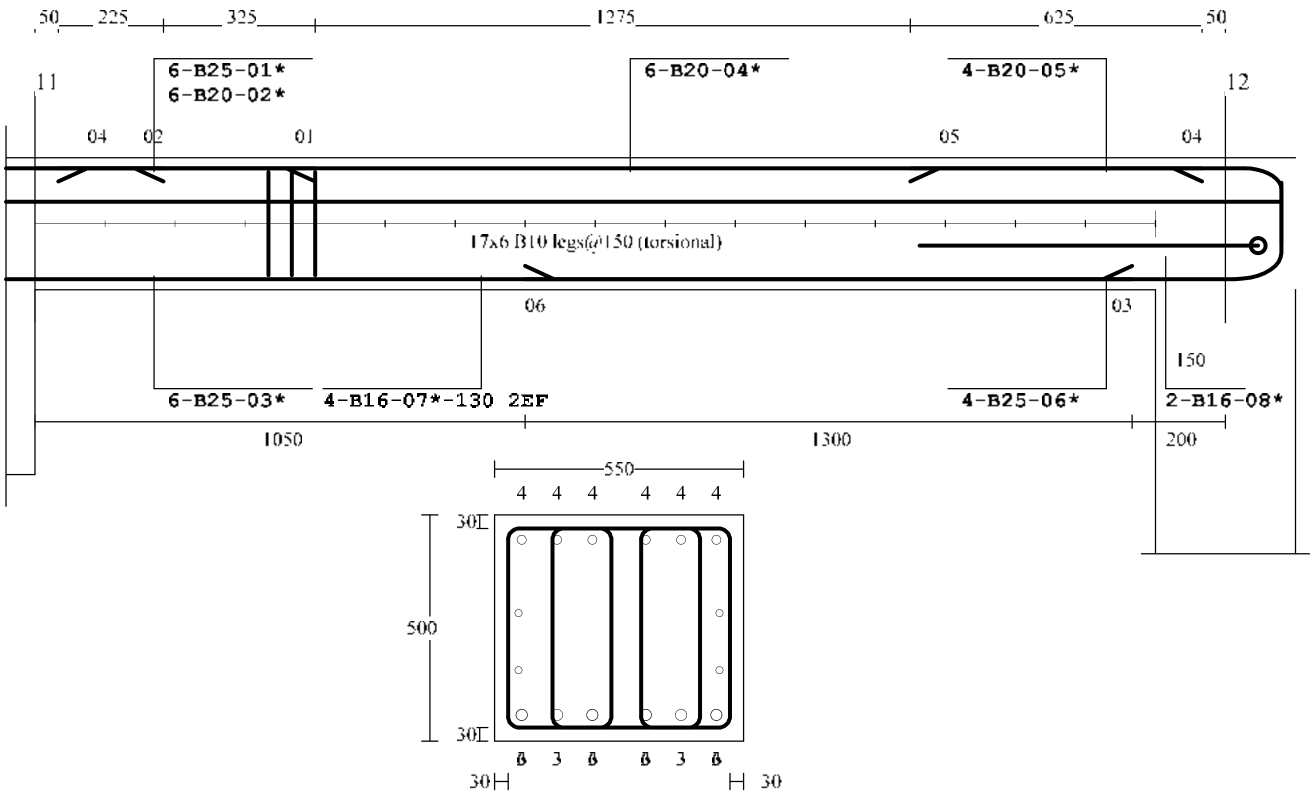
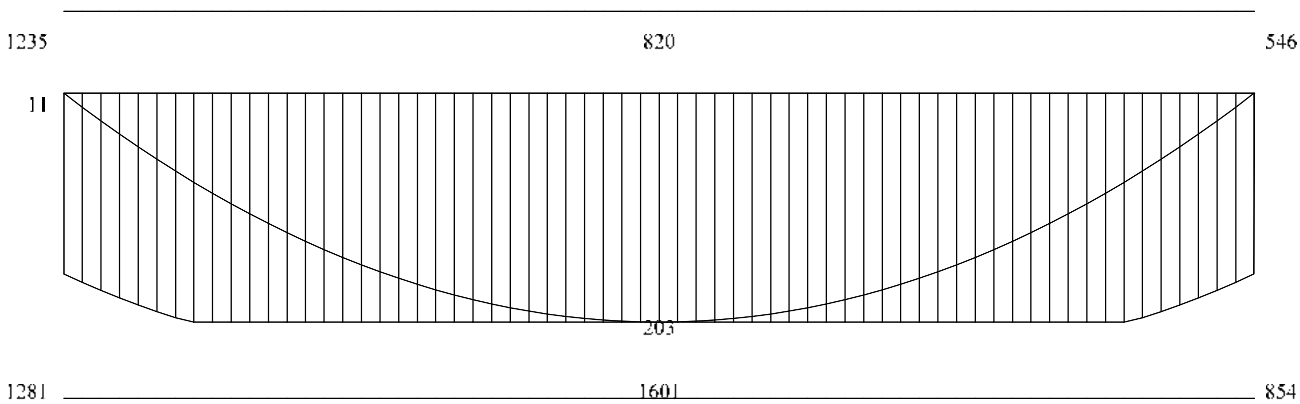
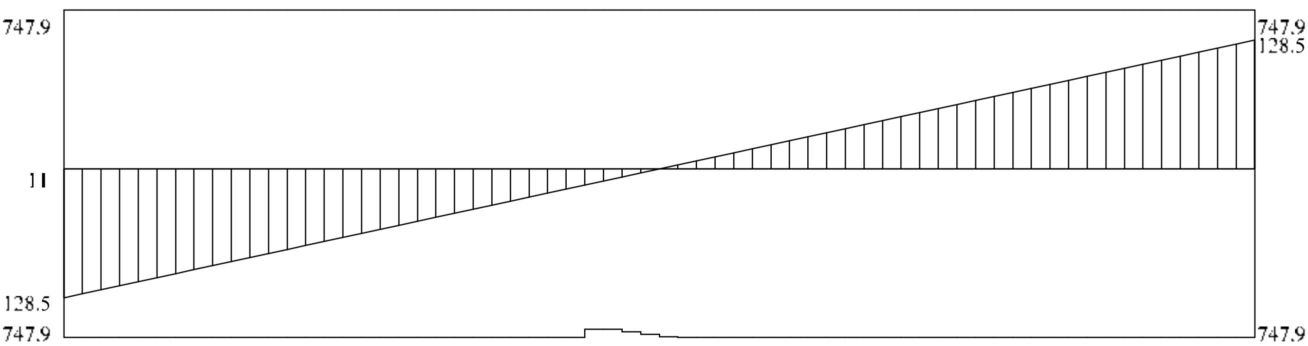




Tension Force Envelope (EC2 Fig 9.2)



Shear Force Envelope

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Member CBL1Id 10

Basic Data

Design to	EC 2: 2004 - Using UK values		
$f_{ck}, f_{yk}, \gamma_c, \gamma_s, \eta, \lambda$	C32/40, 500, 1.5, 1.15, 1.0, 0.8		
f_{ykv}, $v crit	500, 0.1		

Torsional Design

Ak = Fn(A, u, B, H, Cvr_{max}, t)	275000, 2100, 550, 500, 53, 105.00	175775 mm²	
$T_{Rd\text{-}c} = f_{ctd} \cdot t \cdot 2 \cdot A_k$	1.41 • 105.00 • 2 • 175775	52.1 kN.m	
$T_{Rd\text{-}max} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_k \cdot t \cdot Sin\theta \cdot Cos\theta$	2•0.52•1.00•18.13•175775•105.0•0.89•0.45	140.1 kN.m	
Unity $T_{Ed} / T_{Rd}.c + V_{Ed} / V_{Rd\text{-}c}$	81.24/52.09 + 128.46/167.34	2.33	Design Torsion
$A_{sl} = T_{Ed} / A_k / 2 \cdot Cot\theta \cdot u_k / (F_{ywk}/\gamma_s)$	81.24 / 175775 / 2 • 2.00 • 1680/(500 / 1.15)	1785.9 mm²	
$A_{s\ Per_n} = A_{sl} / (No._{Hor} + No._{Vert}) / 2$	1785.9/ (3 – 1) /2	223.2 mm²	
A_{sr} deduction T+B = $A_s \cdot No._{Hor}$	223.2 • 3	669.7 mm²	
$q = T / A_k / 2$	81.2 / 175775/2	231.1 N/mm	
A_s/S_vreduction = $(q \cdot \gamma_s)/(Cot\theta \cdot F_{ywk}) \cdot 2$	(231.1 • 1.15) / (2.00 • 500) • 2	0.532 mm	
$A_s$prov	min(4830, 2945, 1885, 1257, 1963)	1257	OK
A_{sv}/S_v prov (2 external legs only)	min(1.05)	1.05	OK

Bending Moments

In-Span Steel @ 1275 mm. Sagging

A_sRequired to EC2 Cl 3.1.7, Fig 3.5	Assuming d = 446, d' = 54	top 670, bottom 1101 mm²

Shear

Max Shear

$V_{Rd}.max = 0.5 \cdot B_w \cdot d \cdot v \cdot f_{cd}$	0.5•550•447.5•0.523•18	1167.5 kN	
$V_{Ed}.max$		128.5 kN	OK

Nominal Shear Zone at 1 mm

$V_{Rd}.c.a = Fn(C_{rdc}, K, A_{sl}, f_{ck}, K_1, \sigma_{cp}, B_w, d)$	0.12, 1.669, 2276, 32, 0.15, 0.0, 550, 447.5	152.4 kN	(6.2.a)
$V_{Rd}.c.b = (V_{min} + K_1.\sigma_{cp}) \cdot B_w \cdot d$	(0.43 + 0.15 • 0.0) • 550.0 • 447.5)	105.0 kN	(6.2.b)
$V_{app} / max(V_{Rd}.c.a, V_{Rd}.c.b)$	128.3 / Max(152.4, 105.0)	0.842	no Links req
$V_{Rd}.s = A_{sw} / S \cdot Z \cdot 0.8 \cdot f_{ywk} \cdot Cot\theta$	391 • 150.0 • 402.75 • 0.8 • 500.0 • 2.5	1050.6 kN	(6.8)
$V_{Rd}.max = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd} / (cot\theta + tan\theta)$	1 • 550 • 402.75 • 0.54 • 18.1 / (2.5 + 0.4)	747.9 kN	(6.9)
$V_{app} / max(V_{Rd}.s, V_{Rd}.max)$	128.3 / Min(1050.6, 747.9)	0.172	OK

Nominal Shear Zone at 2399 mm

$V_{Rd}.c.a = Fn(C_{rdc}, K, A_{sl}, f_{ck}, K_1, \sigma_{cp}, B_w, d)$	0.12, 1.669, 1294, 32, 0.15, 0.0, 550, 447.5	126.3 kN	(6.2.a)
$V_{Rd}.c.b = (V_{min} + K_1.\sigma_{cp}) \cdot B_w \cdot d$	(0.43 + 0.15 • 0.0) • 550.0 • 447.5)	104.9 kN	(6.2.b)
$V_{app} / max(V_{Rd}.c.a, V_{Rd}.c.b)$	113.3 / Max(126.3, 104.9)	0.897	no Links req
$V_{Rd}.s = A_{sw} / S \cdot Z \cdot 0.8 \cdot f_{ywk} \cdot Cot\theta$	391 • 150.0 • 402.75 • 0.8 • 500.0 • 2.5	1050.6 kN	(6.8)
$V_{Rd}.max = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd} / (cot\theta + tan\theta)$	1 • 550 • 402.75 • 0.54 • 18.1 / (2.5 + 0.4)	747.9 kN	(6.9)
$V_{app} / max(V_{Rd}.s, V_{Rd}.max)$	113.3 / Min(1050.6, 747.9)	0.151	OK

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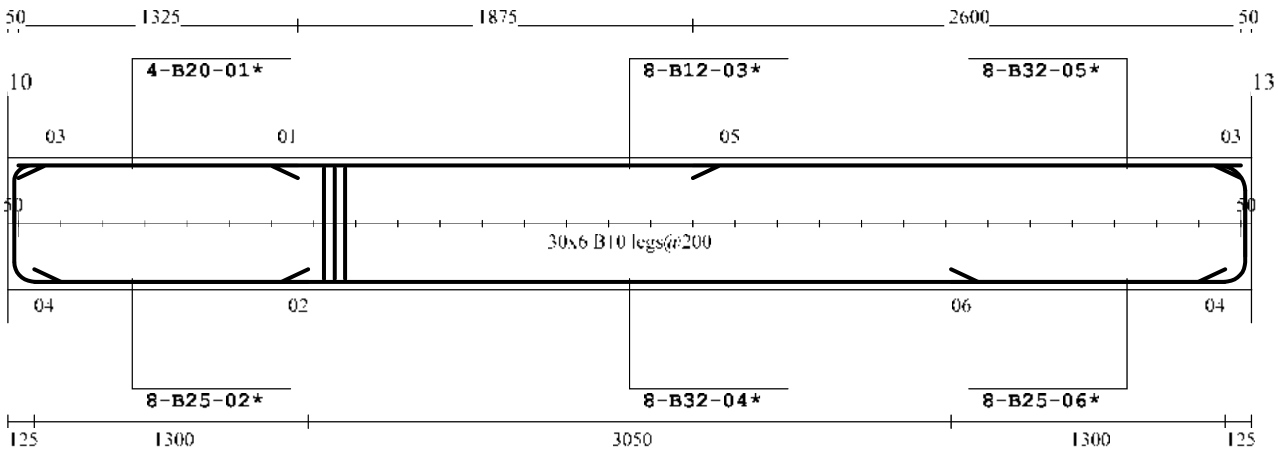
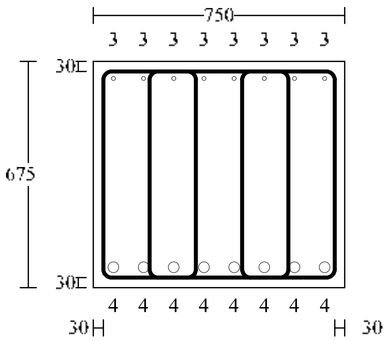
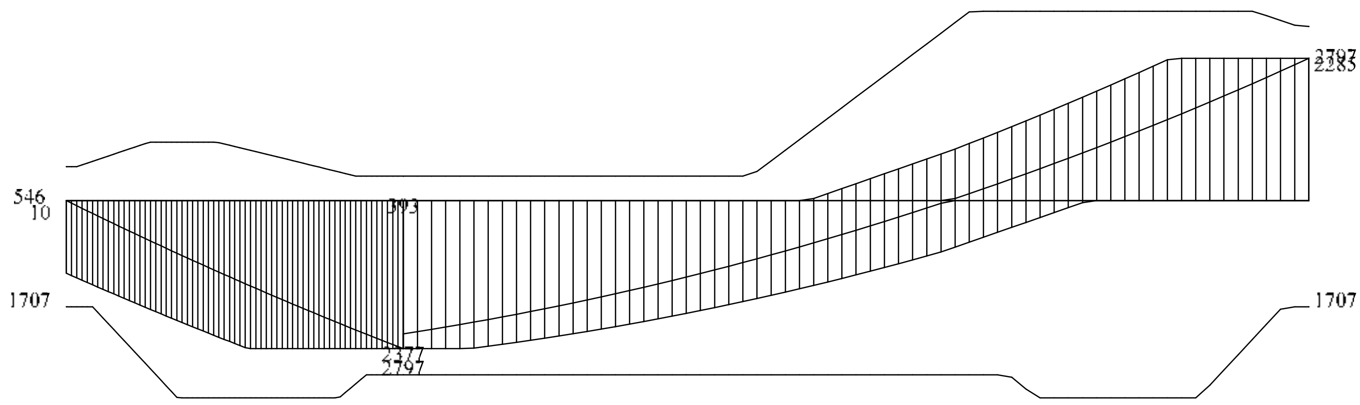
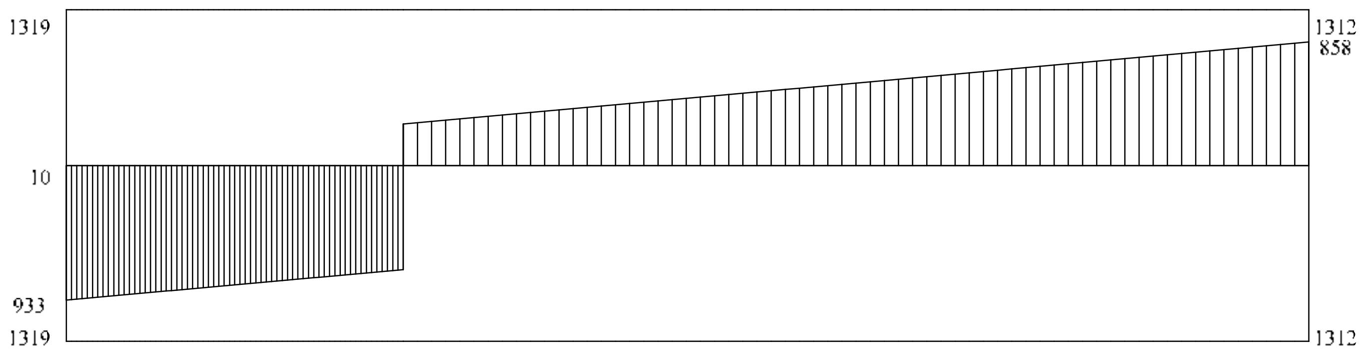






Tension Force Envelope (EC2 Fig 9.2)



Shear Force Envelope

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Basic Data

Design to	EC 2: 2004 - Using UK values
$f_{ck}, f_{yk}, \gamma_c, \gamma_s, \eta, \lambda$	C32/40, 500, 1.5, 1.15, 1.0, 0.8
f_{ykv}, $v crit	500, 0.1

Minor Axis Moments

My max = 30.5 kN.m	Minor axis moments ignored by user

Bending Moments

In-Span Steel @ 1599 mm. Sagging

A_s'Required to EC2 Cl 3.1.7, Fig 3.5	Assuming d = 621, d' = 54	bottom 5757 mm²

Right Support Steel Hogging

A_s'Required to EC2 Cl 3.1.7, Fig 3.5	Assuming d = 621, d' = 54	top 5492

Shear

Max Shear

V_{Rd},max = 0.5 • B_w• d • v • f_{cd}	0.5•750•619.0•0.523•18	2202.3 kN	
V_{Ed},max		933.1 kN	OK

Nominal Shear Zone at 1 mm

V_{Rd},c.a=Fn(C_{rdc},K, A_{sl},f_{ck},K_1,σ_{cp},B_w,d)	0.12, 1.567, 3927, 32, 0.15, 0.0, 750, 622.5	263.1 kN	(6.2.a)
V_{Rd},c.b=(V_{min}+ K_1.σ_{cp}) • B_w• d	(0.39 + 0.15 • 0.0) • 750.0 • 622.5)	181.3 kN	(6.2.b)
V_{app}/ max(V_{Rd},c.a, V_{Rd},c.b)	932.9 / Max(263.1, 181.3)	3.546	Links Req
V_{Rd},s = A_{sw}/ S • Z • 0.8 • f_{ywk}• Cotθ	471 • 200.0 • 560.25 • 0.8 • 500.0 • 2.5	1319.4 kN	(6.8)
V_{Rd},max =α_{cw}• b_w•z•v_1•f_{cd}/(cotθ +tanθ)	1 • 750 • 560.25 • 0.54 • 18.1 / (2.5 + 0.4)	1418.8 kN	(6.9)
V_{app}/ max(V_{Rd},s, V_{Rd},max)	932.9 / Min(1319.4, 1418.8)	0.707	OK

Nominal Shear Zone at 5899 mm

V_{Rd},c.a=Fn(C_{rdc},K, A_{sl},f_{ck},K_1,σ_{cp},B_w,d)	0.12, 1.568, 6434, 32, 0.15, 0.0, 750, 619.0	309.3 kN	(6.2.a)
V_{Rd},c.b=(V_{min}+ K_1.σ_{cp}) • B_w• d	(0.39 + 0.15 • 0.0) • 750.0 • 619.0)	180.5 kN	(6.2.b)
V_{app}/ max(V_{Rd},c.a, V_{Rd},c.b)	858.3 / Max(309.3, 180.5)	2.775	Links Req
V_{Rd},s = A_{sw}/ S • Z • 0.8 • f_{ywk}• Cotθ	471 • 200.0 • 557.1 • 0.8 • 500.0 • 2.5	1312.0 kN	(6.8)
V_{Rd},max =α_{cw}• b_w•z•v_1•f_{cd}/(cotθ +tanθ)	1 • 750 • 557.1 • 0.54 • 18.1 / (2.5 + 0.4)	1410.8 kN	(6.9)
V_{app}/ max(V_{Rd},s, V_{Rd},max)	858.3 / Min(1312.0, 1410.8)	0.654	OK

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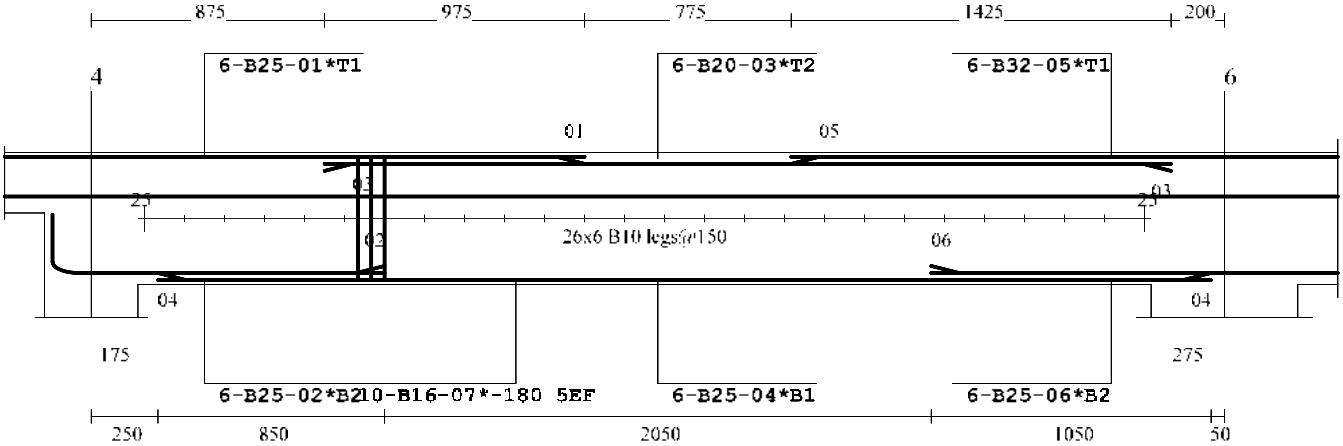
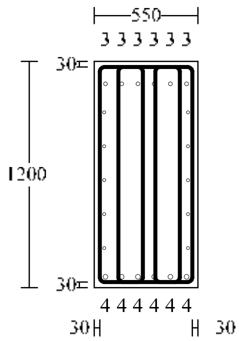
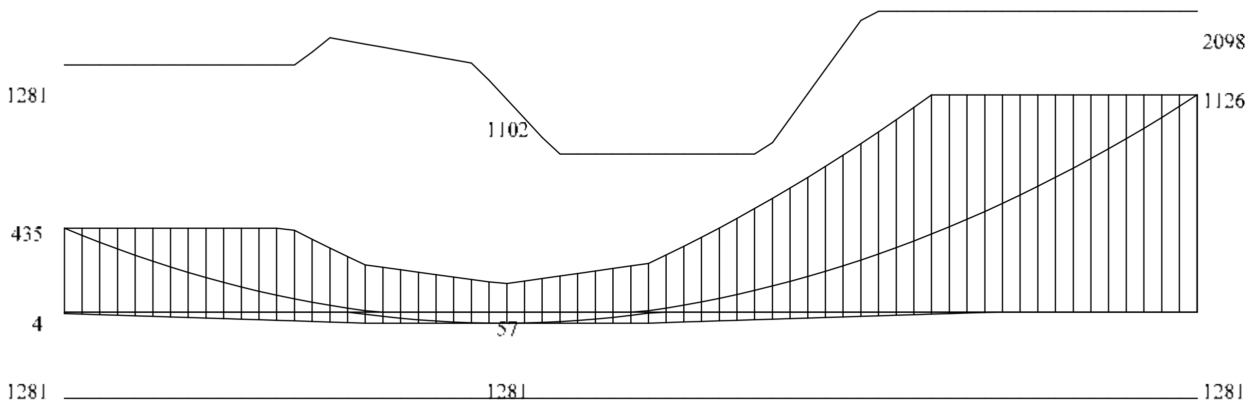
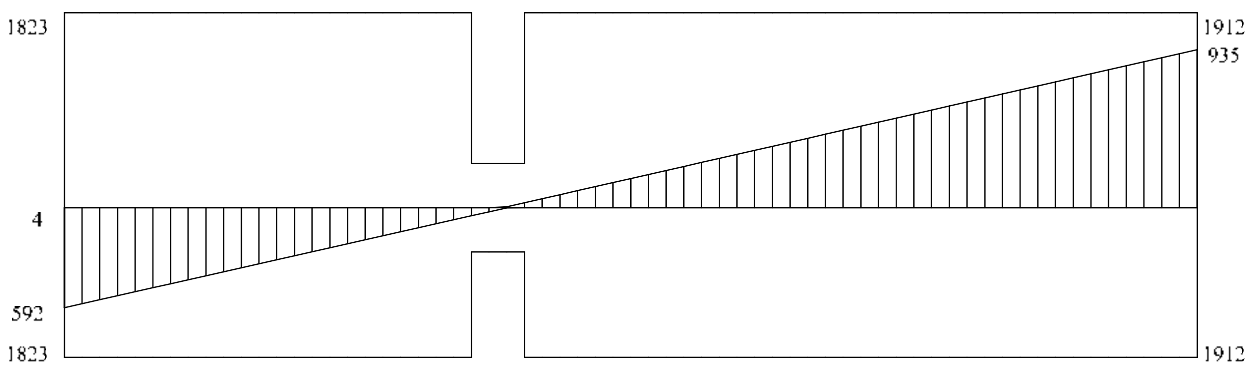
Member CBL1Id 2 @ Level 1







Tension Force Envelope (EC2 Fig 9.2)



Shear Force Envelope

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Member CBL1Id 2

Basic Data

Design to	EC 2: 2004 - Using UK values
f_{ck}, f_{yk}, γ_c, γ_s, η, λ	C32/40, 500, 1.5, 1.15, 1.0, 0.8
f_{ykv}, $v crit	500, 0.1

Minor Axis Moments

My max = 3.7 kN.m	Minor axis moments ignored by user

Torsional Design

Ak = Fn(A, u, B, H, Cvr_{max}, t)	660000, 3500, 550, 1200, 56, 112.00	476544 mm²	
$T_{Rd\text{-}c} = f_{ctd} \cdot t \cdot 2 \cdot A_k$	1.41 • 112.00 • 2 • 476544	150.6 kN.m	
$T_{Rd\text{-}max} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_k \cdot t \cdot \sin\theta \cdot \cos\theta$	2•0.52•1.00•18.13•476544•112.0•0.89•0.45	405.1 kN.m	
Unity $T_{Ed} / T_{Rd}.c + V_{Ed} / V_{Rd\text{-}c}$	25.35/150.63 + 935.02/261.54	3.74	Design Torsion
$A_{sl} = T_{Ed} / A_k / 2 \cdot \cot\theta \cdot u_k / (F_{ywk}/\gamma_s)$	25.35 / 476544 / 2 • 2.00 • 3052/(500 / 1.15)	373.4 mm²	
$A_{s\,Per_n} = A_{sl} / (No._{Horz} + No._{Vert}) / 2$	373.4/ (3 + 3) /2	31.1 mm²	
A_{sr} deduction T+B = $A_s \cdot No._{Horz}$	31.1 • 3	93.4 mm²	
$q = T / A_k / 2$	25.3 / 476544/2	26.6 N/mm	
A_s/S_vreduction = $(q \cdot \gamma_s)/(\cot\theta \cdot F_{ywk}) \cdot 2$	(26.6 • 1.15) / (2.00 • 500) • 2	0.061 mm	
$A_s$prov	min(2945, 2945, 1885, 2945, 4825, 2945)	1885	OK

Bending Moments

Left Support Steel Hogging

A'sRequired to EC2 Cl 3.1.7, Fig 3.5	Assuming d = 1146, d' = 54	top 1011, bottom 93 mm²

In-Span Steel @ 1660 mm. Sagging

A'sRequired to EC2 Cl 3.1.7, Fig 3.5	Assuming d = 1146, d' = 54	top 93, bottom 211 mm²

Right Support Steel Hogging

A'sRequired to EC2 Cl 3.1.7, Fig 3.5	Assuming d = 1146, d' = 54	top 2534, bottom 93 mm²

Shear

Max Shear

V_{Rd},max = $0.5 \cdot B_w \cdot d \cdot v \cdot f_{cd}$	0.5•550•1144.0•0.523•18	2984.7 kN	
V_{Ed},max		815.9 kN	OK

Nominal Shear Zone at 176 mm

V_{Rd},c.a=Fn(C_{rdc},K, A_{sl},f_{ck},K_1,σ_{cp},B_w,d)	0.12, 1.428, 2852, 32, 0.15, 0.0, 550, 1090.5	254.7 kN	(6.2.a)
V_{Rd},c.b=(V_{min}+ $K_1 \cdot \sigma_{cp}$) • B_w • d	(0.34 + 0.15 • 0.0) • 550.0 • 1090.5)	202.7 kN	(6.2.b)
V_{app}/ max(V_{Rd},c.a, V_{Rd},c.b)	529.0 / Max(254.7, 202.7)	2.077	Links Req
V_{Rd},s = A_{sw}/ S • Z • 0.8 • f_{ywk} • $\cot\theta$	462 • 150.0 • 981.45 • 0.8 • 500.0 • 2.5	3021.7 kN	(6.8)
V_{Rd},max = $\alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd} / (\cot\theta + \tan\theta)$	1 • 550 • 981.45 • 0.54 • 18.1 / (2.5 + 0.4)	1822.7 kN	(6.9)
V_{app}/ max(V_{Rd},s, V_{Rd},max)	529.0 / Min(3021.7, 1822.7)	0.29	OK

Nominal Shear Zone at 3974 mm

V_{Rd},c.a=Fn(C_{rdc},K, A_{sl},f_{ck},K_1,σ_{cp},B_w,d)	0.12, 1.419, 4949, 32, 0.15, 0.0, 550, 1141.5	313.4 kN	(6.2.a)
V_{Rd},c.b=(V_{min}+ $K_1 \cdot \sigma_{cp}$) • B_w • d	(0.33 + 0.15 • 0.0) • 550.0 • 1141.5)	210.0 kN	(6.2.b)
V_{app}/ max(V_{Rd},c.a, V_{Rd},c.b)	836.1 / Max(313.4, 210.0)	2.668	Links Req
V_{Rd},s = A_{sw}/ S • Z • 0.8 • f_{ywk} • $\cot\theta$	462 • 150.0 • 1027.35 • 0.8 • 500.0 • 2.5	3163.0 kN	(6.8)
V_{Rd},max = $\alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd} / (\cot\theta + \tan\theta)$	1 • 550 • 1027.35 • 0.54 • 18.1 / (2.5 + 0.4)	1907.9 kN	(6.9)
V_{app}/ max(V_{Rd},s, V_{Rd},max)	836.1 / Min(3163.0, 1907.9)	0.438	OK

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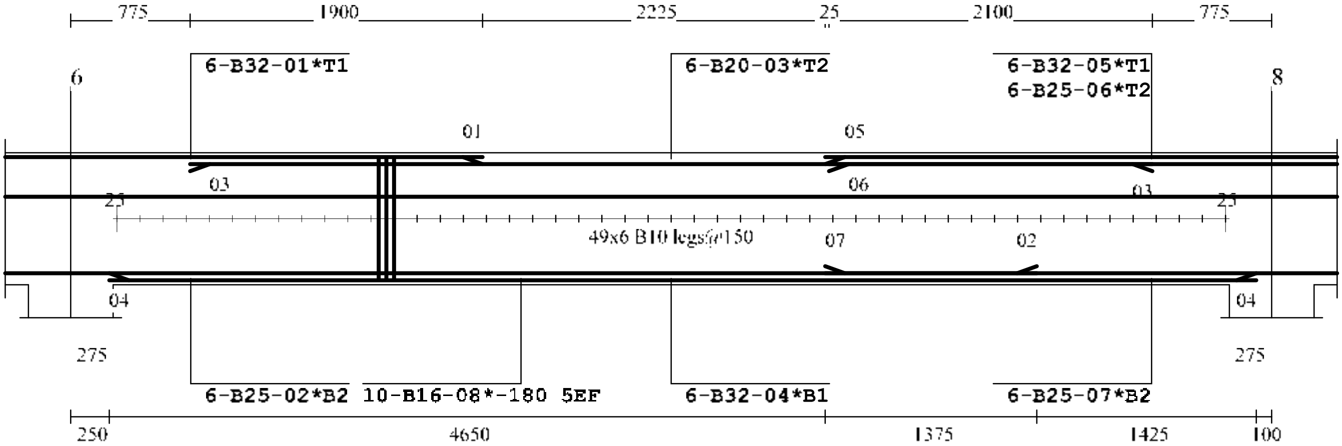
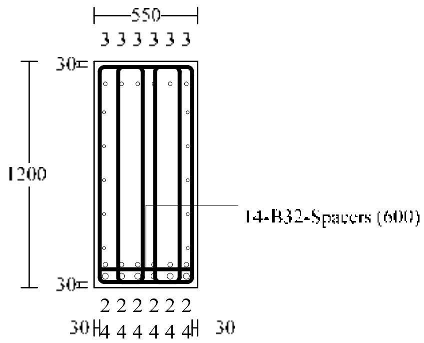
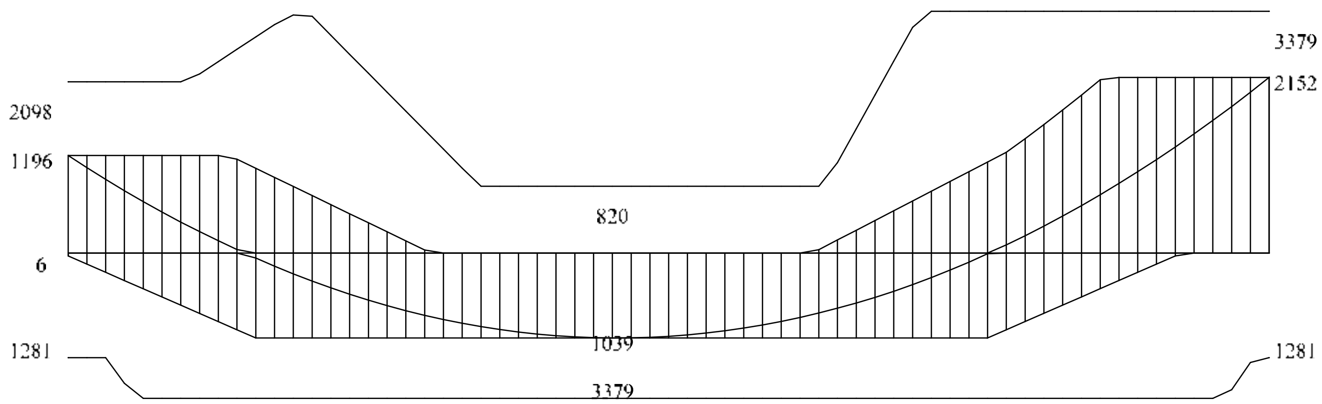
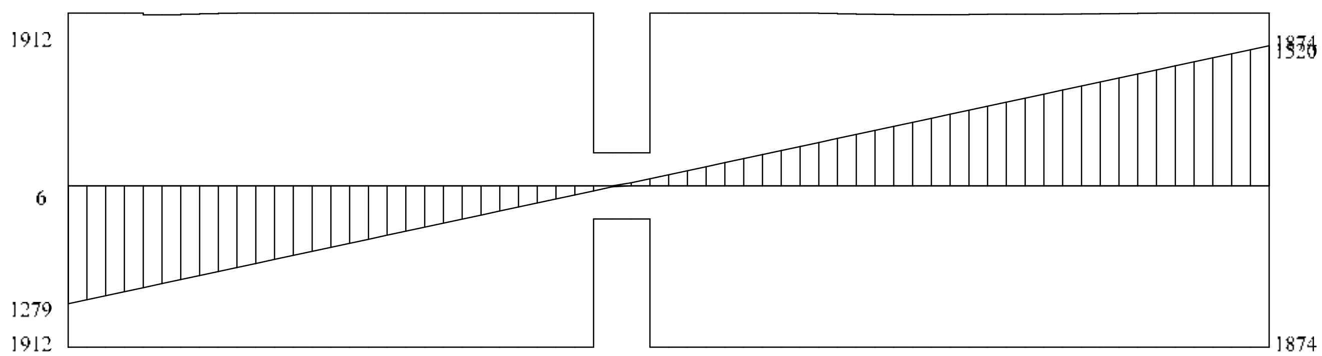






Tension Force Envelope (EC2 Fig 9.2)



Shear Force Envelope

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Member CBL1Id 3

Basic Data

Design to	EC 2: 2004 - Using UK values		
$f_{ck}, f_{yk}, \gamma_c, \gamma_s, \eta, \lambda$	C32/40, 500, 1.5, 1.15, 1.0, 0.8		
f_{ykv}, \$v crit	500, 0.1		

Minor Axis Moments

My max = 17.0 kN.m	Minor axis moments ignored by user		

Torsional Design

$A_k = Fn(A, u, B, H, Cvr_{max}, t)$	660000, 3500, 550, 1200, 56, 112.00	476544 mm²	
$T_{Rd\cdot c} = f_{ctd} \cdot t \cdot 2 \cdot A_k$	1.41 • 112.00 • 2 • 476544	150.6 kN.m	
$T_{Rd\cdot max} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_k \cdot t \cdot Sin\theta \cdot Cos\theta$	2•0.52•1.00•18.13•476544•112.0•0.89•0.45	405.1 kN.m	
Unity $T_{Ed}/T_{Rd \cdot c} + V_{Ed}/V_{Rd \cdot c}$	30.32/150.63 + 1520.21/359.17	4.43	Design Torsion
$A_{sl} = T_{Ed}/A_k/2 \cdot Cot\theta \cdot u_k/(F_{ywk}/\gamma_s)$	30.32 / 476544 / 2 • 2.00 • 3052/(500 / 1.15)	446.7 mm²	
$A_{s\,Per_n} = A_{sl}/(No._{Hor} + No._{Vert})/2$	446.7/ (3 + 3) /2	37.2 mm²	
A_{sr} deduction T+B = $A_s \cdot No._{Hor}$	37.2 • 3	111.7 mm²	
$q = T / A_k / 2$	30.3 / 476544/2	31.8 N/mm	
A_s/S_vreduction = $(q \cdot \gamma_s)/(Cot\theta \cdot F_{ywk}) \cdot 2$	(31.8 • 1.15) / (2.00 • 500) • 2	0.073 mm	
$A_s$prov	min(4825, 2945, 1885, 4825, 7771, 2945)	1885	OK

Bending Moments

Left Support Steel Hogging

A_s'Required to EC2 Cl 3.1.7, Fig 3.5	Assuming d = 1146, d' = 54	top 2711, bottom 112 mm²

In-Span Steel @ 3534 mm. Sagging

A_s'Required to EC2 Cl 3.1.7, Fig 3.5	Assuming d = 1146, d' = 54	top 112, bottom 2307 mm²

Right Support Steel Hogging

A_s'Required to EC2 Cl 3.1.7, Fig 3.5	Assuming d = 1146, d' = 54	top 4907, bottom 112 mm²

Shear

Max Shear

V_{Rd}.max = $0.5 \cdot B_w \cdot d \cdot v \cdot f_{cd}$	0.5•550•1121.1•0.523•18	2924.9 kN	
V_{Ed}.max		1389 kN	OK

Nominal Shear Zone at 276 mm

V_{Rd}.c.a=Fn(C_{rdc},K, A_{sl},f_{ck},K_1,σ_{cp},B_w,d)	0.12, 1.418, 4714, 32, 0.15, 0.0, 550, 1144.0	308.7 kN	(6.2.a)
V_{Rd}.c.b=(V_{min}+ $K_1 \cdot \sigma_{cp}$) • B_w• d	(0.34 + 0.15 • 0.0) • 550.0 • 1144.0)	212.8 kN	(6.2.b)
V_{app}/ max(V_{Rd}.c.a, V_{Rd}.c.b)	1179.8 / Max(308.7, 212.8)	3.821	Links Req
V_{Rd}.s = A_{sw}/ S • Z • 0.8 • f_{ywk}• Cotθ	460 • 150.0 • 1029.6 • 0.8 • 500.0 • 2.5	3157.6 kN	(6.8)
V_{Rd}.max =$\alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd}/(\cot\theta + \tan\theta)$	1 • 550 • 1029.6 • 0.54 • 18.1 / (2.5 + 0.4)	1912.1 kN	(6.9)
V_{app}/ max(V_{Rd}.s, V_{Rd}.max)	1179.8 / Min(3157.6, 1912.1)	0.617	OK

Nominal Shear Zone at 7524 mm

V_{Rd}.c.a=Fn(C_{rdc},K, A_{sl},f_{ck},K_1,σ_{cp},B_w,d)	0.12, 1.422, 7659, 32, 0.15, 0.0, 550, 1121.1	359.2 kN	(6.2.a)
V_{Rd}.c.b=(V_{min}+ $K_1 \cdot \sigma_{cp}$) • B_w• d	(0.34 + 0.15 • 0.0) • 550.0 • 1121.07)	207.1 kN	(6.2.b)
V_{app}/ max(V_{Rd}.c.a, V_{Rd}.c.b)	1421.3 / Max(359.2, 207.1)	3.957	Links Req
V_{Rd}.s = A_{sw}/ S • Z • 0.8 • f_{ywk}• Cotθ	460 • 150.0 • 1008.96 • 0.8 • 500.0 • 2.5	3094.3 kN	(6.8)
V_{Rd}.max =$\alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd}/(\cot\theta + \tan\theta)$	1 • 550 • 1008.96 • 0.54 • 18.1 / (2.5 + 0.4)	1873.7 kN	(6.9)
V_{app}/ max(V_{Rd}.s, V_{Rd}.max)	1421.3 / Min(3094.3, 1873.7)	0.759	OK

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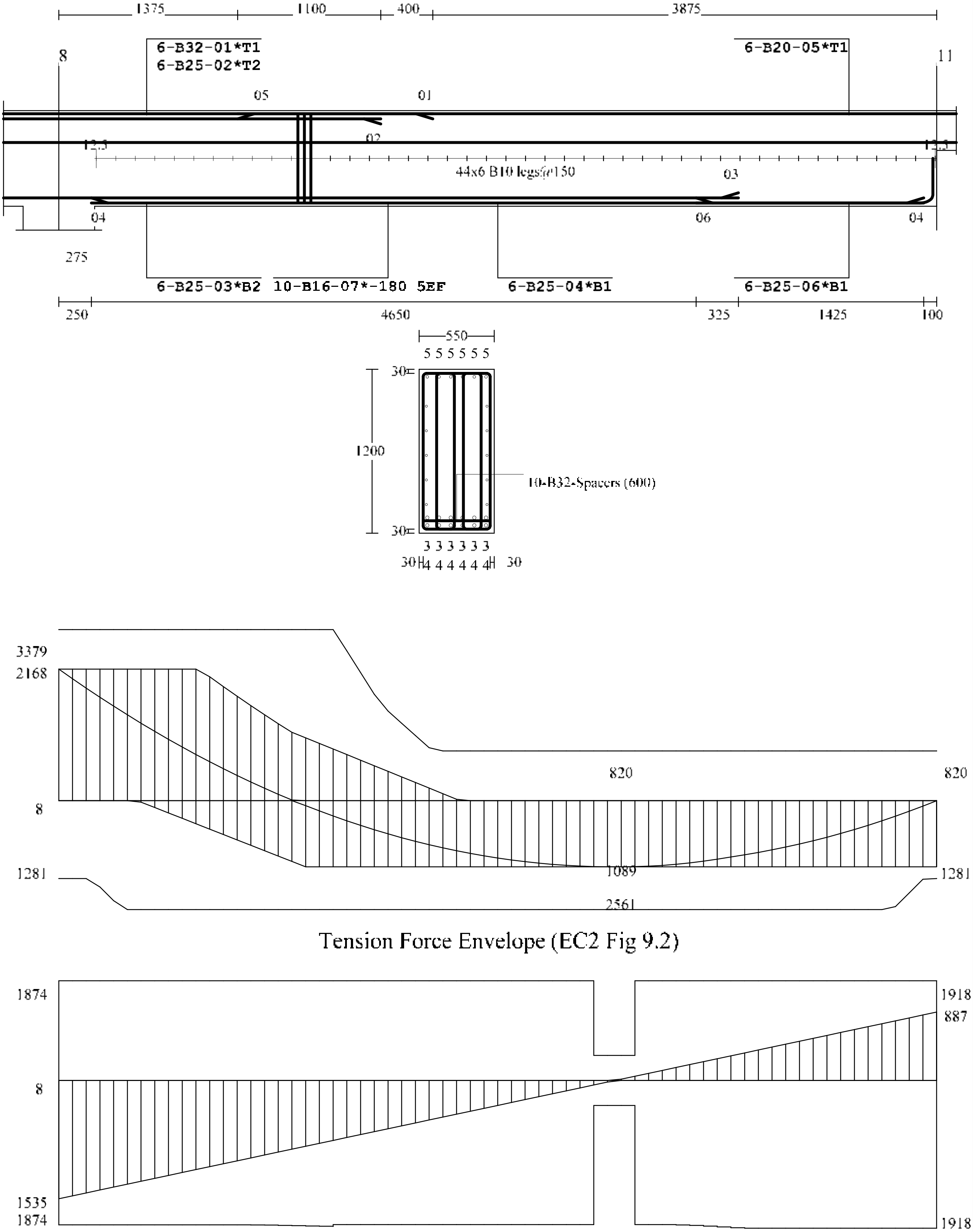

Tension Force Envelope (EC2 Fig 9.2)

Shear Force Envelope

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Basic Data

Design to	EC 2: 2004 - Using UK values		
$f_{ck}, f_{yk}, \gamma_c, \gamma_s, \eta, \lambda$	C32/40, 500, 1.5, 1.15, 1.0, 0.8		
f_{ykv}, $v crit	500, 0.1		

Minor Axis Moments

My max = 18.8 kN.m	Minor axis moments ignored by user		

Torsional Design

$Ak = Fn(A, u, B, H, Cvr_{max}, t)$	660000, 3500, 550, 1200, 56, 112.00	476544 mm²	
$T_{Rd\cdot c} = f_{ctd} \cdot t \cdot 2 \cdot A_k$	1.41 • 112.00 • 2 • 476544	150.6 kN.m	
$T_{Rd\cdot max} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_k \cdot t \cdot Sin\theta \cdot Cos\theta$	2•0.52•1.00•18.13•476544•112.0•0.89•0.45	405.1 kN.m	
Unity $T_{Ed}/T_{Rd}.c + V_{Ed}/V_{Rd\cdot c}$	51.03/150.63 + 1535.22/301.03	5.44	Design Torsion
$A_{sl} = T_{Ed}/A_k/2 \cdot Cot\theta \cdot u_k/(F_{ywk}/\gamma_s)$	51.03 / 476544 / 2 • 2.00 • 3052/(500 / 1.15)	751.7 mm²	
$A_{s\,Per_n} = A_{sl}/(No._{Hor} + No._{Vert})/2$	751.7/ (3 + 3) /2	62.6 mm²	
A_{sr} deduction T+B = $A_s \cdot No._{Hor}$	62.6 • 3	187.9 mm²	
$q = T / A_k / 2$	51.0 / 476544/2	53.5 N/mm	
A_s/S_vreduction = $(q \cdot \gamma_s)/(Cot\theta \cdot F_{ywk}) \cdot 2$	(53.5 • 1.15) / (2.00 • 500) • 2	0.123 mm	
$A_s$prov	min(7771, 2945, 2945, 1885, 2945)	1885	OK

Bending Moments

Left Support Steel Hogging

A'_sRequired to EC2 Cl 3.1.7, Fig 3.5	Assuming d = 1146, d' = 54	top 5021, bottom 188 mm²	

In-Span Steel @ 4324 mm. Sagging

A'_sRequired to EC2 Cl 3.1.7, Fig 3.5	Assuming d = 1146, d' = 54	top 188, bottom 2489 mm²	

Shear

Max Shear

$V_{Rd}.max = 0.5 \cdot B_w \cdot d \cdot v \cdot f_{cd}$	0.5•550•1147.5•0.523•18	2993.9 kN	
$V_{Ed}.max$		1421.7 kN	OK

Nominal Shear Zone at 276 mm

$V_{Rd}.c.a = Fn(C_{rdc}, K, A_{sl}, f_{ck}, K_1, \sigma_{cp}, B_w, d)$	0.12, 1.422, 7583, 32, 0.15, 0.0, 550, 1121.1	358.0 kN	(6.2.a)
$V_{Rd}.c.b = (V_{min} + K_1 \cdot \sigma_{cp}) \cdot B_w \cdot d$	(0.34 + 0.15 • 0.0) • 550.0 • 1121.07)	208.3 kN	(6.2.b)
$V_{app}/\max(V_{Rd}.c.a, V_{Rd}.c.b)$	1436.0 / Max(358.0, 208.3)	4.011	Links Req
$V_{Rd}.s = A_{sw}/S \cdot Z \cdot 0.8 \cdot f_{ywk} \cdot Cot\theta$	453 • 150.0 • 1008.96 • 0.8 • 500.0 • 2.5	3043.9 kN	(6.8)
$V_{Rd}.max = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd}/(cot\theta + tan\theta)$	1 • 550 • 1008.96 • 0.54 • 18.1 / (2.5 + 0.4)	1873.7 kN	(6.9)
$V_{app}/\max(V_{Rd}.s, V_{Rd}.max)$	1436.0 / Min(3043.9, 1873.7)	0.766	OK

Nominal Shear Zone at 6749 mm

$V_{Rd}.c.a = Fn(C_{rdc}, K, A_{sl}, f_{ck}, K_1, \sigma_{cp}, B_w, d)$	0.12, 1.417, 2757, 32, 0.15, 0.0, 550, 1147.5	258.6 kN	(6.2.a)
$V_{Rd}.c.b = (V_{min} + K_1 \cdot \sigma_{cp}) \cdot B_w \cdot d$	(0.33 + 0.15 • 0.0) • 550.0 • 1147.5)	210.8 kN	(6.2.b)
$V_{app}/\max(V_{Rd}.c.a, V_{Rd}.c.b)$	887.2 / Max(258.6, 210.8)	3.431	Links Req
$V_{Rd}.s = A_{sw}/S \cdot Z \cdot 0.8 \cdot f_{ywk} \cdot Cot\theta$	453 • 150.0 • 1032.75 • 0.8 • 500.0 • 2.5	3115.7 kN	(6.8)
$V_{Rd}.max = \alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd}/(cot\theta + tan\theta)$	1 • 550 • 1032.75 • 0.54 • 18.1 / (2.5 + 0.4)	1917.9 kN	(6.9)
$V_{app}/\max(V_{Rd}.s, V_{Rd}.max)$	887.2 / Min(3115.7, 1917.9)	0.463	OK

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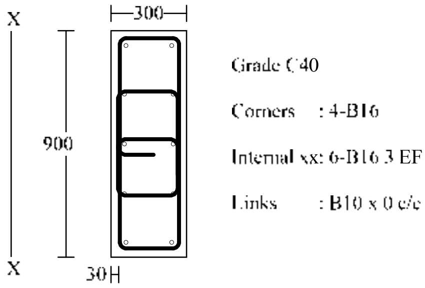

Basic Data

Design to	EC 2: 2004 - Using UK values		
Grades f_{ck}, f_{yk}, γ_c, γ_s, η, λ	C32/40, 500, 1.5, 1.15, 1.0, 0.8		
f_{ykv}, v_{crit}	500, 0.1		

Loading Case 1 : Dead plus Live b

Loading N Mx,Mx,My,My	324.7 kN, 1.0 kN.m, 0.0 kN.m, -16.3 kN.m, -15.2 kN.m		

Slenderness Classification

X-X Braced: λ_{lim}=fn(A,ω , C, n)	0.7, 0.205, 1.70, 0.07	109.76	
X-X Effective len: l_0= fn(k1, k2, l)	0.100, 10.000, 4.000	3.042 m	
X-X Slenderness: $\lambda = l_0/i$	3.042 / 0.087 = 35.1	$\lambda <= \lambda_{lim}$	Short
Y-Y Braced λ_{lim}=fn(A,ω , C, n)	0.7, 0.205, 0.77, 0.07	49.49	
Y-Y Effective len: l_0= fn(k1, k2, l)	0.100, 0.115, 4.000	2.385 m	
Y-Y Slenderness $\lambda = l_0/i$	2.385 / 0.260 = 9.2	$\lambda <= \lambda_{lim}$	Short
Equ. 5.38a $\lambda_{xx}/\lambda_{yy}$	35.1 / 9.2 >> Limit 0.5 to 2	3.825	Bi-Axial

Axial Capacity

N_{uz}= F_{av}•(B • H - Asc) + Asc • f_{yk}/γ_s	18.1•(900•300-2011) + 2010.6 • 500 / 1.15	5733.7 kN	OK

Design Moments x-x

M_{ed0}= Max(M, M)	0.0, 1.0 (no nominal moments)	1.0 kN.m	
M_{ed} = Fn(M, M, N_{ed}, e_o, e_i)	0.0, 1.0, 324.7, 20.0, 7.6	6.5 kN.m	

Design Moments y-y

M_{ed0}= Max(M, M)	-15.2, -16.3 (no nominal moments)	16.3 kN.m	
M_{ed} = Fn(M, M, N_{ed}, e_o, e_i)	-15.2, -16.3, 324.7, 30.0, 6.0	18.2 kN.m	
Data N, Mz, Mz_0, My, My_0, Beq, heq	324.7, 6.5, 1.0, 18.2, 16.3, 900, 300		
Equ. 5.38b (ey_0/ Heq) / (ez / Beq)	(3.2/300) / (56.0/900)	0.170	<= 0.2
Equ. 5.38b (ez_0/ Beq) / (ey / Heq)	(50.1/900) / (20.0/300)	0.834	

Uni-Axial Moment Capacity: X-X

Design Loads

N_{ed}, $M_{ed\ x\text{-}x}$, $M_{ed\ y\text{-}y}$, $M_{ed\ res}$, Ang	324.7 kN, 6.5 kN.m, 0.0 kN.m, 6.5 kN.m, 0.0 deg		
Design Data X/h, h, b, X, Ac, Ybar	0.177, 300 mm, 900 mm, 53.14 mm, 38262 mm², 21.3 mm		
Bar group1:M1 fn(bars,d,ε%,σ,la,F)	5 x 16, 252.0, -1.310, -435, -102.0, -437.1	44.6 kNm	
Bar group2:M2 fn(bars,d,ε%,σ,la,F)	5 x 16, 48.0, 0.034, 68, 102.0, 68.1	6.9 kNm	
Concrete F_c=(Ac_{net}•η •fcd)	(38262 x 1.00 x 18.1)	693.8 kN	
F Equilibrum $\sum (F_t) + F_C - F_{app}$= 0	-437 + 68 + 694 - 324.7	0.1 kN	OK
Concrete Mc=Fc•(H/2-Ybar)	693.8 x (300 / 2 - 21.3)	89.3 kNm	
Mu =Mc − (M1+...+M2)	89.3 − (44.6−6.9)	140.9kNm	OK
Max Moment/Mu	6.5 / 140.9	0.046	OK

Uni-Axial Moment Capacity: Y-Y

Design Loads

N_{ed}, $M_{ed\ y\text{-}y}$, $M_{ed\ x\text{-}x}$, $M_{ed\ res}$, Ang	324.7 kN, 18.2 kN.m, 0.0 kN.m, 18.2 kN.m, 0.0 deg		
Design Data X/h, h, b, X, Ac, Ybar	0.200, 900 mm, 300 mm, 179.83 mm, 43160 mm², 71.9 mm		
Bar group1:M1 fn(bars,d,ε%,σ,la,F)	2 x 16, 852.0, -1.308, -435, -402.0, -174.8	70.3 kNm	
Bar group2:M2 fn(bars,d,ε%,σ,la,F)	2 x 16, 651.0, -0.917, -435, -201.0, -174.8	35.1 kNm	
Bar group3:M3 fn(bars,d,ε%,σ,la,F)	2 x 16, 450.0, -0.526, -435, 0.0, -174.8	0.0 kNm	

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Bar group4:M4 fn(bars,d,ε%,σ,la,F)	2 x 16, 249.0, -0.135, -269, 201.0, -108.3	-21.8 kNm	
Bar group5:M5 fn(bars,d,ε%,σ,la,F)	2 x 16, 48.0, 0.257, 435, 402.0, 174.8	70.3 kNm	
Concrete F_c=(Ac_{net}•η •fcd)	(43160 x 1.00 x 18.1)	782.6 kN	
F Equilibrum$\sum$ (F_t) + F_C - F_{app}= 0	-175 -175 -175 -108 + 175 + 783 -324.7	0.0 kN	OK
Concrete Mc=Fc•(H/2-Ybar)	782.6 x (900 / 2 - 71.9)	295.9 kNm	
Mu =Mc − (M1+...+M5)	295.9 + (70.3+35.1+0.0+-21.8+70.3)	449.8kNm	OK
Max Moment/Mu	18.2 / 449.8	0.040	OK

Bi-Axial Moment Capacity: X-X Axis Dominant
Design Loads

N_{ed}, $M_{ed\ x\text{-}x}$, $M_{ed\ y\text{-}y}$, $M_{ed\ res}$, Ang	324.7 kN, 6.5 kN.m, 16.3 kN.m, 17.5 kN.m, 68.2 deg		
Design Data X/h, h, b, X, Ac, Ybar	0.260, 947 mm, 900 mm, 246.44 mm, 45713 mm², 122.7 mm		
Bar group1:M1 fn(bars,d,ε%,σ,la,F)	1 x 16, 884.7, -0.906, -435, -411.2, -87.4	35.9 kNm	
Bar group2:M2 fn(bars,d,ε%,σ,la,F)	1 x 16, 138.1, 0.154, 308, 335.5, 61.9	20.8 kNm	
Bar group3:M3 fn(bars,d,ε%,σ,la,F)	1 x 16, 698.0, -0.641, -435, -224.5, -87.4	19.6 kNm	
Bar group4:M4 fn(bars,d,ε%,σ,la,F)	1 x 16, 511.4, -0.376, -435, -37.8, -87.4	3.3 kNm	
Bar group5:M5 fn(bars,d,ε%,σ,la,F)	1 x 16, 324.7, -0.111, -222, 148.8, -44.7	-6.7 kNm	
Bar group6:M6 fn(bars,d,ε%,σ,la,F)	1 x 16, 809.0, -0.799, -435, -335.5, -87.4	29.3 kNm	
Bar group7:M7 fn(bars,d,ε%,σ,la,F)	1 x 16, 62.4, 0.261, 435, 411.2, 87.4	35.9 kNm	
Bar group8:M8 fn(bars,d,ε%,σ,la,F)	1 x 16, 622.4, -0.534, -435, -148.8, -87.4	13.0 kNm	
Bar group9:M9 fn(bars,d,ε%,σ,la,F)	1 x 16, 435.7, -0.269, -435, 37.8, -87.4	-3.3 kNm	
Bar group10:M10 fn(bars,d,ε%,σ,la,F)	1 x 16, 249.0, -0.004, -7, 224.5, -1.5	-0.3 kNm	
Concrete F_c=(Ac_{net}•η •fcd)	(45713 x 0.90 x 18.1)	746.0 kN	
F Equilibrum$\sum$ (F_t) + F_C - F_{app}= 0	-87+62-87-87-45-87+87-87-87-1−746-324.7	-0.1 kN	OK
Concrete Mc=Fc•(H/2-Ybar)	746.0 x (947 / 2 - 122.7)	261.7 kNm	
Mu =Mc − (M1+...+M10)	261.7 + (35.9−20.8+19.6+3.3+-6.7+29.3−35.9)		
	+ (13.0+-3.3+-0.3)	409.3kNm	OK
Max Moment/Mu	17.5 / 409.3	0.043	OK

Bi-Axial Moment Capacity: Y-Y Axis Dominant
Design Loads

N_{ed}, $M_{ed\ y\text{-}y}$, $M_{ed\ x\text{-}x}$, $M_{ed\ res}$, Ang	324.7 kN, 18.2 kN.m, 1.0 kN.m, 18.2 kN.m, 3.2 deg		
Design Data X/h, h, b, X, Ac, Ybar	0.206, 915 mm, 300 mm, 188.68 mm, 42813 mm², 79.6 mm		
Bar group1:M1 fn(bars,d,ε%,σ,la,F)	1 x 16, 864.9, -1.254, -435, -407.1, -87.4	35.6 kNm	
Bar group2:M2 fn(bars,d,ε%,σ,la,F)	1 x 16, 853.4, -1.233, -435, -395.6, -87.4	34.6 kNm	
Bar group3:M3 fn(bars,d,ε%,σ,la,F)	1 x 16, 664.2, -0.882, -435, -206.4, -87.4	18.0 kNm	
Bar group4:M4 fn(bars,d,ε%,σ,la,F)	1 x 16, 463.5, -0.510, -435, -5.8, -87.4	0.5 kNm	
Bar group5:M5 fn(bars,d,ε%,σ,la,F)	1 x 16, 262.8, -0.138, -275, 194.9, -55.3	-10.8 kNm	
Bar group6:M6 fn(bars,d,ε%,σ,la,F)	1 x 16, 62.1, 0.235, 435, 395.6, 87.4	34.6 kNm	
Bar group7:M7 fn(bars,d,ε%,σ,la,F)	1 x 16, 50.6, 0.256, 435, 407.1, 87.4	35.6 kNm	
Bar group8:M8 fn(bars,d,ε%,σ,la,F)	1 x 16, 652.7, -0.861, -435, -194.9, -87.4	17.0 kNm	
Bar group9:M9 fn(bars,d,ε%,σ,la,F)	1 x 16, 452.0, -0.488, -435, 5.8, -87.4	-0.5 kNm	
Bar group10:M10 fn(bars,d,ε%,σ,la,F)	1 x 16, 251.3, -0.116, -232, 206.4, -46.7	-9.6 kNm	
Concrete F_c=(Ac_{net}•η •fcd)	(42813 x 1.00 x 18.1)	776.3 kN	
F Equilibrum$\sum$ (F_t) + F_C - F_{app}= 0	-87-87-87-87-55+87+87-87-87-47+776-324.7	-0.1 kN	OK
Concrete Mc=Fc•(H/2-Ybar)	776.3 x (915 / 2 - 79.6)	293.6 kNm	
Mu =Mc − (M1+...+M10)	293.6 + (35.6−34.6+18.0+0.5+-10.8+34.6+35.6)		
	+ (17.0+-0.5+-9.6)	448.6kNm	OK
Max Moment/Mu	18.2 / 448.6	0.041	OK

Shear Check

V_{app}/ max(V_{Rd},c.a, V_{Rd},c.b)	0.4 / Max(93.3, 119.7)	0.003 no Links req

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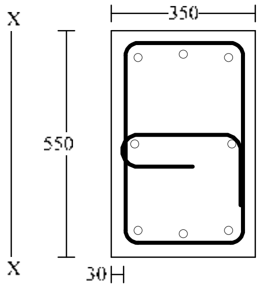

Grade C40

Corners : 4-B20

Internal xx: 2-B20 1 EF

Internal yy: 2-B20 1 EF

Links : B12 x 100 c/c

Basic Data

Design to	EC 2: 2004 - Using UK values		
Grades $f_{ck}, f_{yk}, \gamma_c, \gamma_s, \eta, \lambda$	C32/40, 500, 1.5, 1.15, 1.0, 0.8		
f_{ykv}, v_{crit}	500, 0.1		

Loading Case 1 : Dead plus Live b

Loading N Mx,Mx,My,My	1096.0 kN, 0.0 kN.m, 0.0 kN.m, -13.1 kN.m, -10.2 kN.m		

Slenderness Classification

X-X Braced:λ_{lim}=fn(A,ω , C, n)	0.7, 0.360, 0.70, 0.31	22.94	
X-X Effective len: l_0= fn(k1, k2, l)	0.100, 0.754, 4.000	2.773 m	
X-X Slenderness:$\lambda = l_0/i$	2.773 / 0.101 = 27.4	$\lambda > \lambda_{lim}$	Slender
Y-Y Bracedλ_{lim}=fn(A,ω , C, n)	0.7, 0.360, 0.92, 0.31	30.28	
Y-Y Effective len: l_0= fn(k1, k2, l)	0.100, 0.463, 4.000	2.669 m	
Y-Y Slenderness$\lambda = l_0/i$	2.669 / 0.159 = 16.8	$\lambda <= \lambda_{lim}$	Short
Equ. 5.38a$\lambda_{xx}/\lambda_{yy}$	27.4 / 16.8 >> Limit 0.5 to 2	1.632	OK

Axial Capacity

$N_{uz}= F_{av} \cdot (B \cdot H - Asc) + Asc \cdot f_{yk}/\gamma_s$	18.1•(550•350-2513) + 2513.3 • 500 / 1.15	4537.8 kN	OK

Design Moments x-x

e_2= Fn(K_r, K_{φ},φ_{ef}, n, n_u, n_{bal},ω d, ...)	1.00, 1.65, 2.0, 0.314, 1.313, 0.396, 0.313, 298	20.62 mm	
M_{ed0} = Fn(M, M, M_2, $M0_{ed}$, N_{ed}, e_2)	0, 0, 22.6, 0, 1096, 20.6 (no nominal moments)	22.6 kN.m	
M_{ed} = Fn(M, M, M_2, $M0_{ed}$, N_{ed}, e_0, e_i, e_2)	0, 0, 22.6, 21.9, 1096, 20, 6.9, 20.6	44.5 kN.m	

Design Moments y-y

M_{ed0}= Max(M, M)	-10.2, -13.1 (no nominal moments)	13.1 kN.m	
M_{ed} = Fn(M, M, N_{ed}, e_o, e_i)	-10.2, -13.1, 1096.0, 20.0, 6.7	21.9 kN.m	
Data N, Mz, Mz_0, My, My_0, Beq, heq	1096.0, 44.5, 22.6, 21.9, 13.1, 550, 350		
Equ. 5.38b $(ey_0/Heq) / (ez / Beq)$	(20.6/350) / (20.0/550)	1.620	
Equ. 5.38b $(ez_0/Beq) / (ey / Heq)$	(12.0/550) / (40.6/350)	0.187	<= 0.2
Bi-Axial Design	Not required as Equ. 5.38a & 5.38b satisfied		

Uni-Axial Moment Capacity: X-X

Design Loads

N_{ed}, $M_{ed\ x\text{-}x}$, $M_{ed\ y\text{-}y}$, $M_{ed\ res}$, Ang	1096.0 kN, 44.5 kN.m, 0.0 kN.m, 44.5 kN.m, 0.0 deg		
Design Data X/h, h, b, X, Ac, Ybar	0.422, 350 mm, 550 mm, 147.60 mm, 64943 mm², 59.0 mm		
Bar group1:M1 fn(bars,d,ε%,σ,la,F)	3 x 20, 298.0, -0.357, -435, -123.0, -409.8	50.4 kNm	
Bar group2:M2 fn(bars,d,ε%,σ,la,F)	2 x 20, 175.0, -0.065, -130, 0.0, -81.7	0.0 kNm	
Bar group3:M3 fn(bars,d,ε%,σ,la,F)	3 x 20, 52.0, 0.227, 435, 123.0, 409.8	50.4 kNm	
Concrete F_c=($Ac_{net} \cdot \eta \cdot fcd$)	(64943 x 1.00 x 18.1)	1177.6 kN	
F Equilibrum$\sum (F_t) + F_C - F_{app} = 0$	-410 - 82 + 410 + 1178 - 1096.0	0.0 kN	OK
Concrete Mc=Fc•(H/2-Ybar)	1177.6 x (350 / 2 - 59.0)	136.6 kNm	
Mu =Mc + (M1+...+M3)	136.6 + (50.4+0.0+50.4)	237.4kNm	OK
Max Moment/Mu	44.5 / 237.4	0.188	OK

Uni-Axial Moment Capacity: Y-Y

Design Loads

N_{ed}, $M_{ed\ y\text{-}y}$, $M_{ed\ x\text{-}x}$, $M_{ed\ res}$, Ang	1096.0 kN, 21.9 kN.m, 0.0 kN.m, 21.9 kN.m, 0.0 deg
Design Data X/h, h, b, X, Ac, Ybar	0.422, 550 mm, 350 mm, 231.94 mm, 64943 mm², 92.8 mm

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Bar group1:M1 fn(bars,d,ε%,σ,la,F)	3 x 20, 498.0, -0.401, -435, -223.0, -409.8	91.4 kNm	
Bar group2:M2 fn(bars,d,ε%,σ,la,F)	2 x 20, 275.0, -0.065, -130, 0.0, -81.7	0.0 kNm	
Bar group3:M3 fn(bars,d,ε%,σ,la,F)	3 x 20, 52.0, 0.272, 435, 223.0, 409.8	91.4 kNm	
Concrete F_c=(Ac_{net}•η •fcd)	(64943 x 1.00 x 18.1)	1177.6 kN	
F Equilibrum$\sum$ (F_t) + F_C - F_{app}= 0	-410 - 82 + 410 + 1178 - 1096.0	0.0 kN	OK
Concrete Mc=Fc•(H/2-Ybar)	1177.6 x (550 / 2 - 92.8)	214.6 kNm	
Mu =Mc + (M1+...+M3)	214.6 + (91.4+0.0+91.4)	397.4kNm	OK
Max Moment/Mu	21.9 / 397.4	0.055	OK

Shear Check

V_{app}/ max(V_{Rd},c.a, V_{Rd},c.b)	0.7 / Max(96.1, 81.7)	0.008	no Links req

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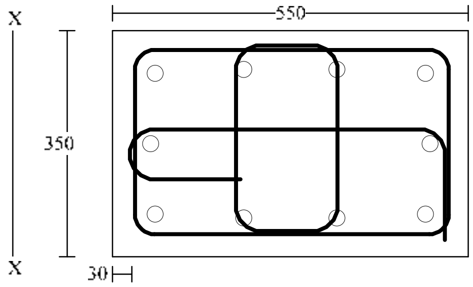

Grade C40

Corners : 4-B25

Internal xx: 2-B25 1 EF

Internal yy: 4-B25 2 EF

Links : B12 x 250 c/c

Basic Data

Design to	EC 2: 2004 - Using UK values		
Grades f_{ck}, f_{yk}, γ_c, γ_s, η, λ	C32/40, 500, 1.5, 1.15, 1.0, 0.8		
f_{ykv}, v_{crit}	500, 0.1		

Loading Case 2 : Dead plus Live a

Loading N Mx,Mx,My,My	2239.1 kN, -33.3 kN.m, 71.6 kN.m, -8.6 kN.m, -5.0 kN.m		

Slenderness Classification

X-X Braced:λ_{lim}=fn(A,ω , C, n)	0.7, 0.703, 2.16, 0.64	58.70	
X-X Effective len: l_0= fn(k1, k2, l)	0.100, 0.268, 4.000	2.548 m	
X-X Slenderness:λ = l_0/i	2.548 / 0.159 = 16.0	λ <=λ_{lim}	Short
Y-Y Bracedλ_{lim}=fn(A,ω , C, n)	0.7, 0.703, 1.12, 0.64	30.39	
Y-Y Effective len: l_0= fn(k1, k2, l)	0.100, 3.366, 4.000	2.983 m	
Y-Y Slendernessλ = l_0/i	2.983 / 0.101 = 29.5	λ <=λ_{lim}	Short
Equ. 5.38aλ_{xx}/λ_{yy}	16.0 / 29.5 >> Limit 0.5 to 2	0.544	OK

Axial Capacity

N_{uz}= F_{av}•(B • H - Asc) + Asc • f_{yk}/γ_s	18.1•(350•550-4909) + 4908.7 • 500 / 1.15	5535.9 kN	OK

Design Moments x-x

M_{ed0}= Max(M, M)	-33.3, 71.6 (no nominal moments)	71.6 kN.m	
M_{ed} = Fn(M, M, N_{ed}, e_o, e_i)	-33.3, 71.6, 2239.1, 20.0, 6.4	85.8 kN.m	

Design Moments y-y

M_{ed0}= Max(M, M)	-5.0, -8.6 (no nominal moments)	8.6 kN.m	
M_{ed} = Fn(M, M, N_{ed}, e_o, e_i)	-5.0, -8.6, 2239.1, 20.0, 7.5	44.8 kN.m	
Data N, Mz, Mz_0, My, My_0, Beq, heq	2239.1, 85.8, 71.6, 44.8, 8.6, 350, 550		
Equ. 5.38b (ey_0/ Heq) / (ez / Beq)	(32.0/550) / (20.0/350)	1.017	
Equ. 5.38b (ez_0/ Beq) / (ey / Heq)	(3.8/350) / (38.3/550)	0.157	<= 0.2
Bi-Axial Design	Not required as Equ. 5.38a & 5.38b satisfied		

Uni-Axial Moment Capacity: X-X

Design Loads

N_{ed}, $M_{ed\ x\text{-}x}$, $M_{ed\ y\text{-}y}$, $M_{ed\ res}$, Ang	2239.1 kN, 85.8 kN.m, 0.0 kN.m, 85.8 kN.m, 0.0 deg		
Design Data X/h, h, b, X, Ac, Ybar	0.630, 550 mm, 350 mm, 346.43 mm, 97000 mm², 138.6 mm		
Bar group1:M1 fn(bars,d,ε%,σ,la,F)	3 x 25, 495.5, -0.151, -301, -220.5, -443.6	97.8 kNm	
Bar group2:M2 fn(bars,d,ε%,σ,la,F)	2 x 25, 348.5, -0.002, -4, -73.5, -4.1	0.3 kNm	
Bar group3:M3 fn(bars,d,ε%,σ,la,F)	2 x 25, 201.5, 0.146, 293, 73.5, 287.5	21.1 kNm	
Bar group4:M4 fn(bars,d,ε%,σ,la,F)	3 x 25, 54.5, 0.295, 435, 220.5, 640.3	141.2 kNm	
Concrete F_c=(Ac_{net}•η •fcd)	(97000 x 1.00 x 18.1)	1758.9 kN	
F Equilibrum$\sum$ (F_t) + F_C - F_{app}= 0	-444 - 4 + 287 + 640 + 1759 - 2239.1	-0.1 kN	OK
Concrete Mc=Fc•(H/2-Ybar)	1758.9 x (550 / 2 - 138.6)	240.0 kNm	
Mu =Mc + (M1+...+M4)	240.0 + (97.8+0.3+21.1+141.2)	500.4kNm	OK
Max Moment/Mu	85.8 / 500.4	0.172	OK

Uni-Axial Moment Capacity: Y-Y

Design Loads

N_{ed}, $M_{ed\ y\text{-}y}$, $M_{ed\ x\text{-}x}$, $M_{ed\ res}$, Ang	2239.1 kN, 44.8 kN.m, 0.0 kN.m, 44.8 kN.m, 0.0 deg		
Design Data X/h, h, b, X, Ac, Ybar	0.623, 350 mm, 550 mm, 217.95 mm, 95900 mm², 87.2 mm		

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Bar group1:M1 fn(bars,d,ε%,σ,la,F)	4 x 25, 295.5, -0.125, -249, -120.5, -489.0	58.9 kNm	
Bar group2:M2 fn(bars,d,ε%,σ,la,F)	2 x 25, 175.0, 0.069, 138, 0.0, 135.4	0.0 kNm	
Bar group3:M3 fn(bars,d,ε%,σ,la,F)	4 x 25, 54.5, 0.262, 435, 120.5, 853.7	102.9 kNm	
Concrete F_c=(Ac_{net}•η •fcd)	(95900 x 1.00 x 18.1)	1739.0 kN	
F Equilibrum$\sum$ (F_t) + F_C - F_{app}= 0	-489 + 135 − 854 + 1739 - 2239.1	0.0 kN	OK
Concrete Mc=Fc•(H/2-Ybar)	1739.0 x (350 / 2 - 87.2)	152.7 kNm	
Mu =Mc − (M1+...+M3)	152.7 + (58.9+0.0+102.9)	314.5kNm	OK
Max Moment/Mu	44.8 / 314.5	0.142	OK

Shear Check

V_{app}/ max(V_{Rd},c.a, V_{Rd},c.b)	26.2 / Max(103.5, 73.0)	0.253	no Links req

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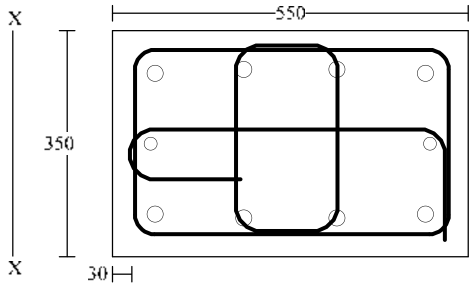
Made by	**:**
Date	**: 09 May 2019 / Ver. 2018.15**
Checked	**:**
Approved	**:**

Member CCL1Id 8 @ Level 1



Grade C40

Corners : 4-B25

Internal xx: 2-B20 1 EF

Internal yy: 4-B25 2 EF

Links : B12 x 200 c/c

Basic Data

Design to	EC 2: 2004 - Using UK values		
Grades f_{ck}, f_{yk}, γ_c, γ_s, η , λ	C32/40, 500, 1.5, 1.15, 1.0, 0.8		
f_{ykv}, v_{crit}	500, 0.1		

Loading Case 2 : Dead plus Live a

Loading N Mx,Mx,My,My	3080.4 kN, -5.6 kN.m, 15.6 kN.m, -1.9 kN.m, -20.7 kN.m		

Slenderness Classification

X-X Braced:λ_{lim}=fn(A,ω , C, n)	0.7, 0.652, 2.06, 0.88	46.54	
X-X Effective len: l_0= fn(k1, k2, l)	0.100, 0.322, 4.000	2.588 m	
X-X Slenderness:λ = l_0/i	2.588 / 0.159 = 16.3	λ <=λ_{lim}	Short
Y-Y Bracedλ_{lim}=fn(A,ω , C, n)	0.7, 0.652, 1.61, 0.88	36.35	
Y-Y Effective len: l_0= fn(k1, k2, l)	0.100, 1.350, 4.000	2.876 m	
Y-Y Slendernessλ = l_0/i	2.876 / 0.101 = 28.5	λ <=λ_{lim}	Short
Equ. 5.38aλ_{xx}/λ_{yy}	16.3 / 28.5 >> Limit 0.5 to 2	0.573	OK

Axial Capacity

N_{uz}= F_{av}•(B • H - Asc) + Asc • f_{yk}/γ_s	18.1•(350•550-4555) + 4555.3 • 500 / 1.15	5388.6 kN	OK

Design Moments x-x

M_{ed0}= Max(M, M)	-5.6, 15.6 (no nominal moments)	15.6 kN.m	
M_{ed} = Fn(M, M, N_{ed}, e_o, e_i)	-5.6, 15.6, 3080.4, 20.0, 6.5	61.6 kN.m	

Design Moments y-y

M_{ed0}= Max(M, M)	-1.9, -20.7 (no nominal moments)	20.7 kN.m	
M_{ed} = Fn(M, M, N_{ed}, e_o, e_i)	-1.9, -20.7, 3080.4, 20.0, 7.2	61.6 kN.m	
Data N, Mz, Mz_0, My, My_0, Beq, heq	3080.4, 61.6, 15.6, 61.6, 20.7, 350, 550		
Equ. 5.38b (ey_0/ Heq) / (ez / Beq)	(5.1/550) / (20.0/350)	0.161	<= 0.2
Equ. 5.38b (ez_0/ Beq) / (ey / Heq)	(6.7/350) / (20.0/550)	0.528	
Bi-Axial Design	Not required as Equ. 5.38a & 5.38b satisfied		

Uni-Axial Moment Capacity: X-X

Design Loads

N_{ed}, $M_{ed\ x-x}$, $M_{ed\ y-y}$, $M_{ed\ res}$, Ang	3080.4 kN, 61.6 kN.m, 0.0 kN.m, 61.6 kN.m, 0.0 deg		
Design Data X/h, h, b, X, Ac, Ybar	0.778, 550 mm, 350 mm, 427.67 mm, 119749 mm², 171.1 mm		
Bar group1:M1 fn(bars,d,ε%,σ,la,F)	2 x 25, 495.5, -0.056, -111, -220.5, -109.0	24.0 kNm	
Bar group2:M2 fn(bars,d,ε%,σ,la,F)	1 x 20, 498.0, -0.058, -115, -223.0, -36.2	8.1 kNm	
Bar group3:M3 fn(bars,d,ε%,σ,la,F)	2 x 25, 348.5, 0.065, 130, -73.5, 127.2	-9.4 kNm	
Bar group4:M4 fn(bars,d,ε%,σ,la,F)	2 x 25, 201.5, 0.185, 370, 73.5, 363.4	26.7 kNm	
Bar group5:M5 fn(bars,d,ε%,σ,la,F)	2 x 25, 54.5, 0.305, 435, 220.5, 426.8	94.1 kNm	
Bar group6:M6 fn(bars,d,ε%,σ,la,F)	1 x 20, 52.0, 0.307, 435, 223.0, 136.6	30.5 kNm	
Concrete F_c=(Ac_{net}•η •fcd)	(119749 x 1.00 x 18.1)	2171.4 kN	
F Equilibrum$\sum$ (F_t) + F_C - F_{app}= 0	-109-36+127+363+427-137+2171-3080.4	0.0 kN	OK
Concrete Mc=Fc•(H/2-Ybar)	2171.4 x (550 / 2 - 171.1)	225.7 kNm	
Mu =Mc − (M1+...+M6)	225.7 + (24.0+8.1+-9.4+26.7+94.1+30.5)	399.7kNm	OK
Max Moment/Mu	61.6 / 399.7	0.154	OK

Uni-Axial Moment Capacity: Y-Y

Design Loads

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N_{ed}, $M_{ed\ y\text{-}y}$, $M_{ed\ x\text{-}x}$, $M_{ed\ res}$, Ang	3080.4 kN, 61.6 kN.m, 0.0 kN.m, 61.6 kN.m, 0.0 deg		
Design Data X/h, h, b, X, Ac, Ybar	0.781, 350 mm, 550 mm, 273.27 mm, 120240 mm², 109.3 mm		
Bar group1:M1 fn(bars,d,ε%,σ,la,F)	4 x 25, 295.5, -0.028, -57, -120.5, -111.8	13.5 kNm	
Bar group2:M2 fn(bars,d,ε%,σ,la,F)	2 x 20, 175.0, 0.126, 252, 0.0, 158.2	0.0 kNm	
Bar group3:M3 fn(bars,d,ε%,σ,la,F)	4 x 25, 54.5, 0.280, 435, 120.5, 853.7	102.9 kNm	
Concrete F_c=(Ac_{net}•η •fcd)	(120240 x 1.00 x 18.1)	2180.3 kN	
F Equilibrum$\sum$ (F_t) + F_C - F_{app}= 0	-112 + 158 − 854 + 2180 - 3080.4	0.0 kN	OK
Concrete Mc=Fc•(H/2-Ybar)	2180.3 x (350 / 2 - 109.3)	143.2 kNm	
Mu =Mc − (M1+...+M3)	143.2 + (13.5+0.0+102.9)	259.6kNm	OK
Max Moment/Mu	61.6 / 259.6	0.237	OK

Shear Check

V_{app}/ max(V_{Rd},c.a, V_{Rd},c.b)	7.1 / Max(99.2, 73.0)	0.071	no Links req

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

Design shear force at 570mm from support V_{Ed} = **397** kN

Design shear stress $v_{Ed} = V_{Ed} / (b \times z)$ = **1.250** N/mm^2

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

Area of shear reinforcement required - exp.6.8 $A_{sv,des} = v_{Ed} \times b / (f_{yd} \times \cot(\theta))$ = **747** mm^2/m

Area of shear reinforcement required $A_{sv,req} = \max(A_{sv,min}, A_{sv,des})$ = **747** mm^2/m

Shear reinforcement provided 4 × 8 legs @ 200 c/c

Area of shear reinforcement provided $A_{sv,prov}$ = **1005** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N $s_{vl,max} = 0.75 \times d$ = **427** mm

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

Zone 2 (2000 mm - 6000 mm) shear - section 6.2

Design shear force at support $V_{Ed,max} = \max(\mathrm{abs}(V_{z2\_max}), \mathrm{abs}(V_{z2\_red\_max}))$ = **232** kN

Min lever arm in shear zone z = **489** mm

Maximum design shear resistance - exp.6.9 $V_{Rd,max} = \alpha_{cw} \times b \times z \times v_1 \times f_{cwd} / (\cot(\theta_{max}) + \tan(\theta_{max}))$ = **1774** kN

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

Design shear force within zone V_{Ed} = **232** kN

Design shear stress $v_{Ed} = V_{Ed} / (b \times z)$ = **0.728** N/mm^2

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

Area of shear reinforcement required - exp.6.8 $A_{sv,des} = v_{Ed} \times b / (f_{yd} \times \cot(\theta))$ = **436** mm^2/m

Area of shear reinforcement required $A_{sv,req} = \max(A_{sv,min}, A_{sv,des})$ = **588** mm^2/m

Shear reinforcement provided 4 × 8 legs @ 250 c/c

Area of shear reinforcement provided $A_{sv,prov}$ = **804** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N $s_{vl,max} = 0.75 \times d$ = **427** mm

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

Zone 3 (6000 mm - 8000 mm) shear - section 6.2

Design shear force at support $V_{Ed,max} = \max(\mathrm{abs}(V_{z3\_max}), \mathrm{abs}(V_{z3\_red\_max}))$ = **463** kN

Min lever arm in shear zone z = **489** mm

Maximum design shear resistance - exp.6.9 $V_{Rd,max} = \alpha_{cw} \times b \times z \times v_1 \times f_{cwd} / (\cot(\theta_{max}) + \tan(\theta_{max}))$ = **1774** kN

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

Design shear force at 570mm from support V_{Ed} = **397** kN

Design shear stress $v_{Ed} = V_{Ed} / (b \times z)$ = **1.250** N/mm^2

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

Area of shear reinforcement required - exp.6.8 $A_{sv,des} = v_{Ed} \times b / (f_{yd} \times \cot(\theta))$ = **747** mm^2/m

Area of shear reinforcement required $A_{sv,req} = \max(A_{sv,min}, A_{sv,des})$ = **747** mm^2/m

Shear reinforcement provided 4 × 8 legs @ 200 c/c

Area of shear reinforcement provided $A_{sv,prov}$ = **1005** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing - exp.9.6N $s_{vl,max} = 0.75 \times d$ = **427** mm

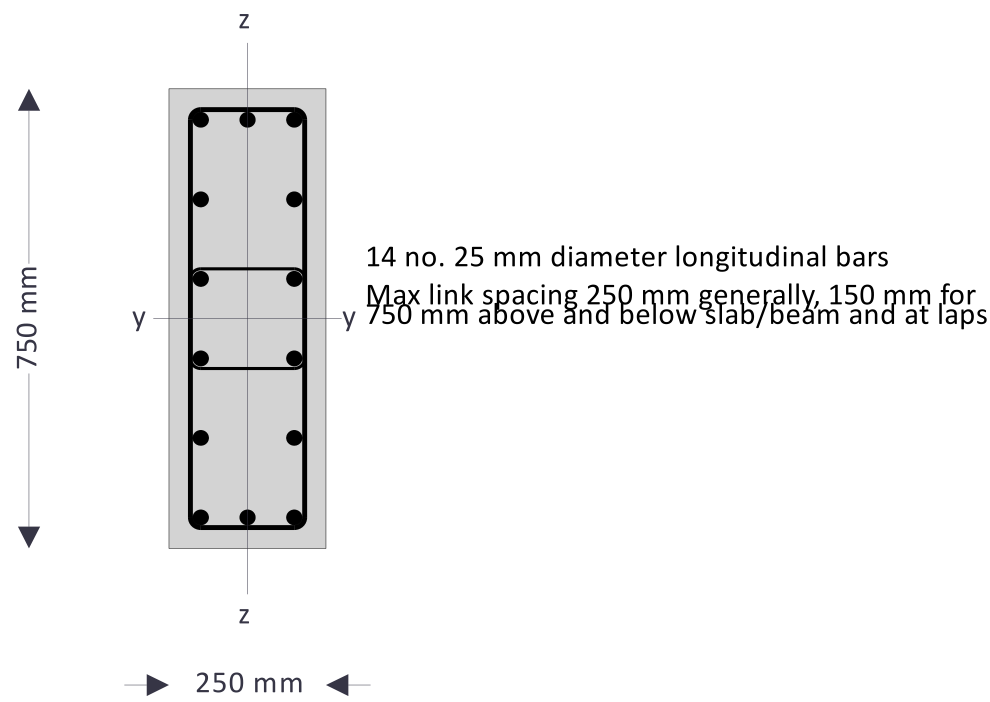
PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Tekla Tedds Cranston Consulting Limited 16 Jubilee Road Newtownards BT23 4YH	Project 24-26 Redington Gardens				Job no. 180709	
	Calcs for Main Spine Column				Start page no./Revision C1	
	Calcs by S	Calcs date 04/01/2019	Checked by	Checked date	Approved by	Approved date

RC COLUMN DESIGN

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

Tedds calculation version 1.3.02



Column input details

Column geometry

Overall depth (perpendicular to y axis)	h = **750** mm
Overall breadth (perpendicular to z axis)	b = **250** mm
Stability in the z direction	**Braced**
Stability in the y direction	**Braced**

Concrete details

Concrete strength class	**C32/40**
Partial safety factor for concrete (2.4.2.4(1))	γ_C = **1.50**
Coefficient α_{cc} (3.1.6(1))	α_{cc} = **0.85**
Maximum aggregate size	d_g = **20** mm

Reinforcement details

Nominal cover to links	c_{nom} = **30** mm
Longitudinal bar diameter	ϕ = **25** mm
Link diameter	ϕ_v = **8** mm
Total number of longitudinal bars	N = **14**
No. of bars per face parallel to y axis	N_y = **3**
No. of bars per face parallel to z axis	N_z = **6**
Area of longitudinal reinforcement	$A_s = N \times \pi \times \phi^2 / 4$ = **6872** mm^2
Characteristic yield strength	f_{yk} = **500** N/mm^2
Partial safety factor for reinft (2.4.2.4(1))	γ_S = **1.15**
Modulus of elasticity of reinft (3.2.7(4))	E_s = **200** kN/mm^2

Fire resistance details

Fire resistance period: R = **60** min

Exposure to fire: **Exposed on one side only**

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} = **3288.0** kN

Moment about y axis at top: M_{topy} = **75.0** kNm

Moment about y axis at bottom: M_{btmy} = **75.0** kNm

Moment about z axis at top: M_{topz} = **50.0** kNm

Moment about z axis at bottom: M_{btmz} = **50.0** kNm

Column effective lengths

Effective length for buckling about y axis: l_{0y} = **3925** mm

Effective length for buckling about z axis: l_{0z} = **3925** mm

Calculated column properties

Concrete properties

Area of concrete: $A_c = h \times b$ = **187500** mm^2

Characteristic compression cylinder strength: f_{ck} = **32** N/mm^2

Design compressive strength (3.1.6(1)): $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **18.1** N/mm^2

Mean value of cylinder strength (Table 3.1): $f_{cm} = f_{ck} + 8$ MPa = **40.0** N/mm^2

Secant modulus of elasticity (Table 3.1): $E_{cm} = 22000 \text{ MPa} \times (f_{cm} / 10 \text{ MPa})^{0.3}$ = **33.3** kN/mm^2

Rectangular stress block factors

Depth factor (3.1.7(3)): $\lambda_{sb} = 0.8$

Stress factor (3.1.7(3)): $\eta = 1.0$

Strain limits

Compression strain limit (Table 3.1): ε_{cu3} = **0.00350**

Pure compression strain limit (Table 3.1): ε_{c3} = **0.00175**

Design yield strength of reinforcement

Design yield strength (3.2.7(2)): $f_{yd} = f_{yk} / \gamma_S$ = **434.8** N/mm^2

Check nominal cover for fire and bond requirements

Min. cover reqd for bond (to links) (4.4.1.2(3)): $c_{min,b} = \max(\phi_v, \phi - \phi_v)$ = **17** mm

Min axis distance for fire (EN1992-1-2 T 5.2a): a_{fi} = **25** mm

Allowance for deviations from min cover (4.4.1.3): Δc_{dev} = **5** mm

Min allowable nominal cover: $c_{nom\_min} = \max(a_{fi} - \phi / 2 - \phi_v, c_{min,b} + \Delta c_{dev})$ = **22.0** mm

PASS - the nominal cover is greater than the minimum required

Effective depths of bars for bending about y axis

Area per bar: $A_{bar} = \pi \times \phi^2 / 4$ = **491** mm^2

Spacing of bars in faces parallel to z axis (c/c): $s_z = (h - 2 \times (c_{nom} + \phi_v) - \phi) / (N_z - 1)$ = **130** mm

Layer 1 (in tension face): $d_{y1} = h - c_{nom} - \phi_v - \phi / 2$ = **700** mm

Layer 2: $d_{y2} = d_{y1} - s_z$ = **570** mm

Layer 3: $d_{y3} = d_{y2} - s_z$ = **440** mm

Layer 4: $d_{y4} = d_{y3} - s_z$ = **310** mm

Layer 5: $d_{y5} = d_{y4} - s_z$ = **180** mm

Layer 6: $d_{y6} = d_{y5} - s_z$ = **51** mm

2nd moment of area of reinft about y axis: $I_{sy} = 2 \times A_{bar} \times [N_y \times (d_{y1}-h/2)^2 + 2 \times (d_{y2}-h/2)^2 + 2 \times (d_{y3}-h/2)^2]$ = **39284** cm^4

Radius of gyration of reinft about y axis $i_{sy} = \sqrt{(I_{sy} / A_s)} = \mathbf{239}$ mm

Effective depth about y axis (5.8.8.3(2)) $d_y = h / 2 + i_{sy} = \mathbf{614}$ mm

Effective depths of bars for bending about z axis

Area of per bar $A_{bar} = \pi \times \phi^2 / 4 = \mathbf{491}$ mm^2

Spacing of bars in faces parallel to y axis (c/c) $s_y = (b - 2 \times (c_{nom} + \phi_v) - \phi) / (N_y - 1) = \mathbf{75}$ mm

Layer 1 (in tension face) $d_{z1} = b - c_{nom} - \phi_v - \phi / 2 = \mathbf{199}$ mm

Layer 2 $d_{z2} = d_{z1} - s_y = \mathbf{125}$ mm

Layer 3 $d_{z3} = d_{z2} - s_y = \mathbf{50}$ mm

2nd moment of area of reinft about z axis $I_{sz} = 2 \times A_{bar} \times N_z \times (d_{z1}-b/2)^2 = \mathbf{3269}$ cm^4

Radius of gyration of reinft about z axis $i_{sz} = \sqrt{(I_{sz} / A_s)} = \mathbf{69}$ mm

Effective depth about z axis (5.8.8.3(2)) $d_z = b / 2 + i_{sz} = \mathbf{194}$ mm

Column slenderness about y axis

Radius of gyration $i_y = h / \sqrt{(12)} = \mathbf{21.7}$ cm

Slenderness ratio (5.8.3.2(1)) $\lambda_y = l_{0y} / i_y = \mathbf{18.1}$

Column slenderness about z axis

Radius of gyration $i_z = b / \sqrt{(12)} = \mathbf{7.2}$ cm

Slenderness ratio (5.8.3.2(1)) $\lambda_z = l_{0z} / i_z = \mathbf{54.4}$

Design bending moments

Frame analysis moments about y axis combined with moments due to imperfections (cl. 5.2 & 6.1(4))

Ecc. due to geometric imperfections (y axis) $e_{iy} = l_{0y} /400 = \mathbf{9.8}$ mm

Min end moment about y axis $M_{01y} = \min(\text{abs}(M_{topy}), \text{abs}(M_{btmy})) + e_{iy} \times N_{Ed} = \mathbf{107.3}$ kNm

Max end moment about y axis $M_{02y} = \max(\text{abs}(M_{topy}), \text{abs}(M_{btmy})) + e_{iy} \times N_{Ed} = \mathbf{107.3}$ kNm

Slenderness limit for buckling about y axis (cl. 5.8.3.1)

Factor A $A = \mathbf{0.7}$

Mechanical reinforcement ratio $\omega = A_s \times f_{yd} / (A_c \times f_{cd}) = \mathbf{0.879}$

Factor B $B = \sqrt{(1 + 2 \times \omega)} = \mathbf{1.661}$

Moment ratio $r_{my} = M_{01y} / M_{02y} = \mathbf{1.000}$

Factor C $C_y = 1.7 - r_{my} = \mathbf{0.700}$

Relative normal force $n = N_{Ed} / (A_c \times f_{cd}) = \mathbf{0.967}$

Slenderness limit $\lambda_{limy} = 20 \times A \times B \times C_y / \sqrt{(n)} = \mathbf{16.5}$

λ_y>=λ_{limy} - Second order effects must be considered

Frame analysis moments about z axis combined with moments due to imperfections (cl. 5.2 & 6.1(4))

Ecc. due to geometric imperfections (z axis) $e_{iz} = l_{0z} /400 = \mathbf{9.8}$ mm

Min end moment about z axis $M_{01z} = \min(\text{abs}(M_{topz}), \text{abs}(M_{btmz})) + e_{iz} \times N_{Ed} = \mathbf{82.3}$ kNm

Max end moment about z axis $M_{02z} = \max(\text{abs}(M_{topz}), \text{abs}(M_{btmz})) + e_{iz} \times N_{Ed} = \mathbf{82.3}$ kNm

Slenderness limit for buckling about y axis (cl. 5.8.3.1)

Factor A $A = \mathbf{0.7}$

Mechanical reinforcement ratio $\omega = A_s \times f_{yd} / (A_c \times f_{cd}) = \mathbf{0.879}$

Factor B $B = \sqrt{(1 + 2 \times \omega)} = \mathbf{1.661}$

Moment ratio $r_{mz} = M_{01z} / M_{02z} = \mathbf{1.000}$

Factor C $C_z = 1.7 - r_{mz} = \mathbf{0.700}$

Relative normal force $n = N_{Ed} / (A_c \times f_{cd}) = \mathbf{0.967}$

Slenderness limit $\lambda_{limz} = 20 \times A \times B \times C_z / \sqrt{(n)} = \mathbf{16.5}$

λ_z>=λ_{limz} - Second order effects must be considered

Local second order bending moment about y axis (cl. 5.8.8.2 & 5.8.8.3)

Relative humidity of ambient environment — RH = **50** %

Column perimeter in contact with atmosphere — u = **1700** mm

Age of concrete at loading — t_0 = **28** day

Parameter n_u — $n_u = 1 + \omega$ = **1.879**

Approx value of n at max moment of resistance — n_{bal} = **0.4**

Axial load correction factor — $K_r = \min(1.0\ ,\ (n_u - n) / (n_u - n_{bal}))$ = **0.617**

Reinforcement design strain — $\varepsilon_{yd} = f_{yd} / E_s$ = **0.00217**

Basic curvature — $curve_{basic\_y} = \varepsilon_{yd} / (0.45 \times d_y)$ = **0.0000079** mm^{-1}

Notional size of column — $h_0 = 2 \times A_c / u$ = **221** mm

Factor α_1 (Annex B.1(1)) — $\alpha_1 = (35\ MPa / f_{cm})^{0.7}$ = **0.911**

Factor α_2 (Annex B.1(1)) — $\alpha_2 = (35\ MPa / f_{cm})^{0.2}$ = **0.974**

Relative humidity factor (Annex B.1(1)) — $\phi_{RH} = [1 + ((1 - RH / 100\%) / (0.1\ mm^{-1/3} \times (h_0)^{1/3})) \times \alpha_1] \times \alpha_2$ = **1.707**

Concrete strength factor (Annex B.1(1)) — $\beta_{fcm} = 16.8 \times (1\ MPa)^{1/2} / \sqrt{(f_{cm})}$ = **2.656**

Concrete age factor (Annex B.1(1)) — $\beta_{t0} = 1 / (0.1 + (t_0 / 1\ day)^{0.2})$ = **0.488**

Notional creep coefficient (Annex B.1(1)) — $\phi_0 = \phi_{RH} \times \beta_{fcm} \times \beta_{t0}$ = **2.215**

Final creep development factor (at t = ¥) — $\beta_{c\infty}$ = 1.0

Final creep coefficient (Annex B.1(1)) — $\phi_\infty = \phi_0 \times \beta_{c\infty}$ = **2.215**

Ratio of SLS to ULS moments — r_{My} = **0.80**

Effective creep ratio — $\phi_{efy} = \phi_\infty \times r_{My}$ = **1.772**

Factor β — $\beta_y = 0.35 + f_{ck} / 200\ MPa - \lambda_y / 150$ = **0.389**

Creep factor — $K_{\phi y} = \max(1.0\ ,\ 1 + \beta_y \times \phi_{efy})$ = **1.690**

Modified curvature — $curve_{mod\_y} = K_r \times K_{\phi y} \times curve_{basic\_y}$ = **0.0000082** mm^{-1}

Curvature distribution factor — c = **10**

Deflection — $e_{2y} = curve_{mod\_y} \times l_{0y}^2 / c$ = **12.6** mm

Nominal 2nd order moment — $M_{2y} = N_{Ed} \times e_{2y}$ = **41.5** kNm

Design bending moment about y axis (cl. 5.8.8.2 & 6.1(4))

Equivalent moment from frame analysis — $M_{0ey} = \max(0.6 \times M_{02y} + 0.4 \times M_{01y},\ 0.4 \times M_{02y})$ = **107.3** kNm

Design moment — $M_{Edy} = \max(M_{02y},\ M_{0ey} + M_{2y},\ M_{01y} + 0.5 \times M_{2y},\ N_{Ed} \times \max(h/30,\ 20\ mm))$

M_{Edy} = **148.8** kNm

Local second order bending moment about z axis (cl. 5.8.8.2 & 5.8.8.3)

Basic curvature — $curve_{basic\_z} = \varepsilon_{yd} / (0.45 \times d_z)$ = **0.0000249** mm^{-1}

Ratio of SLS to ULS moments — r_{Mz} = **0.80**

Effective creep ratio (5.8.4(2)) — $\phi_{efz} = \phi_\infty \times r_{Mz}$ = **1.772**

Factor β — $\beta_z = 0.35 + f_{ck} / 200\ MPa - \lambda_z / 150$ = **0.147**

Creep factor — $K_{\phi z} = \max(1.0\ ,\ 1 + \beta_z \times \phi_{efz})$ = **1.261**

Modified curvature — $curve_{mod\_z} = K_r \times K_{\phi z} \times curve_{basic\_z}$ = **0.0000194** mm^{-1}

Curvature distribution factor — c = **10**

Deflection — $e_{2z} = curve_{mod\_z} \times l_{0z}^2 / c$ = **29.8** mm

Nominal 2nd order moment — $M_{2z} = N_{Ed} \times e_{2z}$ = **98.1** kNm

Design bending moment about z axis (cl. 5.8.8.2 & 6.1(4))

Equivalent moment from frame analysis — $M_{0ez} = \max(0.6 \times M_{02z} + 0.4 \times M_{01z},\ 0.4 \times M_{02z})$ = **82.3** kNm

Design moment — $M_{Edz} = \max(M_{02z},\ M_{0ez} + M_{2z},\ M_{01z} + 0.5 \times M_{2z},\ N_{Ed} \times \max(b/30,\ 20\ mm))$

M_{Edz} = **180.4** kNm

Moment capacity about y axis with axial load N_{Ed}

Moment of resistance of concrete

By iteration:-

Position of neutral axis — y = **568.0** mm

Concrete compression force (3.1.7(3)) — $F_{yc} = \eta \times f_{cd} \times \min(\lambda_{sb} \times y, h) \times b$ = **2059.9** kN

Moment of resistance — $M_{Rdyc} = F_{yc} \times [h / 2 - (\min(\lambda_{sb} \times y, h)) / 2]$ = **304.5** kNm

Moment of resistance of reinforcement

Strain in layer 1 — $\varepsilon_{y1} = \varepsilon_{cu3} \times (1 - d_{y1} / y)$ = **-0.00081**

Stress in layer 1 — $\sigma_{y1} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{y1})$ = **-162.1** N/mm^2

Force in layer 1 — $F_{y1} = N_y \times A_{bar} \times \sigma_{y1}$ = **-238.7** kN

Moment of resistance of layer 1 — $M_{Rdy1} = F_{y1} \times (h / 2 - d_{y1})$ = **77.4** kNm

Strain in layer 2 — $\varepsilon_{y2} = \varepsilon_{cu3} \times (1 - d_{y2} / y)$ = **-0.00001**

Stress in layer 2 — $\sigma_{y2} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{y2})$ = **-2.1** N/mm^2

Force in layer 2 — $F_{y2} = 2 \times A_{bar} \times \sigma_{y2}$ = **-2.1** kN

Moment of resistance of layer 2 — $M_{Rdy2} = F_{y2} \times (h / 2 - d_{y2})$ = **0.4** kNm

Strain in layer 3 — $\varepsilon_{y3} = \varepsilon_{cu3} \times (1 - d_{y3} / y)$ = **0.00079**

Stress in layer 3 — $\sigma_{y3} = \min(f_{yd}, E_s \times \varepsilon_{y3}) - \eta \times f_{cd}$ = **139.7** N/mm^2

Force in layer 3 — $F_{y3} = 2 \times A_{bar} \times \sigma_{y3}$ = **137.2** kN

Moment of resistance of layer 3 — $M_{Rdy3} = F_{y3} \times (h / 2 - d_{y3})$ = **-8.9** kNm

Strain in layer 4 — $\varepsilon_{y4} = \varepsilon_{cu3} \times (1 - d_{y4} / y)$ = **0.00159**

Stress in layer 4 — $\sigma_{y4} = \min(f_{yd}, E_s \times \varepsilon_{y4}) - \eta \times f_{cd}$ = **299.7** N/mm^2

Force in layer 4 — $F_{y4} = 2 \times A_{bar} \times \sigma_{y4}$ = **294.2** kN

Moment of resistance of layer 4 — $M_{Rdy4} = F_{y4} \times (h / 2 - d_{y4})$ = **19.1** kNm

Strain in layer 5 — $\varepsilon_{y5} = \varepsilon_{cu3} \times (1 - d_{y5} / y)$ = **0.00239**

Stress in layer 5 — $\sigma_{y5} = \min(f_{yd}, E_s \times \varepsilon_{y5}) - \eta \times f_{cd}$ = **416.6** N/mm^2

Force in layer 5 — $F_{y5} = 2 \times A_{bar} \times \sigma_{y5}$ = **409.0** kN

Moment of resistance of layer 5 — $M_{Rdy5} = F_{y5} \times (h / 2 - d_{y5})$ = **79.6** kNm

Strain in layer 6 — $\varepsilon_{y6} = \varepsilon_{cu3} \times (1 - d_{y6} / y)$ = **0.00319**

Stress in layer 6 — $\sigma_{y6} = \min(f_{yd}, E_s \times \varepsilon_{y6}) - \eta \times f_{cd}$ = **416.6** N/mm^2

Force in layer 6 — $F_{y6} = N_y \times A_{bar} \times \sigma_{y6}$ = **613.6** kN

Moment of resistance of layer 6 — $M_{Rdy6} = F_{y6} \times (h / 2 - d_{y6})$ = **199.1** kNm

Resultant concrete/steel force — F_y = **3273.3** kN

PASS - This is within half of one percent of the applied axial load

Combined moment of resistance

Moment of resistance about y axis — M_{Rdy} = **671.2** kNm

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

Moment capacity about z axis with axial load N_{Ed}

Moment of resistance of concrete

By iteration:-

Position of neutral axis — z = **184.6** mm

Concrete compression force (3.1.7(3)) — $F_{zc} = \eta \times f_{cd} \times \min(\lambda_{sb} \times z, b) \times h$ = **2008.1** kN

Moment of resistance — $M_{Rdzc} = F_{zc} \times [b / 2 - (\min(\lambda_{sb} \times z, b)) / 2]$ = **102.8** kNm

Moment of resistance of reinforcement

Strain in layer 1	$\varepsilon_{z1} = \varepsilon_{cu3} \times (1 - d_{z1} / z) = \mathbf{-0.00028}$
Stress in layer 1	$\sigma_{z1} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{z1}) = \mathbf{-56.7}$ N/mm^2
Force in layer 1	$F_{z1} = N_z \times A_{bar} \times \sigma_{z1} = \mathbf{-166.8}$ kN
Moment of resistance of layer 1	$M_{Rdz1} = F_{z1} \times (b / 2 - d_{z1}) = \mathbf{12.4}$ kNm
Strain in layer 2	$\varepsilon_{z2} = \varepsilon_{cu3} \times (1 - d_{z2} / z) = \mathbf{0.00113}$
Stress in layer 2	$\sigma_{z2} = \min(f_{yd}, E_s \times \varepsilon_{z2}) - \eta \times f_{cd} = \mathbf{207.8}$ N/mm^2
Force in layer 2	$F_{z2} = 2 \times A_{bar} \times \sigma_{z2} = \mathbf{204.0}$ kN
Moment of resistance of layer 2	$M_{Rdz2} = F_{z2} \times (b / 2 - d_{z2}) = \mathbf{0.0}$ kNm
Strain in layer 3	$\varepsilon_{z3} = \varepsilon_{cu3} \times (1 - d_{z3} / z) = \mathbf{0.00254}$
Stress in layer 3	$\sigma_{z3} = \min(f_{yd}, E_s \times \varepsilon_{z3}) - \eta \times f_{cd} = \mathbf{416.6}$ N/mm^2
Force in layer 3	$F_{z3} = N_z \times A_{bar} \times \sigma_{z3} = \mathbf{1227.1}$ kN
Moment of resistance of layer 3	$M_{Rdz3} = F_{z3} \times (b / 2 - d_{z3}) = \mathbf{91.4}$ kNm
Resultant concrete/steel force	$F_z = \mathbf{3272.3}$ kN

PASS - This is within half of one percent of the applied axial load

Combined moment of resistance

Moment of resistance about z axis	$M_{Rdz} = \mathbf{206.6}$ kNm

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

Biaxial bending

Determine if a biaxial bending check is required (5.8.9(3))

Ratio of column slenderness ratios	$ratio_{\lambda} = \max(\lambda_y, \lambda_z) / \min(\lambda_y, \lambda_z) = \mathbf{3.00}$
Eccentricity in direction of y axis	$e_y = M_{Edz} / N_{Ed} = \mathbf{54.9}$ mm
Eccentricity in direction of z axis	$e_z = M_{Edy} / N_{Ed} = \mathbf{45.2}$ mm
Equivalent depth	$h_{eq} = i_y \times \sqrt{(12)} = \mathbf{750}$ mm
Equivalent width	$b_{eq} = i_z \times \sqrt{(12)} = \mathbf{250}$ mm
Relative eccentricity in direction of y axis	$e_{rel\_y} = e_y / b_{eq} = \mathbf{0.219}$
Relative eccentricity in direction of z axis	$e_{rel\_z} = e_z / h_{eq} = \mathbf{0.060}$
Ratio of relative eccentricities	$ratio_e = \min(e_{rel\_y}, e_{rel\_z}) / \max(e_{rel\_y}, e_{rel\_z}) = \mathbf{0.275}$

$ratio_{\lambda}$ > 2 & $ratio_e$ > 0.2 - Biaxial bending check is required

Biaxial bending (5.8.9(4))

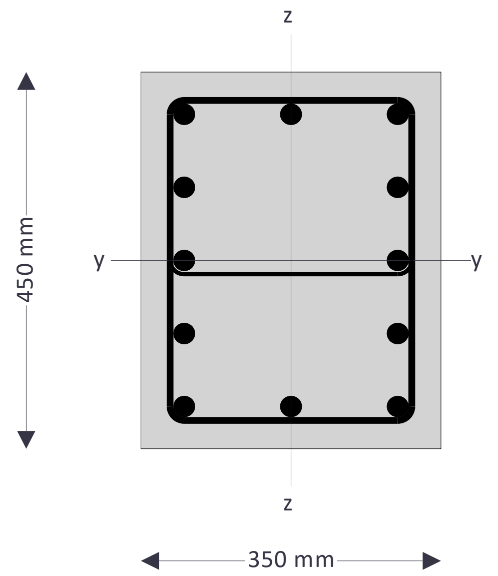
Design axial resistance of section	$N_{Rd} = (A_c \times f_{cd}) + (A_s \times f_{yd}) = \mathbf{6387.9}$ kN
Ratio of applied to resistance axial loads	$ratio_N = N_{Ed} / N_{Rd} = \mathbf{0.515}$
Exponent a	$a = \mathbf{1.35}$
Biaxial bending utilisation	$UF = (M_{Edy} / M_{Rdy})^a + (M_{Edz} / M_{Rdz})^a = \mathbf{0.965}$

PASS - The biaxial bending capacity is adequate

RC COLUMN DESIGN

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

Tedds calculation version 1.3.02



12 no. 25 mm diameter longitudinal bars

Max link spacing 350 mm generally, 210 mm for 450 mm above and below slab/beam and at laps

Column input details

Column geometry

Overall depth (perpendicular to y axis)	h = **450** mm
Overall breadth (perpendicular to z axis)	b = **350** mm
Stability in the z direction	**Braced**
Stability in the y direction	**Braced**

Concrete details

Concrete strength class	**C32/40**
Partial safety factor for concrete (2.4.2.4(1))	γ_C = **1.50**
Coefficient α_{cc} (3.1.6(1))	α_{cc} = **0.85**
Maximum aggregate size	d_g = **20** mm

Reinforcement details

Nominal cover to links	c_{nom} = **30** mm
Longitudinal bar diameter	ϕ = **25** mm
Link diameter	ϕ_v = **8** mm
Total number of longitudinal bars	N = **12**
No. of bars per face parallel to y axis	N_y = **3**
No. of bars per face parallel to z axis	N_z = **5**
Area of longitudinal reinforcement	$A_s = N \times \pi \times \phi^2 / 4$ = **5890** mm^2
Characteristic yield strength	f_{yk} = **500** N/mm^2
Partial safety factor for reinft (2.4.2.4(1))	γ_S = **1.15**
Modulus of elasticity of reinft (3.2.7(4))	E_s = **200** kN/mm^2

Fire resistance details

Fire resistance period	R = **60** min
Exposure to fire	**Exposed on one side only**

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} = **1901.0** kN

Moment about y axis at top M_{topy} = **75.0** kNm

Moment about y axis at bottom M_{btmy} = **75.0** kNm

Moment about z axis at top M_{topz} = **50.0** kNm

Moment about z axis at bottom M_{btmz} = **50.0** kNm

Column effective lengths

Effective length for buckling about y axis l_{0y} = **5925** mm

Effective length for buckling about z axis l_{0z} = **5925** mm

Calculated column properties

Concrete properties

Area of concrete $A_c = h \times b$ = **157500** mm^2

Characteristic compression cylinder strength f_{ck} = **32** N/mm^2

Design compressive strength (3.1.6(1)) $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **18.1** N/mm^2

Mean value of cylinder strength (Table 3.1) $f_{cm} = f_{ck}$ + 8 MPa = **40.0** N/mm^2

Secant modulus of elasticity (Table 3.1) E_{cm} = 22000 MPa $\times$ (f_{cm} / 10 MPa)$^{0.3}$ = **33.3** kN/mm^2

Rectangular stress block factors

Depth factor (3.1.7(3)) λ_{sb} = 0.8

Stress factor (3.1.7(3)) η = 1.0

Strain limits

Compression strain limit (Table 3.1) ε_{cu3} = **0.00350**

Pure compression strain limit (Table 3.1) ε_{c3} = **0.00175**

Design yield strength of reinforcement

Design yield strength (3.2.7(2)) $f_{yd} = f_{yk} / \gamma_S$ = **434.8** N/mm^2

Check nominal cover for fire and bond requirements

Min. cover reqd for bond (to links) (4.4.1.2(3)) $c_{min,b} = \max(\phi_v, \phi - \phi_v)$ = **17** mm

Min axis distance for fire (EN1992-1-2 T 5.2a) a_{fi} = **25** mm

Allowance for deviations from min cover (4.4.1.3) Δc_{dev} = **5** mm

Min allowable nominal cover $c_{nom\_min} = \max(a_{fi} - \phi / 2 - \phi_v, c_{min,b} + \Delta c_{dev})$ = **22.0** mm

PASS - the nominal cover is greater than the minimum required

Effective depths of bars for bending about y axis

Area per bar $A_{bar} = \pi \times \phi^2 / 4$ = **491** mm^2

Spacing of bars in faces parallel to z axis (c/c) $s_z = (h - 2 \times (c_{nom} + \phi_v) - \phi) / (N_z - 1)$ = **87** mm

Layer 1 (in tension face) $d_{y1} = h - c_{nom} - \phi_v - \phi / 2$ = **399** mm

Layer 2 $d_{y2} = d_{y1} - s_z$ = **312** mm

Layer 3 $d_{y3} = d_{y2} - s_z$ = **225** mm

Layer 4 $d_{y4} = d_{y3} - s_z$ = **138** mm

Layer 5 $d_{y5} = d_{y4} - s_z$ = **50** mm

2nd moment of area of reinft about y axis $I_{sy} = 2 \times A_{bar} \times [N_y \times (d_{y1}-h/2)^2 + 2 \times (d_{y2} - h/2)^2]$ = **10463** cm^4

Radius of gyration of reinft about y axis $i_{sy} = \sqrt{(I_{sy} / A_s)}$ = **133** mm

Effective depth about y axis (5.8.8.3(2)) $d_y = h / 2 + i_{sy}$ = **358** mm

Effective depths of bars for bending about z axis

Area of per bar $A_{bar} = \pi \times \phi^2 / 4$ = **491** mm^2

Spacing of bars in faces parallel to y axis (c/c) $s_y = (b - 2 \times (c_{nom} + \phi_v) - \phi) / (N_y - 1)$ = **125** mm

Layer 1 (in tension face) $d_{z1} = b - c_{nom} - \phi_v - \phi / 2$ = **300** mm

Layer 2 $d_{z2} = d_{z1} - s_y$ = **175** mm

Layer 3 $d_{z3} = d_{z2} - s_y$ = **50** mm

2nd moment of area of reinft about z axis $I_{sz} = 2 \times A_{bar} \times N_z \times (d_{z1}-b/2)^2$ = **7609** cm^4

Radius of gyration of reinft about z axis $i_{sz} = \sqrt{(I_{sz} / A_s)}$ = **114** mm

Effective depth about z axis (5.8.8.3(2)) $d_z = b / 2 + i_{sz}$ = **289** mm

Column slenderness about y axis

Radius of gyration $i_y = h / \sqrt{(12)}$ = **13.0** cm

Slenderness ratio (5.8.3.2(1)) $\lambda_y = l_{0y} / i_y$ = **45.6**

Column slenderness about z axis

Radius of gyration $i_z = b / \sqrt{(12)}$ = **10.1** cm

Slenderness ratio (5.8.3.2(1)) $\lambda_z = l_{0z} / i_z$ = **58.6**

Design bending moments

Frame analysis moments about y axis combined with moments due to imperfections (cl. 5.2 & 6.1(4))

Ecc. due to geometric imperfections (y axis) $e_{iy} = l_{0y} /400$ = **14.8** mm

Min end moment about y axis $M_{01y} = \min(\text{abs}(M_{topy}), \text{abs}(M_{btmy})) + e_{iy} \times N_{Ed}$ = **103.2** kNm

Max end moment about y axis $M_{02y} = \max(\text{abs}(M_{topy}), \text{abs}(M_{btmy})) + e_{iy} \times N_{Ed}$ = **103.2** kNm

Slenderness limit for buckling about y axis (cl. 5.8.3.1)

Factor A A = **0.7**

Mechanical reinforcement ratio $\omega = A_s \times f_{yd} / (A_c \times f_{cd})$ = **0.897**

Factor B $B = \sqrt{(1 + 2 \times \omega)}$ = **1.671**

Moment ratio $r_{my} = M_{01y} / M_{02y}$ = **1.000**

Factor C $C_y = 1.7 - r_{my}$ = **0.700**

Relative normal force $n = N_{Ed} / (A_c \times f_{cd})$ = **0.666**

Slenderness limit $\lambda_{limy} = 20 \times A \times B \times C_y / \sqrt{(n)}$ = **20.1**

λ_y>=λ_{limy} - Second order effects must be considered

Frame analysis moments about z axis combined with moments due to imperfections (cl. 5.2 & 6.1(4))

Ecc. due to geometric imperfections (z axis) $e_{iz} = l_{0z} /400$ = **14.8** mm

Min end moment about z axis $M_{01z} = \min(\text{abs}(M_{topz}), \text{abs}(M_{btmz})) + e_{iz} \times N_{Ed}$ = **78.2** kNm

Max end moment about z axis $M_{02z} = \max(\text{abs}(M_{topz}), \text{abs}(M_{btmz})) + e_{iz} \times N_{Ed}$ = **78.2** kNm

Slenderness limit for buckling about y axis (cl. 5.8.3.1)

Factor A A = **0.7**

Mechanical reinforcement ratio $\omega = A_s \times f_{yd} / (A_c \times f_{cd})$ = **0.897**

Factor B $B = \sqrt{(1 + 2 \times \omega)}$ = **1.671**

Moment ratio $r_{mz} = M_{01z} / M_{02z}$ = **1.000**

Factor C $C_z = 1.7 - r_{mz}$ = **0.700**

Relative normal force $n = N_{Ed} / (A_c \times f_{cd})$ = **0.666**

Slenderness limit $\lambda_{limz} = 20 \times A \times B \times C_z / \sqrt{(n)}$ = **20.1**

λ_z>=λ_{limz} - Second order effects must be considered

Local second order bending moment about y axis (cl. 5.8.8.2 & 5.8.8.3)

Relative humidity of ambient environment RH = **50** %

Column perimeter in contact with atmosphere u = **1400** mm

Age of concrete at loading	t_0 = **28** day
Parameter n_u	$n_u = 1 + \omega =$ **1.897**
Approx value of n at max moment of resistance	n_{bal} = **0.4**
Axial load correction factor	$K_r = \min(1.0\ ,\ (n_u - n) / (n_u - n_{bal}))$ = **0.823**
Reinforcement design strain	$\varepsilon_{yd} = f_{yd} / E_s$ = **0.00217**
Basic curvature	$curve_{basic\_y} = \varepsilon_{yd} / (0.45 \times d_y)$ = **0.0000135** mm^{-1}
Notional size of column	$h_0 = 2 \times A_c / u$ = **225** mm
Factor α_1 (Annex B.1(1))	$\alpha_1 = (35\ MPa / f_{cm})^{0.7}$ = **0.911**
Factor α_2 (Annex B.1(1))	$\alpha_2 = (35\ MPa / f_{cm})^{0.2}$ = **0.974**
Relative humidity factor (Annex B.1(1))	$\phi_{RH} = [1 + ((1 - RH / 100\%) / (0.1\ mm^{-1/3} \times (h_0)^{1/3})) \times \alpha_1] \times \alpha_2$ = **1.703**
Concrete strength factor (Annex B.1(1))	$\beta_{fcm} = 16.8 \times (1\ MPa)^{1/2} / \sqrt{(f_{cm})}$ = **2.656**
Concrete age factor (Annex B.1(1))	$\beta_{t0} = 1 / (0.1 + (t_0 / 1\ day)^{0.2})$ = **0.488**
Notional creep coefficient (Annex B.1(1))	$\phi_0 = \phi_{RH} \times \beta_{fcm} \times \beta_{t0}$ = **2.209**
Final creep development factor (at t = ¥)	$\beta_{c\infty}$ = 1.0
Final creep coefficient (Annex B.1(1))	$\phi_\infty = \phi_0 \times \beta_{c\infty}$ = **2.209**
Ratio of SLS to ULS moments	r_{My} = **0.80**
Effective creep ratio	$\phi_{efy} = \phi_\infty \times r_{My}$ = **1.767**
Factor β	$\beta_y = 0.35 + f_{ck} / 200\ MPa - \lambda_y / 150$ = **0.206**
Creep factor	$K_{\phi y} = \max(1.0\ ,\ 1 + \beta_y \times \phi_{efy})$ = **1.364**
Modified curvature	$curve_{mod\_y} = K_r \times K_{\phi y} \times curve_{basic\_y}$ = **0.0000151** mm^{-1}
Curvature distribution factor	c = **10**
Deflection	$e_{2y} = curve_{mod\_y} \times l_{0y}^2 / c$ = **53.1** mm
Nominal 2nd order moment	$M_{2y} = N_{Ed} \times e_{2y}$ = **101.0** kNm

Design bending moment about y axis (cl. 5.8.8.2 & 6.1(4))

Equivalent moment from frame analysis	$M_{0ey} = \max(0.6 \times M_{02y} + 0.4 \times M_{01y}, 0.4 \times M_{02y})$ = **103.2** kNm
Design moment	$M_{Edy} = \max(M_{02y}, M_{0ey} + M_{2y}, M_{01y} + 0.5 \times M_{2y}, N_{Ed} \times \max(h/30, 20\ mm))$
	M_{Edy} = **204.1** kNm

Local second order bending moment about z axis (cl. 5.8.8.2 & 5.8.8.3)

Basic curvature	$curve_{basic\_z} = \varepsilon_{yd} / (0.45 \times d_z)$ = **0.0000167** mm^{-1}
Ratio of SLS to ULS moments	r_{Mz} = **0.80**
Effective creep ratio (5.8.4(2))	$\phi_{efz} = \phi_\infty \times r_{Mz}$ = **1.767**
Factor β	$\beta_z = 0.35 + f_{ck} / 200\ MPa - \lambda_z / 150$ = **0.119**
Creep factor	$K_{\phi z} = \max(1.0\ ,\ 1 + \beta_z \times \phi_{efz})$ = **1.210**
Modified curvature	$curve_{mod\_z} = K_r \times K_{\phi z} \times curve_{basic\_z}$ = **0.0000167** mm^{-1}
Curvature distribution factor	c = **10**
Deflection	$e_{2z} = curve_{mod\_z} \times l_{0z}^2 / c$ = **58.5** mm
Nominal 2nd order moment	$M_{2z} = N_{Ed} \times e_{2z}$ = **111.2** kNm

Design bending moment about z axis (cl. 5.8.8.2 & 6.1(4))

Equivalent moment from frame analysis	$M_{0ez} = \max(0.6 \times M_{02z} + 0.4 \times M_{01z}, 0.4 \times M_{02z})$ = **78.2** kNm
Design moment	$M_{Edz} = \max(M_{02z}, M_{0ez} + M_{2z}, M_{01z} + 0.5 \times M_{2z}, N_{Ed} \times \max(b/30, 20\ mm))$
	M_{Edz} = **189.4** kNm

Moment capacity about y axis with axial load N_{Ed}

Moment of resistance of concrete

By iteration:-

Position of neutral axis $y = \mathbf{274.5}$ mm

Concrete compression force (3.1.7(3)) $F_{yc} = \eta \times f_{cd} \times \min(\lambda_{sb} \times y, h) \times b = \mathbf{1394.0}$ kN

Moment of resistance $M_{Rdyc} = F_{yc} \times [h / 2 - (\min(\lambda_{sb} \times y, h)) / 2] = \mathbf{160.6}$ kNm

Moment of resistance of reinforcement

Strain in layer 1 $\varepsilon_{y1} = \varepsilon_{cu3} \times (1 - d_{y1} / y) = \mathbf{-0.00159}$

Stress in layer 1 $\sigma_{y1} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{y1}) = \mathbf{-318.6}$ N/mm^2

Force in layer 1 $F_{y1} = N_y \times A_{bar} \times \sigma_{y1} = \mathbf{-469.1}$ kN

Moment of resistance of layer 1 $M_{Rdy1} = F_{y1} \times (h / 2 - d_{y1}) = \mathbf{81.9}$ kNm

Strain in layer 2 $\varepsilon_{y2} = \varepsilon_{cu3} \times (1 - d_{y2} / y) = \mathbf{-0.00048}$

Stress in layer 2 $\sigma_{y2} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{y2}) = \mathbf{-96.1}$ N/mm^2

Force in layer 2 $F_{y2} = 2 \times A_{bar} \times \sigma_{y2} = \mathbf{-94.4}$ kN

Moment of resistance of layer 2 $M_{Rdy2} = F_{y2} \times (h / 2 - d_{y2}) = \mathbf{8.2}$ kNm

Strain in layer 3 $\varepsilon_{y3} = \varepsilon_{cu3} \times (1 - d_{y3} / y) = \mathbf{0.00063}$

Stress in layer 3 $\sigma_{y3} = \min(f_{yd}, E_s \times \varepsilon_{y3}) = \mathbf{126.3}$ N/mm^2

Force in layer 3 $F_{y3} = 2 \times A_{bar} \times \sigma_{y3} = \mathbf{124.0}$ kN

Moment of resistance of layer 3 $M_{Rdy3} = F_{y3} \times (h / 2 - d_{y3}) = \mathbf{0.0}$ kNm

Strain in layer 4 $\varepsilon_{y4} = \varepsilon_{cu3} \times (1 - d_{y4} / y) = \mathbf{0.00174}$

Stress in layer 4 $\sigma_{y4} = \min(f_{yd}, E_s \times \varepsilon_{y4}) - \eta \times f_{cd} = \mathbf{330.7}$ N/mm^2

Force in layer 4 $F_{y4} = 2 \times A_{bar} \times \sigma_{y4} = \mathbf{324.6}$ kN

Moment of resistance of layer 4 $M_{Rdy4} = F_{y4} \times (h / 2 - d_{y4}) = \mathbf{28.3}$ kNm

Strain in layer 5 $\varepsilon_{y5} = \varepsilon_{cu3} \times (1 - d_{y5} / y) = \mathbf{0.00286}$

Stress in layer 5 $\sigma_{y5} = \min(f_{yd}, E_s \times \varepsilon_{y5}) - \eta \times f_{cd} = \mathbf{416.6}$ N/mm^2

Force in layer 5 $F_{y5} = N_y \times A_{bar} \times \sigma_{y5} = \mathbf{613.6}$ kN

Moment of resistance of layer 5 $M_{Rdy5} = F_{y5} \times (h / 2 - d_{y5}) = \mathbf{107.1}$ kNm

Resultant concrete/steel force $F_y = \mathbf{1892.7}$ kN

PASS - This is within half of one percent of the applied axial load

Combined moment of resistance

Moment of resistance about y axis $M_{Rdy} = \mathbf{386.0}$ kNm

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

Moment capacity about z axis with axial load N_{Ed}

Moment of resistance of concrete

By iteration:-

Position of neutral axis $z = \mathbf{215.6}$ mm

Concrete compression force (3.1.7(3)) $F_{zc} = \eta \times f_{cd} \times \min(\lambda_{sb} \times z, b) \times h = \mathbf{1407.7}$ kN

Moment of resistance $M_{Rdzc} = F_{zc} \times [b / 2 - (\min(\lambda_{sb} \times z, b)) / 2] = \mathbf{124.9}$ kNm

Moment of resistance of reinforcement

Strain in layer 1 $\varepsilon_{z1} = \varepsilon_{cu3} \times (1 - d_{z1} / z) = \mathbf{-0.00136}$

Stress in layer 1 $\sigma_{z1} = \max(-1 \times f_{yd}, E_s \times \varepsilon_{z1}) = \mathbf{-272.2}$ N/mm^2

Force in layer 1 $F_{z1} = N_z \times A_{bar} \times \sigma_{z1} = \mathbf{-668.1}$ kN

Moment of resistance of layer 1 $M_{Rdz1} = F_{z1} \times (b / 2 - d_{z1}) = \mathbf{83.2}$ kNm

Strain in layer 2 $\varepsilon_{z2} = \varepsilon_{cu3} \times (1 - d_{z2} / z) = \mathbf{0.00066}$

Stress in layer 2 $\sigma_{z2} = \min(f_{yd}, E_s \times \varepsilon_{z2}) = \mathbf{131.9}$ N/mm^2

Force in layer 2 $F_{z2} = 2 \times A_{bar} \times \sigma_{z2} = \mathbf{129.5}$ kN

Moment of resistance of layer 2 $M_{Rdz2} = F_{z2} \times (b / 2 - d_{z2}) = \mathbf{0.0}$ kNm

Strain in layer 3 $\varepsilon_{z3} = \varepsilon_{cu3} \times (1 - d_{z3} / z) =$ **0.00268**

Stress in layer 3 $\sigma_{z3} = \min(f_{yd}, E_s \times \varepsilon_{z3}) - \eta \times f_{cd} =$ **416.6** N/mm^2

Force in layer 3 $F_{z3} = N_z \times A_{bar} \times \sigma_{z3} =$ **1022.6** kN

Moment of resistance of layer 3 $M_{Rdz3} = F_{z3} \times (b / 2 - d_{z3}) =$ **127.3** kNm

Resultant concrete/steel force $F_z =$ **1891.7** kN

PASS - This is within half of one percent of the applied axial load

Combined moment of resistance

Moment of resistance about z axis $M_{Rdz} =$ **335.4** kNm

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

Biaxial bending

Determine if a biaxial bending check is required (5.8.9(3))

Ratio of column slenderness ratios $ratio_\lambda = \max(\lambda_y, \lambda_z) / \min(\lambda_y, \lambda_z) =$ **1.29**

Eccentricity in direction of y axis $e_y = M_{Edz} / N_{Ed} =$ **99.6** mm

Eccentricity in direction of z axis $e_z = M_{Edy} / N_{Ed} =$ **107.4** mm

Equivalent depth $h_{eq} = i_y \times \sqrt{(12)} =$ **450** mm

Equivalent width $b_{eq} = i_z \times \sqrt{(12)} =$ **350** mm

Relative eccentricity in direction of y axis $e_{rel\_y} = e_y / b_{eq} =$ **0.285**

Relative eccentricity in direction of z axis $e_{rel\_z} = e_z / h_{eq} =$ **0.239**

Ratio of relative eccentricities $ratio_e = \min(e_{rel\_y}, e_{rel\_z}) / \max(e_{rel\_y}, e_{rel\_z}) =$ **0.838**

$ratio_e$ > 0.2 - Biaxial bending check is required

Biaxial bending (5.8.9(4))

Design axial resistance of section $N_{Rd} = (A_c \times f_{cd}) + (A_s \times f_{yd}) =$ **5417.1** kN

Ratio of applied to resistance axial loads $ratio_N = N_{Ed} / N_{Rd} =$ **0.351**

Exponent a $a =$ **1.21**

Biaxial bending utilisation $UF = (M_{Edy} / M_{Rdy})^a + (M_{Edz} / M_{Rdz})^a =$ **0.964**

PASS - The biaxial bending capacity is adequate

RETAINING WALL ANALYSIS

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.9.05

Retaining wall details

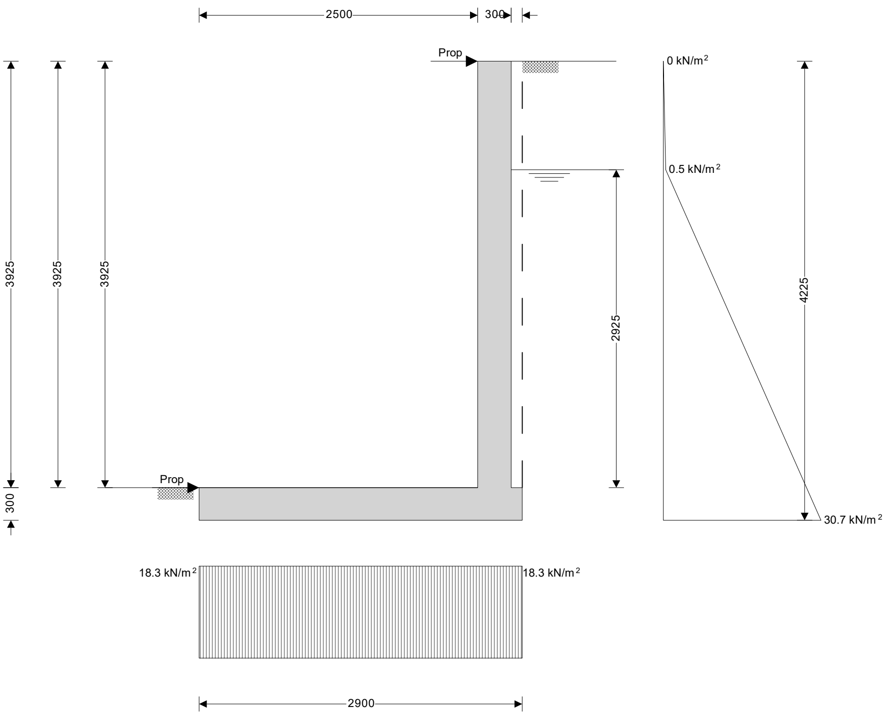
Stem type	Propped cantilever
Stem height	h_{stem} = **3925** mm
Prop height	h_{prop} = **3925** mm
Stem thickness	t_{stem} = **300** mm
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **25** kN/m^3
Toe length	l_{toe} = **2500** mm
Heel length	l_{heel} = **100** mm
Base thickness	t_{base} = **300** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **3925** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Height of water	h_{water} = **2925** mm
Water density	γ_w = **9.8** kN/m^3

Retained soil properties

Soil type	Dense gravel
Moist density	γ_{mr} = **5** kN/m^3
Saturated density	γ_{sr} = **5** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **60** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **30** deg

Base soil properties

Soil type	Stiff clay
Soil density	γ_b = **19** kN/m^3
Characteristic cohesion	$c'_{b.k}$ = **30** kN/m^2
Characteristic effective shear resistance angle	$\phi'_{b.k}$ = **18** deg
Characteristic wall friction angle	$\delta_{b.k}$ = **9** deg
Characteristic base friction angle	$\delta_{bb.k}$ = **12** deg



General arrangement

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem} + l_{heel} = \mathbf{2900}$ mm
Saturated soil height	$h_{sat} = h_{water} + d_{cover} = \mathbf{2925}$ mm
Moist soil height	$h_{moist} = h_{ret} - h_{water} = \mathbf{1000}$ mm
Retained surface length	$l_{sur} = l_{heel} = \mathbf{100}$ mm
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{4225}$ mm
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem} = \mathbf{1.178}$ m^2
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{2650}$ mm
Area of wall base	$A_{base} = l_{base} \times t_{base} = \mathbf{0.87}$ m^2
- Distance to vertical component	$x_{base} = l_{base} / 2 = \mathbf{1450}$ mm
Area of saturated soil	$A_{sat} = h_{sat} \times l_{heel} = \mathbf{0.293}$ m^2
- Distance to vertical component	$x_{sat\_v} = l_{base} - (h_{sat} \times l_{heel}^2 / 2) / A_{sat} = \mathbf{2850}$ mm
- Distance to horizontal component	$x_{sat\_h} = (h_{sat} + h_{base}) / 3 = \mathbf{1075}$ mm
Area of water	$A_{water} = h_{sat} \times l_{heel} = \mathbf{0.293}$ m^2
- Distance to vertical component	$x_{water\_v} = l_{base} - (h_{sat} \times l_{heel}^2 / 2) / A_{sat} = \mathbf{2850}$ mm
- Distance to horizontal component	$x_{water\_h} = (h_{sat} + h_{base}) / 3 = \mathbf{1075}$ mm
Area of moist soil	$A_{moist} = h_{moist} \times l_{heel} = \mathbf{0.1}$ m^2
- Distance to vertical component	$x_{moist\_v} = l_{base} - (h_{moist} \times l_{heel}^2 / 2) / A_{moist} = \mathbf{2850}$ mm
- Distance to horizontal component	$x_{moist\_h} = (h_{moist} \times (t_{base} + h_{sat} + h_{moist} / 3) / 2 + (h_{sat} + t_{base})^2/2) / (h_{sat} + t_{base} + h_{moist} / 2) = \mathbf{1874}$ mm

Design approach 1

Partial factors on actions - Table A.3 - Combination 1

Partial factor set A1

Permanent unfavourable action γ_G = **1.35**

Permanent favourable action γ_{Gf} = **1.00**

Variable unfavourable action γ_Q = **1.50**

Variable favourable action γ_{Qf} = **0.00**

Partial factors for soil parameters – Table A.4 - Combination 1

Soil parameter set M1

Angle of shearing resistance $\gamma_{\phi'}$ = **1.00**

Effective cohesion $\gamma_{c'}$ = **1.00**

Weight density γ_γ = **1.00**

Library item Partial factors output

Water properties

Design water density $\gamma_w' = \gamma_w / \gamma_\gamma$ = **9.8** kN/m^3

Retained soil properties

Design moist density $\gamma_{mr}' = \gamma_{mr} / \gamma_\gamma$ = **5** kN/m^3

Design saturated density $\gamma_{sr}' = \gamma_{sr} / \gamma_\gamma$ = **5** kN/m^3

Design effective shear resistance angle $\phi'_{r.d} = \text{atan}(\tan(\phi'_{r.k}) / \gamma_{\phi'})$ = **60** deg

Design wall friction angle $\delta_{r.d} = \text{atan}(\tan(\delta_{r.k}) / \gamma_{\phi'})$ = **30** deg

Base soil properties

Design soil density $\gamma_b' = \gamma_b / \gamma_\gamma$ = **19** kN/m^3

Design effective shear resistance angle $\phi'_{b.d} = \text{atan}(\tan(\phi'_{b.k}) / \gamma_{\phi'})$ = **18** deg

Design wall friction angle $\delta_{b.d} = \text{atan}(\tan(\delta_{b.k}) / \gamma_{\phi'})$ = **9** deg

Design base friction angle $\delta_{bb.d} = \text{atan}(\tan(\delta_{bb.k}) / \gamma_{\phi'})$ = **12** deg

Design effective cohesion $c'_{b.d} = c'_{b.k} / \gamma_{c'}$ = **30** kN/m^2

Using Coulomb theory

Active pressure coefficient $K_A = \sin(\alpha + \phi'_{r.d})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.d}) \times [1 + \sqrt{[\sin(\phi'_{r.d} + \delta_{r.d}) \times \sin(\phi'_{r.d} - \beta) / (\sin(\alpha - \delta_{r.d}) \times \sin(\alpha + \beta))]}]^2)$ = **0.072**

Passive pressure coefficient $K_P = \sin(90 - \phi'_{b.d})^2 / (\sin(90 + \delta_{b.d}) \times [1 - \sqrt{[\sin(\phi'_{b.d} + \delta_{b.d}) \times \sin(\phi'_{b.d}) / (\sin(90 + \delta_{b.d}))]}]^2)$ = **2.359**

Bearing pressure check

Vertical forces on wall

Wall stem $F_{stem} = \gamma_G \times A_{stem} \times \gamma_{stem}$ = **39.7** kN/m

Wall base $F_{base} = \gamma_G \times A_{base} \times \gamma_{base}$ = **29.4** kN/m

Saturated retained soil $F_{sat\_v} = \gamma_G \times A_{sat} \times (\gamma_{sr}' - \gamma_w')$ = **-1.9** kN/m

Water $F_{water\_v} = \gamma_G \times A_{water} \times \gamma_w'$ = **3.9** kN/m

Moist retained soil $F_{moist\_v} = \gamma_G \times A_{moist} \times \gamma_{mr}'$ = **0.7** kN/m

Total $F_{total\_v} = F_{stem} + F_{base} + F_{sat\_v} + F_{moist\_v} + F_{water\_v}$ = **71.8** kN/m

Horizontal forces on wall

Saturated retained soil $F_{sat\_h} = \gamma_G \times K_A \times \cos(\delta_{r.d}) \times (\gamma_{sr}' - \gamma_w') \times (h_{sat} + h_{base})^2 / 2$ = **-2.1** kN/m

Water $F_{water\_h} = \gamma_G \times \gamma_w' \times (h_{water} + d_{cover} + h_{base})^2 / 2$ = **68.9** kN/m

Moist retained soil — $F_{moist\_h} = \gamma_G \times K_A \times \cos(\delta_{r.d}) \times \gamma_{mr}' \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) =$ **1.6** kN/m

Base soil — $F_{pass\_h} = -\gamma_{Gf} \times K_P \times \cos(\delta_{b.d}) \times \gamma_b' \times (d_{cover} + h_{base})^2 / 2 =$ **-2** kN/m

Total — $F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} =$ **66.3** kN/m

Moments on wall

Wall stem — $M_{stem} = F_{stem} \times x_{stem} =$ **105.3** kNm/m

Wall base — $M_{base} = F_{base} \times x_{base} =$ **42.6** kNm/m

Saturated retained soil — $M_{sat} = F_{sat\_v} \times x_{sat\_v} - F_{sat\_h} \times x_{sat\_h} =$ **-3.1** kNm/m

Water — $M_{water} = F_{water\_v} \times x_{water\_v} - F_{water\_h} \times x_{water\_h} =$ **-63** kNm/m

Moist retained soil — $M_{moist} = F_{moist\_v} \times x_{moist\_v} - F_{moist\_h} \times x_{moist\_h} =$ **-1** kNm/m

Total — $M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{water} =$ **80.7** kNm/m

Check bearing pressure

Propping force to stem — $F_{prop\_stem} = (F_{total\_v} \times l_{base} / 2 - M_{total}) / (h_{prop} + t_{base}) =$ **5.5** kN/m

Propping force to base — $F_{prop\_base} = F_{total\_h} - F_{prop\_stem} =$ **60.8** kN/m

Moment from propping force — $M_{prop} = F_{prop\_stem} \times (h_{prop} + t_{base}) =$ **23.3** kNm/m

Distance to reaction — $\bar{x} = (M_{total} + M_{prop}) / F_{total\_v} =$ **1450** mm

Eccentricity of reaction — $e = \bar{x} - l_{base} / 2 =$ **0** mm

Loaded length of base — $l_{load} = l_{base} =$ **2900** mm

Bearing pressure at toe — $q_{toe} = F_{total\_v} / l_{base} =$ **24.7** kN/m^2

Bearing pressure at heel — $q_{heel} = F_{total\_v} / l_{base} =$ **24.7** kN/m^2

Effective overburden pressure — $q = \max((t_{base} + d_{cover}) \times \gamma_b' - (t_{base} + d_{cover} + h_{water}) \times \gamma_w', 0\ kN/m^2) =$ **0** kN/m^2

Design effective overburden pressure — $q' = q / \gamma_\gamma =$ **0** kN/m^2

Bearing resistance factors — $N_q = Exp(\pi \times \tan(\phi'_{b.d})) \times (\tan(45\ deg + \phi'_{b.d} / 2))^2 =$ **5.258**

$N_c = (N_q - 1) \times \cot(\phi'_{b.d}) =$ **13.104**

$N_\gamma = 2 \times (N_q - 1) \times \tan(\phi'_{b.d}) =$ **2.767**

Foundation shape factors — $s_q = 1$

$s_\gamma = 1$

$s_c = 1$

Load inclination factors — $H = F_{sat\_h} + F_{water\_h} + F_{moist\_h} + F_{pass\_h} - F_{prop\_stem} - F_{prop\_base} =$ **0** kN/m

$V = F_{total\_v} =$ **71.8** kN/m

$m = 2$

$i_q = [1 - H / (V + l_{load} \times c'_{b.d} \times \cot(\phi'_{b.d}))]^m =$ **1**

$i_\gamma = [1 - H / (V + l_{load} \times c'_{b.d} \times \cot(\phi'_{b.d}))]^{(m + 1)} =$ **1**

$i_c = i_q - (1 - i_q) / (N_c \times \tan(\phi'_{b.d})) =$ **1**

Net ultimate bearing capacity

$n_f = c'_{b.d} \times N_c \times s_c \times i_c + q' \times N_q \times s_q \times i_q + 0.5 \times (\gamma_b' - \gamma_w') \times l_{load} \times N_\gamma \times s_\gamma \times i_\gamma =$ **430** kN/m^2

Factor of safety — $FoS_{bp} = n_f / \max(q_{toe}, q_{heel}) =$ **17.378**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

Design approach 1

Partial factors on actions - Table A.3 - Combination 2

Partial factor set — A2

Permanent unfavourable action — $\gamma_G =$ **1.00**

Permanent favourable action — $\gamma_{Gf} =$ **1.00**

Variable unfavourable action — $\gamma_Q =$ **1.30**

Variable favourable action γ_{Qf} = **0.00**

Partial factors for soil parameters – Table A.4 - Combination 2

Soil parameter set M2

Angle of shearing resistance $\gamma_{\phi'}$ = **1.25**

Effective cohesion $\gamma_{c'}$ = **1.25**

Weight density γ_{γ} = **1.00**

Library item Partial factors output

Water properties

Design water density $\gamma_w' = \gamma_w / \gamma_\gamma$ = **9.8** kN/m^3

Retained soil properties

Design moist density $\gamma_{mr}' = \gamma_{mr} / \gamma_\gamma$ = **5** kN/m^3

Design saturated density $\gamma_{sr}' = \gamma_{sr} / \gamma_\gamma$ = **5** kN/m^3

Design effective shear resistance angle $\phi'_{r.d} = \text{atan}(\tan(\phi'_{r.k}) / \gamma_{\phi'})$ = **54.2** deg

Design wall friction angle $\delta_{r.d} = \text{atan}(\tan(\delta_{r.k}) / \gamma_{\phi'})$ = **24.8** deg

Base soil properties

Design soil density $\gamma_b' = \gamma_b / \gamma_\gamma$ = **19** kN/m^3

Design effective shear resistance angle $\phi'_{b.d} = \text{atan}(\tan(\phi'_{b.k}) / \gamma_{\phi'})$ = **14.6** deg

Design wall friction angle $\delta_{b.d} = \text{atan}(\tan(\delta_{b.k}) / \gamma_{\phi'})$ = **7.2** deg

Design base friction angle $\delta_{bb.d} = \text{atan}(\tan(\delta_{bb.k}) / \gamma_{\phi'})$ = **9.7** deg

Design effective cohesion $c'_{b.d} = c'_{b.k} / \gamma_{c'}$ = **24** kN/m^2

Using Coulomb theory

Active pressure coefficient $K_A = \sin(\alpha + \phi'_{r.d})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.d}) \times [1 + \sqrt{[}\sin(\phi'_{r.d} + \delta_{r.d}) \times \sin(\phi'_{r.d} - \beta) / (\sin(\alpha - \delta_{r.d}) \times \sin(\alpha + \beta))]]^2)$ = **0.101**

Passive pressure coefficient $K_P = \sin(90 - \phi'_{b.d})^2 / (\sin(90 + \delta_{b.d}) \times [1 - \sqrt{[}\sin(\phi'_{b.d} + \delta_{b.d}) \times \sin(\phi'_{b.d}) / (\sin(90 + \delta_{b.d}))]]^2)$ = **1.965**

Bearing pressure check

Vertical forces on wall

Wall stem $F_{stem} = \gamma_G \times A_{stem} \times \gamma_{stem}$ = **29.4** kN/m

Wall base $F_{base} = \gamma_G \times A_{base} \times \gamma_{base}$ = **21.8** kN/m

Saturated retained soil $F_{sat\_v} = \gamma_G \times A_{sat} \times (\gamma_{sr}' - \gamma_w')$ = **-1.4** kN/m

Water $F_{water\_v} = \gamma_G \times A_{water} \times \gamma_w'$ = **2.9** kN/m

Moist retained soil $F_{moist\_v} = \gamma_G \times A_{moist} \times \gamma_{mr}'$ = **0.5** kN/m

Total $F_{total\_v} = F_{stem} + F_{base} + F_{sat\_v} + F_{moist\_v} + F_{water\_v}$ = **53.2** kN/m

Horizontal forces on wall

Saturated retained soil $F_{sat\_h} = \gamma_G \times K_A \times \cos(\delta_{r.d}) \times (\gamma_{sr}' - \gamma_w') \times (h_{sat} + h_{base})^2 / 2$ = **-2.3** kN/m

Water $F_{water\_h} = \gamma_G \times \gamma_w' \times (h_{water} + d_{cover} + h_{base})^2 / 2$ = **51** kN/m

Moist retained soil $F_{moist\_h} = \gamma_G \times K_A \times \cos(\delta_{r.d}) \times \gamma_{mr}' \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base}))$ = **1.7** kN/m

Base soil $F_{pass\_h} = -\gamma_{Gf} \times K_P \times \cos(\delta_{b.d}) \times \gamma_b' \times (d_{cover} + h_{base})^2 / 2$ = **-1.7** kN/m

Total $F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h}$ = **48.8** kN/m

Moments on wall

Wall stem $M_{stem} = F_{stem} \times x_{stem}$ = **78** kNm/m

Wall base $M_{base} = F_{base} \times x_{base}$ = **31.5** kNm/m

Saturated retained soil	$M_{sat} = F_{sat\_v} \times x_{sat\_v} - F_{sat\_h} \times x_{sat\_h} = \mathbf{-1.6}$ kNm/m
Water	$M_{water} = F_{water\_v} \times x_{water\_v} - F_{water\_h} \times x_{water\_h} = \mathbf{-46.7}$ kNm/m
Moist retained soil	$M_{moist} = F_{moist\_v} \times x_{moist\_v} - F_{moist\_h} \times x_{moist\_h} = \mathbf{-1.8}$ kNm/m
Total	$M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{water} = \mathbf{59.6}$ kNm/m

Check bearing pressure

Propping force to stem	$F_{prop\_stem} = (F_{total\_v} \times l_{base} / 2 - M_{total}) / (h_{prop} + t_{base}) = \mathbf{4.1}$ kN/m
Propping force to base	$F_{prop\_base} = F_{total\_h} - F_{prop\_stem} = \mathbf{44.6}$ kN/m
Moment from propping force	$M_{prop} = F_{prop\_stem} \times (h_{prop} + t_{base}) = \mathbf{17.5}$ kNm/m
Distance to reaction	$\bar{x} = (M_{total} + M_{prop}) / F_{total\_v} = \mathbf{1450}$ mm
Eccentricity of reaction	$e = \bar{x} - l_{base} / 2 = \mathbf{0}$ mm
Loaded length of base	$l_{load} = l_{base} = \mathbf{2900}$ mm
Bearing pressure at toe	$q_{toe} = F_{total\_v} / l_{base} = \mathbf{18.3}$ kN/m^2
Bearing pressure at heel	$q_{heel} = F_{total\_v} / l_{base} = \mathbf{18.3}$ kN/m^2
Effective overburden pressure	$q = \max((t_{base} + d_{cover}) \times \gamma_b' - (t_{base} + d_{cover} + h_{water}) \times \gamma_w', 0 \text{ kN/m}^2) = \mathbf{0}$ kN/m^2
Design effective overburden pressure	$q' = q / \gamma_\gamma = \mathbf{0}$ kN/m^2
Bearing resistance factors	$N_q = \text{Exp}(\pi \times \tan(\phi'_{b.d})) \times (\tan(45 \text{ deg} + \phi'_{b.d} / 2))^2 = \mathbf{3.784}$
	$N_c = (N_q - 1) \times \cot(\phi'_{b.d}) = \mathbf{10.711}$
	$N_\gamma = 2 \times (N_q - 1) \times \tan(\phi'_{b.d}) = \mathbf{1.447}$
Foundation shape factors	$s_q = 1$
	$s_\gamma = 1$
	$s_c = 1$
Load inclination factors	$H = F_{sat\_h} + F_{water\_h} + F_{moist\_h} + F_{pass\_h} - F_{prop\_stem} - F_{prop\_base} = \mathbf{0}$ kN/m
	$V = F_{total\_v} = \mathbf{53.2}$ kN/m
	$m = 2$
	$i_q = [1 - H / (V + l_{load} \times c'_{b.d} \times \cot(\phi'_{b.d}))]^m = \mathbf{1}$
	$i_\gamma = [1 - H / (V + l_{load} \times c'_{b.d} \times \cot(\phi'_{b.d}))]^{(m + 1)} = \mathbf{1}$
	$i_c = i_q - (1 - i_q) / (N_c \times \tan(\phi'_{b.d})) = \mathbf{1}$

Net ultimate bearing capacity

$n_f = c'_{b.d} \times N_c \times s_c \times i_c + q' \times N_q \times s_q \times i_q + 0.5 \times (\gamma_b' - \gamma_w') \times l_{load} \times N_\gamma \times s_\gamma \times i_\gamma = \mathbf{276.3}$ kN/m^2

Factor of safety $FoS_{bp} = n_f / \max(q_{toe}, q_{heel}) = \mathbf{15.078}$

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 2.9.05

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

Concrete strength class	C32/40
Characteristic compressive cylinder strength	$f_{ck} = \mathbf{32}$ N/mm^2
Characteristic compressive cube strength	$f_{ck,cube} = \mathbf{40}$ N/mm^2
Mean value of compressive cylinder strength	$f_{cm} = f_{ck} + 8 \text{ N/mm}^2 = \mathbf{40}$ N/mm^2
Mean value of axial tensile strength	$f_{ctm} = 0.3 \text{ N/mm}^2 \times (f_{ck} / 1 \text{ N/mm}^2)^{2/3} = \mathbf{3.0}$ N/mm^2
5% fractile of axial tensile strength	$f_{ctk,0.05} = 0.7 \times f_{ctm} = \mathbf{2.1}$ N/mm^2
Secant modulus of elasticity of concrete	$E_{cm} = 22 \text{ kN/mm}^2 \times (f_{cm} / 10 \text{ N/mm}^2)^{0.3} = \mathbf{33346}$ N/mm^2
Partial factor for concrete - Table 2.1N	$\gamma_C = \mathbf{1.50}$

Compressive strength coefficient - cl.3.1.6(1) α_{cc} = **0.85**

Design compressive concrete strength - exp.3.15 $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **18.1** N/mm^2

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

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**

Bending coefficient k_1 K_1 = **0.40**

Bending coefficient k_2 $K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ = **1.00**

Bending coefficient k_3 K_3 =**0.40**

Bending coefficient k_4 $K_4 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ =**1.00**

Reinforcement details

Characteristic yield strength of reinforcement f_{yk} = **500** N/mm^2

Modulus of elasticity of reinforcement E_s = **200000** 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

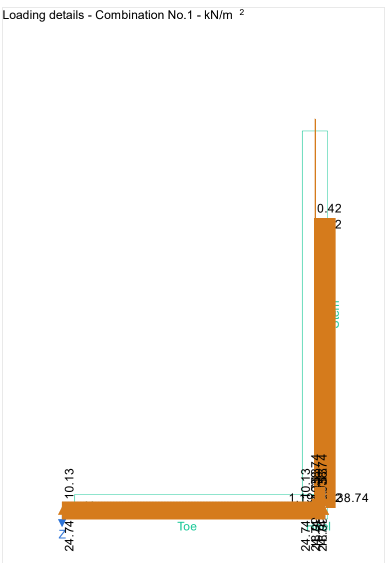
Cover to reinforcement

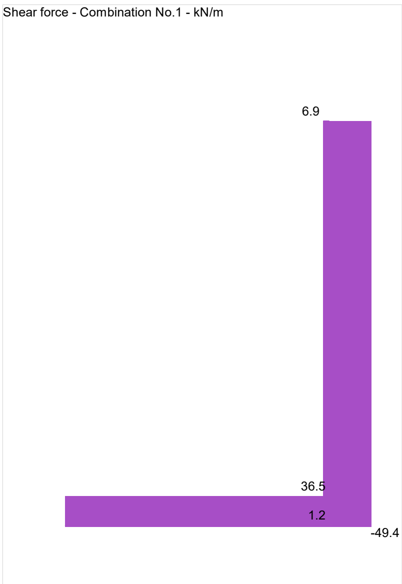
Front face of stem c_{sf} = **40** mm

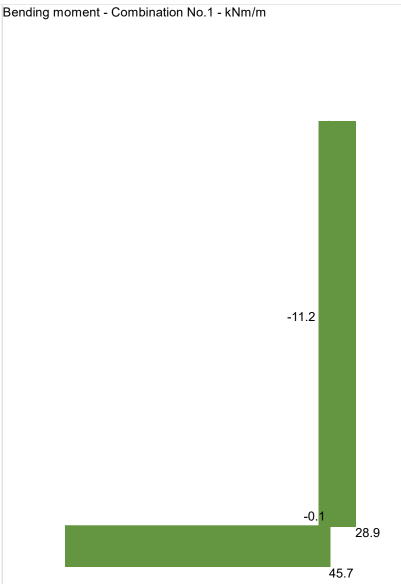
Rear face of stem c_{sr} = **50** mm

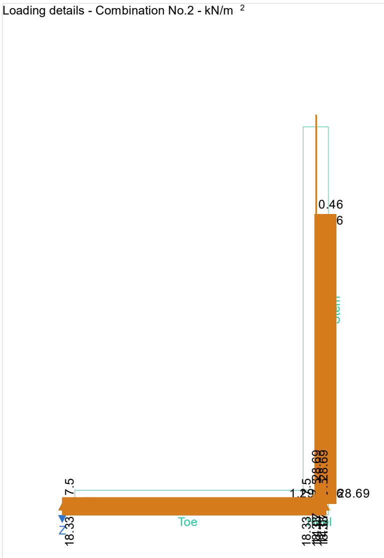
Top face of base c_{bt} = **50** mm

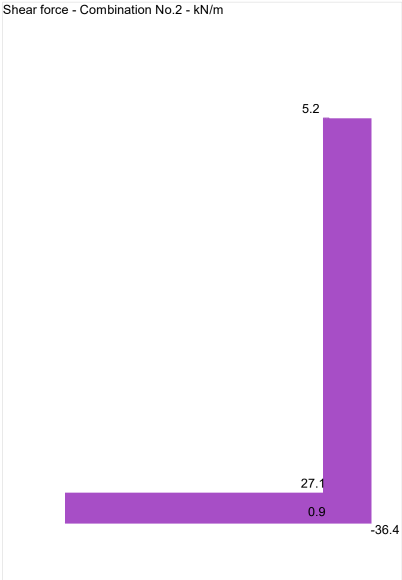
Bottom face of base c_{bb} = **75** mm

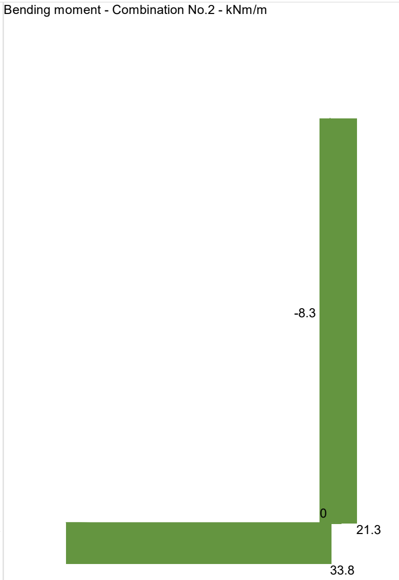












Check stem design at 1935 mm

Depth of section $h = \mathbf{300}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 $M = \mathbf{11.2}$ kNm/m

Depth to tension reinforcement $d = h - c_{sf} - \phi_{sx} - \phi_{sfM} / 2 = \mathbf{242}$ mm

$K = M / (d^2 \times f_{ck}) = \mathbf{0.006}$

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

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = \mathbf{230}$ mm

Depth of neutral axis $x = 2.5 \times (d – z) = \mathbf{30}$ mm

Area of tension reinforcement required $A_{sfM.req} = M / (f_{yd} \times z) = \mathbf{112}$ mm^2/m

Tension reinforcement provided 12 dia.bars @ 150 c/c

Area of tension reinforcement provided $A_{sfM.prov} = \pi \times \phi_{sfM}^2 / (4 \times s_{sfM}) = \mathbf{754}$ mm^2/m

Minimum area of reinforcement - exp.9.1N $A_{sfM.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{381}$ mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{sfM.max} = 0.04 \times h = \mathbf{12000}$ mm^2/m

$\max(A_{sfM.req}, A_{sfM.min}) / A_{sfM.prov} = \mathbf{0.505}$

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

Library item: Rectangular single output

Deflection control - Section 7.4

Reference reinforcement ratio $\rho_0 = \sqrt{}(f_{ck} / 1 \text{ N/mm}^2) / 1000 = \mathbf{0.006}$

Required tension reinforcement ratio $\rho = A_{sfM.req} / d = \mathbf{0.000}$

Required compression reinforcement ratio $\rho' = A_{sfM.2.req} / d_2 = \mathbf{0.000}$

Structural system factor - Table 7.4N $K_b = \mathbf{1}$

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

Limiting span to depth ratio - exp.7.16.a $\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{}(f_{ck} / 1 \text{ N/mm}^2) \times \rho_0 / \rho + 3.2 \times \sqrt{}(f_{ck} / 1 \text{ N/mm}^2) \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b) = \mathbf{40}$

Actual span to depth ratio $h_{prop} / d = \mathbf{16.2}$

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width — w_{max} = **0.2** mm

Variable load factor - EN1990 – Table A1.1 — ψ_2 = **0.3**

Serviceability bending moment — M_{sls} = **8.3** kNm/m

Tensile stress in reinforcement — $\sigma_s = M_{sls} / (A_{sfM.prov} \times z)$ = **47.9** N/mm^2

Load duration — Long term

Load duration factor — k_t = **0.4**

Effective area of concrete in tension — $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$

$A_{c.eff}$ = **89917** mm^2/m

Mean value of concrete tensile strength — $f_{ct.eff} = f_{ctm}$ = **3.0** N/mm^2

Reinforcement ratio — $\rho_{p.eff} = A_{sfM.prov} / A_{c.eff}$ = **0.008**

Modular ratio — $\alpha_e = E_s / E_{cm}$ = **5.998**

Bond property coefficient — k_1 = **0.8**

Strain distribution coefficient — k_2 = **0.5**

k_3 = **3.4**

k_4 = **0.425**

Maximum crack spacing - exp.7.11 — $s_{r.max} = k_3 \times c_{sf} + k_1 \times k_2 \times k_4 \times \phi_{sfM} / \rho_{p.eff}$ = **379** mm

Maximum crack width - exp.7.8 — $w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

w_k = **0.055** mm

w_k / w_{max} = **0.273**

PASS - Maximum crack width is less than limiting crack width

Check stem design at base of stem

Depth of section — h = **300** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — M = **28.9** kNm/m

Depth to tension reinforcement — $d = h - c_{sr} - \phi_{sr} / 2$ = **244** mm

$K = M / (d^2 \times f_{ck})$ = **0.015**

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

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = **232** mm

Depth of neutral axis — $x = 2.5 \times (d - z)$ = **31** mm

Area of tension reinforcement required — $A_{sr.req} = M / (f_{yd} \times z)$ = **286** mm^2/m

Tension reinforcement provided — 12 dia.bars @ 150 c/c

Area of tension reinforcement provided — $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr})$ = **754** mm^2/m

Minimum area of reinforcement - exp.9.1N — $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **384** mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{sr.max} = 0.04 \times h$ = **12000** mm^2/m

$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov}$ = **0.509**

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

Library item: Rectangular single output

Deflection control - Section 7.4

Reference reinforcement ratio — $\rho_0 = \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} / 1000$ = **0.006**

Required tension reinforcement ratio — $\rho = A_{sr.req} / d$ = **0.001**

Required compression reinforcement ratio — $\rho' = A_{sr.2.req} / d_2$ = **0.000**

Structural system factor - Table 7.4N — K_b = **1**

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

Limiting span to depth ratio - exp.7.16.a — $\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1\text{ N/mm}^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1\text{ N/mm}^2)} \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b) =$ **40**

Actual span to depth ratio — $h_{prop} / d =$ **16.1**

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width — $w_{max} =$ **0.2** mm

Variable load factor - EN1990 – Table A1.1 — $\psi_2 =$ **0.3**

Serviceability bending moment — $M_{sls} =$ **21.4** kNm/m

Tensile stress in reinforcement — $\sigma_s = M_{sls} / (A_{sr.prov} \times z) =$ **122.3** N/mm^2

Load duration — Long term

Load duration factor — $k_t =$ **0.4**

Effective area of concrete in tension — $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$

$A_{c.eff} =$ **89833** mm^2/m

Mean value of concrete tensile strength — $f_{ct.eff} = f_{ctm} =$ **3.0** N/mm^2

Reinforcement ratio — $\rho_{p.eff} = A_{sr.prov} / A_{c.eff} =$ **0.008**

Modular ratio — $\alpha_e = E_s / E_{cm} =$ **5.998**

Bond property coefficient — $k_1 =$ **0.8**

Strain distribution coefficient — $k_2 =$ **0.5**

$k_3 =$ **3.4**

$k_4 =$ **0.425**

Maximum crack spacing - exp.7.11 — $s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff} =$ **413** mm

Maximum crack width - exp.7.8 — $w_k = s_{r.max} \times \max(\sigma_s – k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

$w_k =$ **0.152** mm

$w_k / w_{max} =$ **0.758**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force — $V =$ **49.4** kN/m

$C_{Rd,c} = 0.18 / \gamma_C =$ **0.120**

$k = \min(1 + \sqrt{(200\text{ mm} / d)}, 2) =$ **1.905**

Longitudinal reinforcement ratio — $\rho_l = \min(A_{sr.prov} / d, 0.02) =$ **0.003**

$v_{min} = 0.035\text{ N}^{1/2}\text{/mm} \times k^{3/2} \times f_{ck}^{0.5} =$ **0.521** N/mm^2

Design shear resistance - exp.6.2a & 6.2b — $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\text{ N}^2\text{/mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c} =$ **127.1** kN/m

$V / V_{Rd.c} =$ **0.389**

PASS - Design shear resistance exceeds design shear force

Check stem design at prop

Depth of section — $h =$ **300** mm

Rectangular section in shear - Section 6.2

Design shear force — $V =$ **6.9** kN/m

$C_{Rd,c} = 0.18 / \gamma_C =$ **0.120**

$k = \min(1 + \sqrt{(200\text{ mm} / d)}, 2) =$ **1.905**

Longitudinal reinforcement ratio — $\rho_l = \min(A_{sr1.prov} / d, 0.02) =$ **0.002**

$v_{min} = 0.035\text{ N}^{1/2}\text{/mm} \times k^{3/2} \times f_{ck}^{0.5} =$ **0.521** N/mm^2

Design shear resistance - exp.6.2a & 6.2b — $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\text{ N}^2\text{/mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c} =$ **127.1** kN/m

$V / V_{Rd.c} =$ **0.055**

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Minimum area of reinforcement – cl.9.6.3(1) $A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem})$ = **300** mm²/m

Maximum spacing of reinforcement – cl.9.6.3(2) s_{sx_max} = **400** mm

Transverse reinforcement provided 12 dia.bars @ 175 c/c

Area of transverse reinforcement provided $A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx})$ = **646** mm²/m

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

Check base design at toe

Depth of section h = **300** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 M = **45.7** kNm/m

Depth to tension reinforcement $d = h - c_{bb} - \phi_{bb} / 2$ = **217** mm

$K = M / (d^2 \times f_{ck})$ = **0.030**

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

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = **206** mm

Depth of neutral axis $x = 2.5 \times (d - z)$ = **27** mm

Area of tension reinforcement required $A_{bb.req} = M / (f_{yd} \times z)$ = **510** mm²/m

Tension reinforcement provided 16 dia.bars @ 150 c/c

Area of tension reinforcement provided $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **1340** mm²/m

Minimum area of reinforcement - exp.9.1N $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **341** mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{bb.max} = 0.04 \times h$ = **12000** mm²/m

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov}$ = **0.38**

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

Library item: Rectangular single output

Crack control - Section 7.3

Limiting crack width w_{max} = **0.2** mm

Variable load factor - EN1990 – Table A1.1 ψ_2 = **0.3**

Serviceability bending moment M_{sls} = **33.8** kNm/m

Tensile stress in reinforcement $\sigma_s = M_{sls} / (A_{bb.prov} \times z)$ = **122.5** N/mm²

Load duration Long term

Load duration factor k_t = **0.4**

Effective area of concrete in tension $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$

$A_{c.eff}$ = **90958** mm²/m

Mean value of concrete tensile strength $f_{ct.eff} = f_{ctm}$ = **3.0** N/mm²

Reinforcement ratio $\rho_{p.eff} = A_{bb.prov} / A_{c.eff}$ = **0.015**

Modular ratio $\alpha_e = E_s / E_{cm}$ = **5.998**

Bond property coefficient k_1 = **0.8**

Strain distribution coefficient k_2 = **0.5**

k_3 = **3.4**

k_4 = **0.425**

Maximum crack spacing - exp.7.11 $s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff}$ = **440** mm

Maximum crack width - exp.7.8 $w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

w_k = **0.161** mm

w_k / w_{max} = **0.807**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force V = **36.5** kN/m

$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.960**

Longitudinal reinforcement ratio $\rho_l = \min(A_{bb.prov} / d, 0.02)$ = **0.006**

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.543** N/mm^2

Design shear resistance - exp.6.2a & 6.2b $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c}$ = **138** kN/m

$V / V_{Rd.c}$ = **0.265**

PASS - Design shear resistance exceeds design shear force

Check base design at heel

Depth of section h = **300** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 M = **0.1** kNm/m

Depth to tension reinforcement $d = h - c_{bt} - \phi_{bt} / 2$ = **242** mm

$K = M / (d^2 \times f_{ck})$ = **0.000**

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

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = **230** mm

Depth of neutral axis $x = 2.5 \times (d - z)$ = **30** mm

Area of tension reinforcement required $A_{bt.req} = M / (f_{yd} \times z)$ = **1** mm^2/m

Tension reinforcement provided 16 dia.bars @ 150 c/c

Area of tension reinforcement provided $A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt})$ = **1340** mm^2/m

Minimum area of reinforcement - exp.9.1N $A_{bt.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **381** mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{bt.max} = 0.04 \times h$ = **12000** mm^2/m

$\max(A_{bt.req}, A_{bt.min}) / A_{bt.prov}$ = **0.284**

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

Library item: Rectangular single output

Crack control - Section 7.3

Limiting crack width w_{max} = **0.2** mm

Variable load factor - EN1990 – Table A1.1 ψ_2 = **0.3**

Serviceability bending moment M_{sls} = **0** kNm/m

Tensile stress in reinforcement $\sigma_s = M_{sls} / (A_{bt.prov} \times z)$ = **0.1** N/mm^2

Load duration Long term

Load duration factor k_t = **0.4**

Effective area of concrete in tension $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$

$A_{c.eff}$ = **89917** mm^2/m

Mean value of concrete tensile strength $f_{ct.eff} = f_{ctm}$ = **3.0** N/mm^2

Reinforcement ratio $\rho_{p.eff} = A_{bt.prov} / A_{c.eff}$ = **0.015**

Modular ratio $\alpha_e = E_s / E_{cm}$ = **5.998**

Bond property coefficient k_1 = **0.8**

Strain distribution coefficient k_2 = **0.5**

k_3 = **3.4**

k_4 = **0.425**

Maximum crack spacing - exp.7.11 $s_{r.max} = k_3 \times c_{bt} + k_1 \times k_2 \times k_4 \times \phi_{bt} / \rho_{p.eff} = \mathbf{352}$ mm

Maximum crack width - exp.7.8 $w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

$w_k = \mathbf{0}$ mm

$w_k / w_{max} = \mathbf{0.001}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force $V = \mathbf{1.2}$ kN/m

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

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = \mathbf{1.909}$

Longitudinal reinforcement ratio $\rho_l = \min(A_{bt.prov} / d, 0.02) = \mathbf{0.006}$

$v_{min} = 0.035 \text{ N}^{1/2}\text{/mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.522}$ N/mm^2

Design shear resistance - exp.6.2a & 6.2b $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2\text{/mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c} = \mathbf{144.5}$ kN/m

$V / V_{Rd.c} = \mathbf{0.008}$

PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

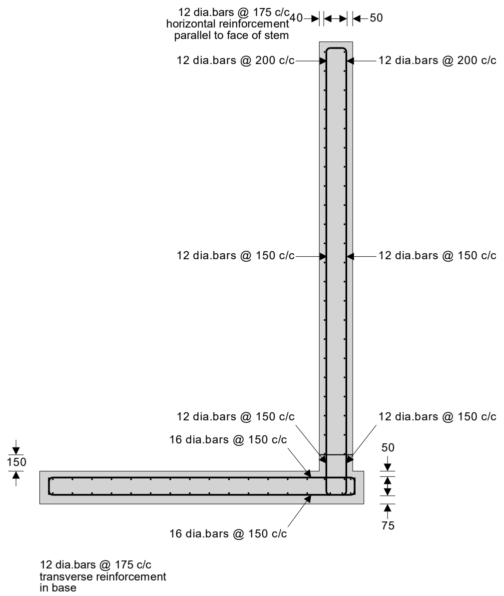
Minimum area of reinforcement – cl.9.3.1.1(2) $A_{bx.req} = 0.2 \times A_{bb.prov} = \mathbf{268}$ mm^2/m

Maximum spacing of reinforcement – cl.9.3.1.1(3) $s_{bx\_max} = \mathbf{450}$ mm

Transverse reinforcement provided 12 dia.bars @ 175 c/c

Area of transverse reinforcement provided $A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx}) = \mathbf{646}$ mm^2/m

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



Reinforcement details

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Calculation for:		Job No.
25-26 Redington Gardens		180709
		Date: Dec 18
Calculation by: RSC	Checked by:	Sheet No. T1

Temporary works

The Construction of the Basement Shall be Carried out in a top Down Sequence. Refer to 180709-PL-01-03 for Construction Sequence

IN General, The Capping Beam Shall be propped in the temporary and permanent Condition by the Lower Ground floor Slab which Shall Be Cast Before Excavation for the Basement Commences.

The Central Spine wall Shall be Supported by a Series of plunge Columns and transfer Beams The Central Spine Beam Shall be designed to Support the Lower Ground floor and upper Sup. Structure until Such times as the permanent Structure is in place.

The Gable wall on Grid line 7 Shall be Supported off a piled wall, however Should it Be decided to Construct No's 25&26 with No 24, the piled Wall Shall be Removed and Replaced with a transfer Beam.

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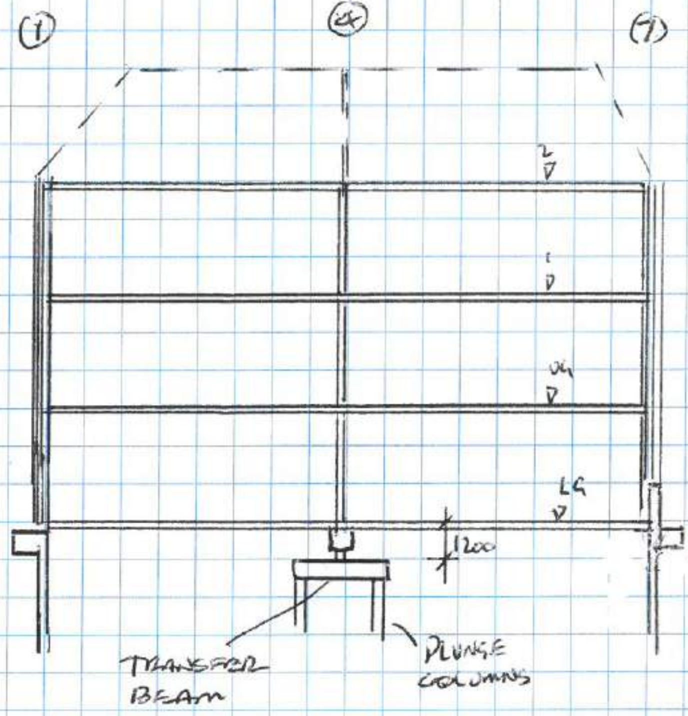
Calculation for: 25-26 Redington Gardens Temporary Works		Job No. 180709
		Date: Dec 18
Calculation by:	Checked by:	Sheet No. T2



GL

Working load - 416kN

∴ for 850 kN pile, Spacing = $850/_{416 \div 2}$ = 4.03m.

Say max 4.5m CTRS.

Transfer Beam.

Worst case : Moment = 416÷1.45 + 4.25^2 x 0.11 = 1193kN

Moment from Analysis : M_{max} = 1198 kNm

V_{max} = 1326 kN

For x RC Downstand Beam.

Provide 6T32 BTM, 2T12 TOP min, 6legs T10 links@150%

Section design to Eurocode 2 (BS EN 1992-1)

RECTANGULAR BEAMS

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T3

INPUT

Location Spine beam — Beam type END SPAN

M	kNm	**1198.0**	f_{ck}	40	N/mm²	γ_c =	1.50
δ		1.00	f_{yk}	500	N/mm²	γ_s =	1.15
span	mm	4250	steel class	A		Δc_{dev}	10
h	mm	750					
b	mm	450					
g_k	kN/m	1.00					
q_k	kN/m	1.00					

REBAR	Ø	COVER
Tension	32	42
Comp'n	12	35
Side	--	42

to main bars

Ψ_2 = 0.3 Dwelling brittle partitions? YES

OUTPUT

Spine beam

Effective depth, d = 750 - 42 - 32/2 = 692.0 mm

Neutral axis, x = [692-√(692² -2E6x1198x1.5/0.85/450/40)]/0.8 = 247.6 mm

(x/d) limit = 0.600 x/d actual = 0.358 ok

Lever arm, z = 692 - 0.4 x 247.6 = 593.0 mm

Tension steel, As = 1198.0E6/593.0/434.8 = 4647 mm²

9.2.1.1 (1) As min = 0.00182 x 450 x 750 = 568 mm²

7.3.2 (2) As crack = 0.4 x 0.8 x 3 x 750/2 x 450/500 = 369 mm²

for deflection, As def = 1359 mm²

PROVIDE 6 H32 TENSION BARS = 4825 mm²

.

Service stress, SLS M =1198 x 2 /2.9 = 840.7 kNm ρ = 299 N/mm²

Modification factor = 310 /298.7 = 1.038

Permissible L/d = MIN(40 x 1.3, 1.038 x 1.000 x 19.527) = 20.27

Actual L/d = 4250 /692 = 6.14 ok

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Section design to Eurocode 2 (BS EN 1992-1)

BEAM SHEAR

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T4

INPUT Location Spine Beam

f_{ck} N/mm² 40 γ_c = 1.50

f_{ywk} N/mm² 500 γ_s = 1.15

d	bw
750	450

Main Steel

Ø 32

No 6

Link Ø	Legs	Side cover	VEd	n
10	4	30	1325.0	0
mm	No	mm	kN at face	kN/m

OUTPUT **Spine Beam**

6.2.2 (1) A_{sL} = 4825 mm² = 1.430% f_{cd} = 26.7 N/mm²

equation (6.6) ν = 0.6(1 - 40/250) = 0.504 . cotθ = 2.50

equation (6.9) $V_{Rd,max}$ = 1 x 450 x 675.0 x 0.504 x 26.7 /2.90 /1000 = 1,407. ok

6.2.1 (8) V_{Ed} @ d = 1325 - 0 x 0.75 = 1,325.0 kN

6.2.2 (1) k = 1 + √(200 /750) = 1.516 .

equation (6.2) $V_{Rd,c}$ = 0.12 x 1.516 cube root(1.430 x 40) = 236.6 kN

9.2.2 (5) A_{sw}/s (min)= 0.08 x 450 /500 x √40 = 0.455 mm

equation (6.9) A_{sw}/s (max)= 0.5 x 450 /500 x 1.15 x 0.504 x 26.7 = 6.955 mm ok

equation (6.7) A_{sw}/s = 1,325.0E3 /(675.0 x 434.8 x 2.50) = 1.806 > 0.455

9.2.2 (6) $s_{max,L}$ = 563 mm $s_{max,T}$ = 563 mm *9.2.2 (8)* ok

PROVIDE 4 legs H10 @ 150

Throughout

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Calculation for:		Job No.
25-26 Redington Gardens Temporary Works.		180709
		Date: Dec 13
Calculation by: KSC	Checked by:	Sheet No. TS

Support Beams.

Reach = 416 × 1.45 × 4.25 = 2536 kN

Moment = $2536 \times \frac{1.35}{4}$ = 856 kNm

Provide 533×210×109 UB

Column (plunge) Reach

F = 2536/2 = 1268 kN

L_e = 4700 mm

Try 152×152×44 UC

$N_{b,y,Rd}$ = 1332 kN

$N_{b,z,Rd}$ = 1280 kN with Restraints at 2.5m c/c

Provide 152×152×44 UC