Notes:
-To be confirmed by Building Control before work commencing
-All dimensions to be confirmed on site
-Supports based on assumption of solid masonry walls
-Adequacy of existing lintels to be confirmed on site by Building control officer
-Adequacy of existing foundation to be confirmed on site by Building control officer.
-Floor and roof joists to be strap tied to walls via steel straps at 600cc
-All floor and roof joists to have noggings at 600cc for lateral support.
-Joists under stud walls, around stairs and around Velux to be doubled.
-Drawing to be read in conjunction with Architects drawings.

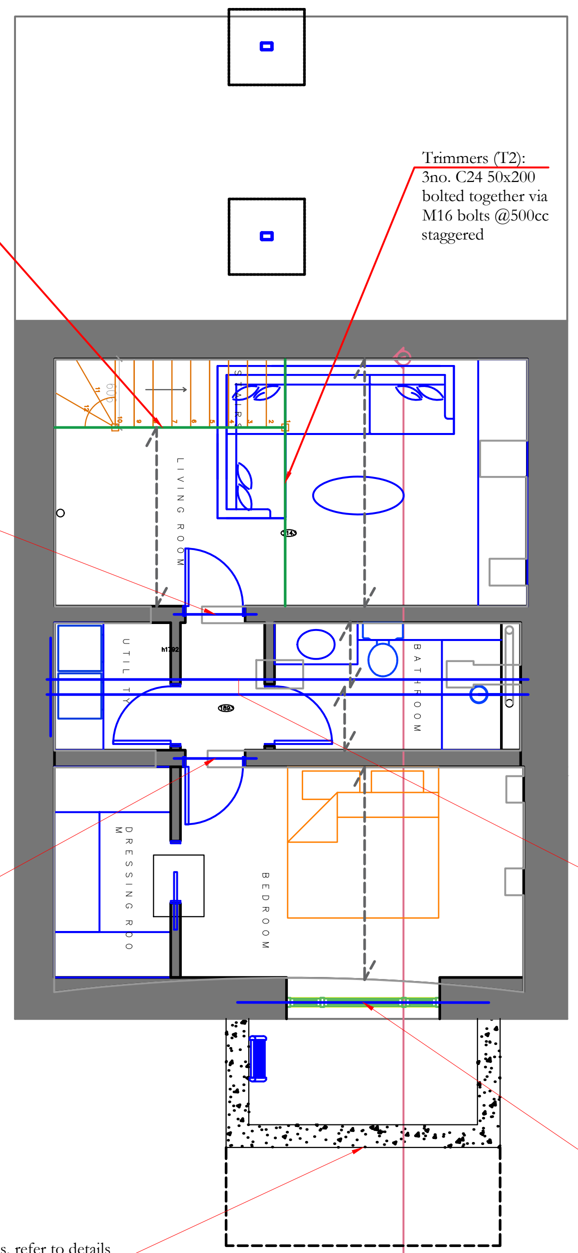
Trimmers (T1):
3no. C24 50x200
bolted together via
M16 bolts @500cc
staggered

Trimmers (T2):
3no. C24 50x200
bolted together via
M16 bolts @500cc
staggered

2no. Naylor R6 lintels
100x140mm,
with minimum bearing
of 150mm either side

2no. Naylor R6 lintels
100x140mm,
with minimum bearing
of 150mm either side

RC retaining walls, refer to details



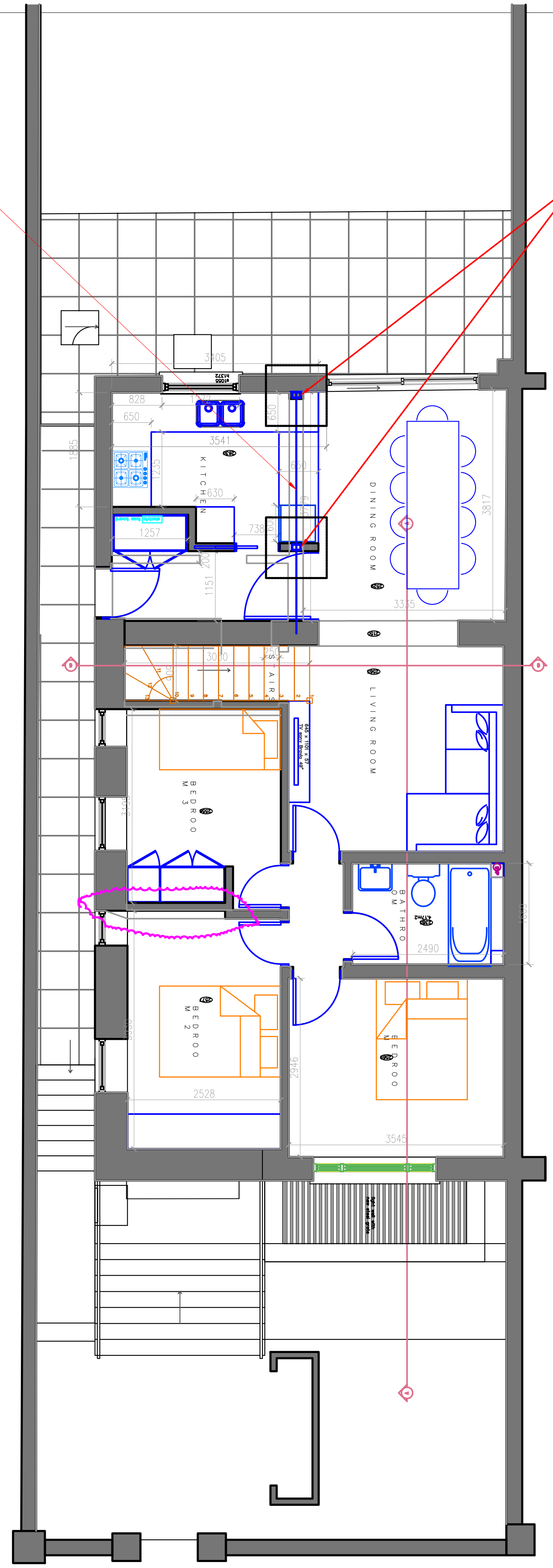
Proposed cellar

Beam (B1):
UC 203x203x46 steel grade S275
with 12mm mild steel plate welded
to top flange to suit wall above
Supports:
a) concrete pad stone
25x15x10cm
b) columns C1, refer to
connection details

Columns (C1):
RHS 150x100x8 steel grade S355
to be anchored to walls via
M12 chemical anchor bolts
@450cc staggered
gap between columns and brick
walls to be dry packed
Supports:
concrete foundations
1000x1000mm x1100mm deep
(depth to be confirmed by
Building Control Officer)

Beam (B2):
2no. UC 203x203x71 steel grade S355
to be bolted together side by side
via M16 bolts grade 8.8 @500cc,
beams to have 12x300mm mild steel
plates welded to bottom flange
Supports:
a) UC 203x203x46 S355, with
minimum bearing length of 125cm
b) mild steel plate 850x100x40mm

Beam (B3):
UC 203x203x71 galvanized steel grade S355
with 12mm mild steel plate welded
to bottom flange to suit wall above
Supports:
minimum bearing of
65cm either side



Proposed ground floor

0 1 2

A&S Design
8, 90 Akerman Road London. SW9 6SN
info@as-d.co.uk | m:07588693045

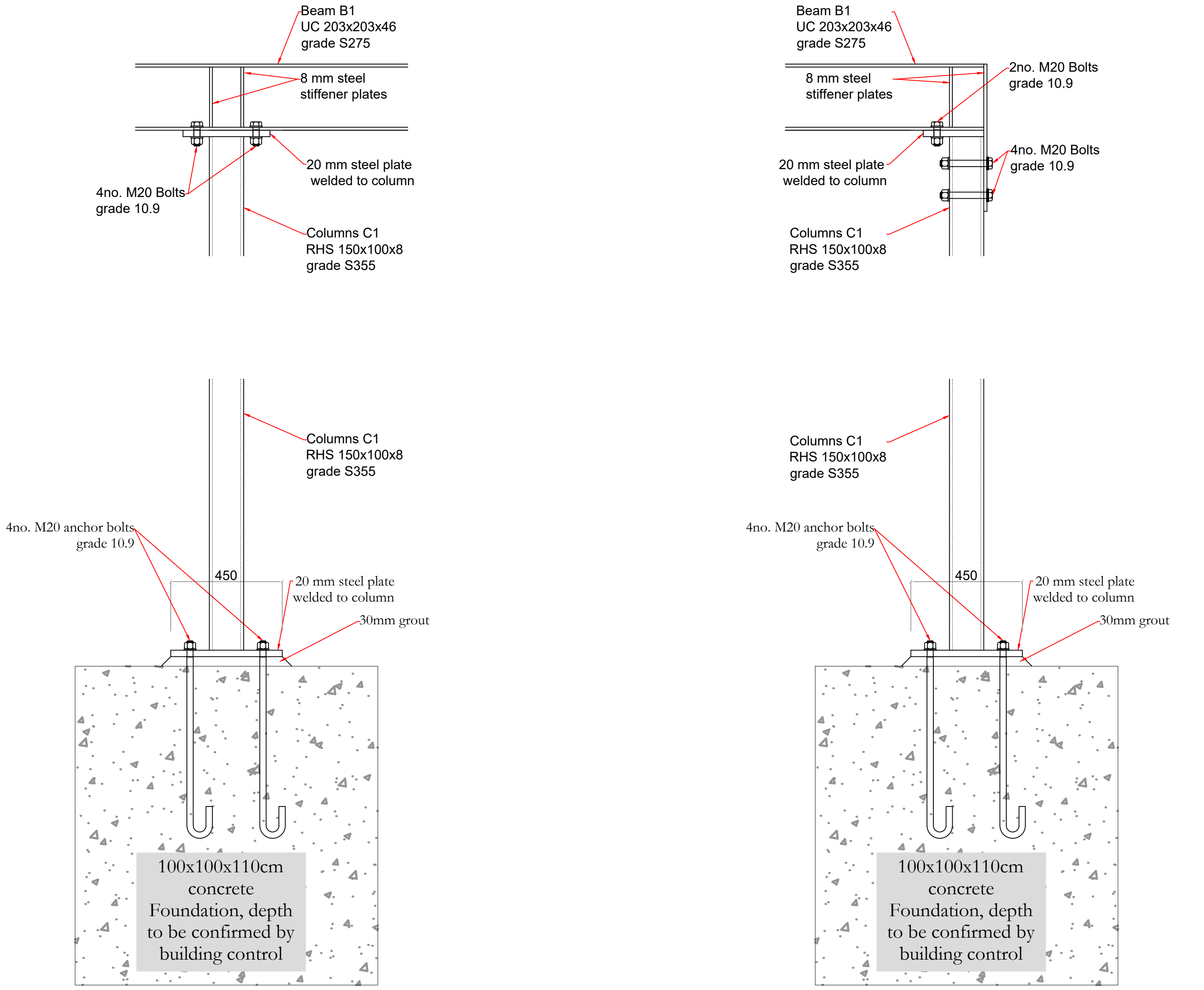
Project:
38 Dartmouth Park Road
NW5

Title:
Structural plans

REV: (A)

Scale:
1:100@a3p

DN: 371-DPR-01



connection of beam B1 to columns C1
scale 1:20

A&S Design
8, 90 Akerman Road London. SW9 6SN
info@as-d.co.uk | m:07588693045

Project:
38 Dartmouth Park Road
NW5

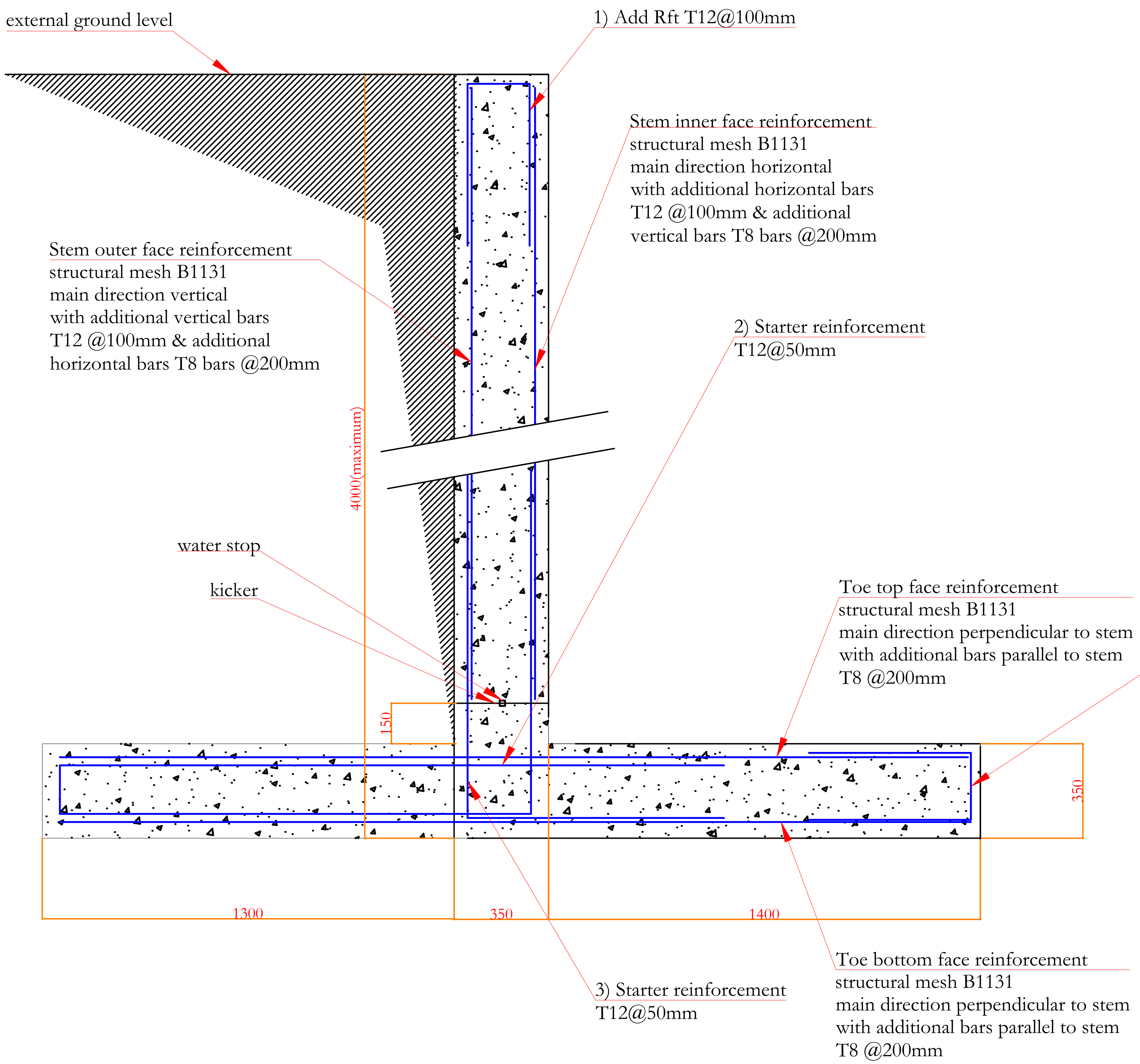
Title:
Details

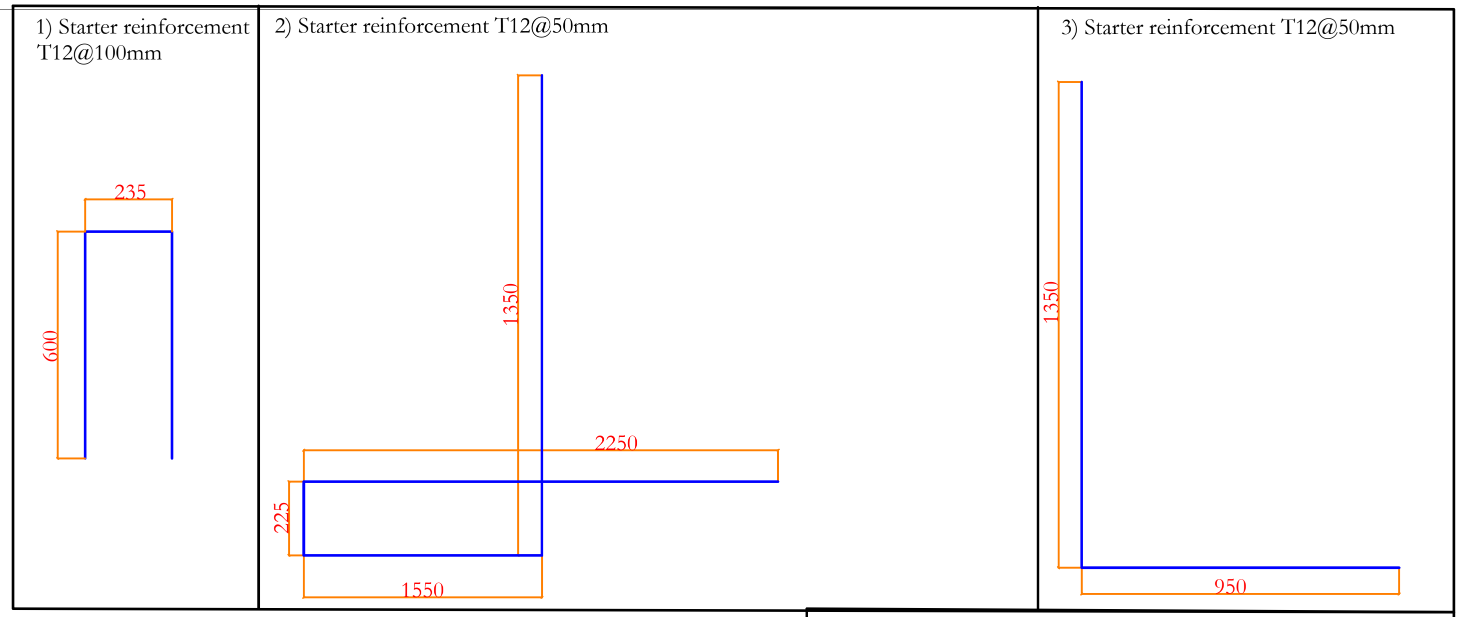
REV: (A)

Scale:
1:20@a3p

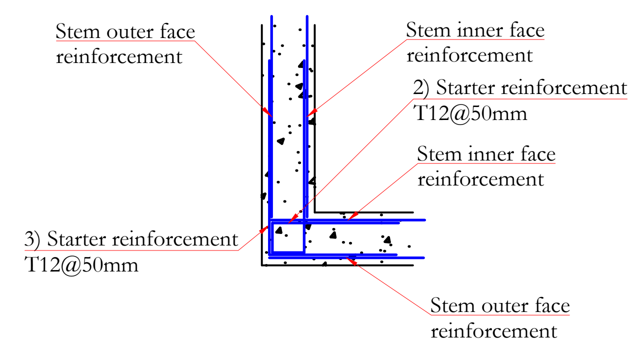
DN: 371-DPR-02

RC retaining wall details
(front light well)





Typical corner detail
scale 1:50



Notes:
-All structural drawings to be read in conjunction with Architects drawings and specification.
-Concrete grade C35 (35 N/mm2).
-Minimum cover to be 50mm.
-Reinforcement steel grade T (460 N/mm2)
-Minimum laps to reinforcement to be:
T12=600mm
T8=500mm
-Minimum anchorage to be
T12=840mm
T8=560mm
-All joints horizontal or vertical to have water stops as per manufacturers guidance.
-All dimensions to be confirmed on site by contractor prior to work commencing.

Notes:
-To be confirmed by Building Control before work commencing
-All dimensions to be confirmed on site
-Supports based on assumption of solid masonry walls
-Adequacy of existing lintels to be confirmed on site by Building control officer
-Adequacy of existing foundation to be confirmed on site by Building control officer.
-Floor and roof joists to be strap tied to walls via steel straps at 600cc
-All floor and roof joists to have noggings at 600cc for lateral support.
-Joists under stud walls, around stairs and around Velux to be doubled.
-Drawing to be read in conjunction with Architects drawings.

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Project:
38 Dartmouth Park Road
NW5

Title:
Details

REV: (A)

Scale:
1:20@a3p

DN: 371-DPR-03

project: 38 Dartmouth Park Road

Trimmers (T1): max span= 305 cm assume 3no. C24 50x200

i- LOADS

a) Uniform loads

1- ground floor (dead+live)		2	kn/m2
	x	1.63	m
	=	**3.25**	**kn/m**
2- stud wall		0.35	kn/m2
	x	2.60	m
	=	**0.91**	**kn/m**

TOTAL U.L= 4.16 Kn/m

ii- FORCES

Ra= 6.52 Kn **Rb= 6.52 Kn**

MAX BENDING MOMENT (M)= 4.97 kn.m

MAX SHEAR FORCE (Q)= 6.52 kn

MAX DEFLECTION(d)= 6.27 mm

iii- CHECK

refer to attached sheet for detailed design check according to BS 5268-2

use 3no. C24 50x200

project: 38 Dartmouth Park Road

Trimmers (T2): max span= 325 cm assume 3no. C24 50x200

i- LOADS

a) Uniform loads

1- ground floor (dead+live)		2	kn/m2
	x	0.40	m
	=	**0.80**	**kn/m**

TOTAL U.L= 0.80 Kn/m

b) reaction of T1 **6.52 Kn @ 2.25m**

ii- FORCES

Ra= 3.50 Kn **Rb= 6.00 Kn**

MAX BENDING MOMENT (M)= 5.55 kn.m

MAX SHEAR FORCE (Q)= 6.00 kn

MAX DEFLECTION(d)= 6.69 mm

iii- CHECK

refer to attached sheet for detailed design check according to BS 5268-2

use 3no. C24 50x200

project: 38 Dartmouth Park Road

Beam (B1):

span= 400 cm   assume UC 203x203x46 grade S275

i- LOADS

a) Uniform Loads

1- own weight		**0.46**	**kn/m**
2- 1st floor (dead+live)		2	kn/m2
	x	3.25	m
	=	**6.50**	**kn/m**
3- wall above		4.5	kn/m2
	x	3.70	m
	=	**16.65**	**kn/m**
4- roof (dead+live)		1.85	kn/m2
	x	3.25	m
	=	**6.01**	**kn/m**

TOTAL U.L= **29.62** **Kn/m**   x1.5 ultimate load factor

ii- forces

R_1=	9.40	kn	R_2=	77.60	kn
M_{ult} =	26.30	kn.m	R3=	30.00	kn
Q_{max}=	66.00	kn			
deflection d_{act}=	0.90	mm			

iii- check

assuming unrestrained compression flange

Refer to attached table for detailed desing check according to BS 5950 2000

Design check ok

use UC 203x203x46 grade S275

iv- supports

a) assume support width = 10 cm

f_{all}= 0.55 N/mm2

L_{req}= 17.09 cm

use concrete pad stone 25x15x10cm

b) columns C1, refer to connection details

project: 38 Dartmouth Park Road

Columns (C1):

height= 300 cm   assume RHS 150x100x8 grade S355

i- LOADS

a) LOADS:

1- Own weight		0.28	kn/m
	x	3.00	m
	=	0.84	kn
2-load from (B1)		77.60	kn
TOTAL=		78.440	Kn
factor	x	1.5	
Total Ultimate load "F"	**=**	**117.66**	**Kn**
b) ultimate bending moment "Mx"	**=**	**5.82**	**Kn.m** (due to eccentricity)

ii- stresses

Pa= 32.02 N/mm2

iii- check

assuming pinned at both ends

Le=	300.00	cm		
λ=	75.38			
From Blue Book				
Pz=	1310.00	Kn		
F/Pz=	0.09			
Mry=	47.20	Kn.m	>Mx	
				OK
Pcy=	916.00	Kn	>F	
Mb=	61.80	Kn.m	>M	
				OK
Ratio (F/Pz+M/Mb)=	0.22		<1	
				OK

use RHS 150x100x8 grade S355

iv- supports

new concrete foundations 1000x1000mm x 1100mm deep

project: 38 Dartmouth Park Road

Beam (B2):

span= 635 cm — assume 2no. UC 203x203x71 grade S355

i- LOADS

a) Uniform Loads

1- own weight		**1.42**	**kn/m**	
2- ground (dead+live)		2	kn/m2	
	x	0.85	m	
	=	**1.70**	**kn/m**	
3- wall above		2.5	kn/m2	(between 0.0 & 2.70m)
	x	6.70	m	
	=	**16.75**	**kn/m**	
4- upper floors (dead+live)		2	kn/m2	(between 0.0 & 2.70m)
	x	4.15	m	
	x	4.00	floors	
	=	**33.20**	**kn/m**	
5- stud wall		0.35	kn/m2	(between 0.0 & 2.70m)
	x	6.15	m	
	=	**2.15**	**kn/m**	
6- roof (dead+live)		1.85	kn/m2	(between 0.0 & 2.70m)
	x	4.15	m	
	=	**7.68**	**kn/m**	

TOTAL U.L=	**62.90**	**Kn/m**	x1.5 ultimate load factor
Load per beam	**31.45**	**Kn/m**	**x1.5 ultimate load factor**

ii- forces

R_1=	**68.50 kn**		**R_2=**	**22.10 kn**
M_{ult} =	**111.90 kn.m**			
Q_{max}=	**102.70 kn**			
deflection d_{act}=	**17.70 mm**			

iii- check

assuming unrestrained compression flange
Refer to attached table for detailed desing check according to BS 5950 2000
Design check ok

use 2no. UC 203x203x71 grade S355

iv- supports

a) assume support width = 20 cm

f_{all}= 0.55 N/mm2

L_{req}= 124.55 cm

use UC 203x203x46 S355 with minimum bearing length of 125cm

b) assume support width = 10 cm

f_{all}= 0.55 N/mm2

L_{req}= 80.36 cm

use mild steel plate 850x100x40mm

project: 38 Dartmouth Park Road
Beam (B3):

span= 255 cm — assume UC 203x203x71 grade S355

i- LOADS

a) Uniform Loads

1- own weight		**0.71**	**kn/m**
2- ground (dead+live)		2	kn/m2
	x	2.00	m
	=	**4.00**	**kn/m**
3- wall above		4.5	kn/m2
	x	13.10	m
	=	**58.95**	**kn/m**
4- upper floors (dead+live)		2	kn/m2
	x	2.00	m
	x	4.00	floors
	=	**16.00**	**kn/m**
5- roof (dead+live)		1.85	kn/m2
	x	2.00	m
	=	**3.70**	**kn/m**

TOTAL U.L= **83.36** **Kn/m** x1.5 ultimate load factor

ii- forces

R_1=	**106.30 kn**	**R_2=**	**106.30 kn**
M_{ult} =	**101.60 kn.m**		
Q_{max}=	**159.40 kn**		
deflection d_{act}=	**2.90 mm**		

iii- check

assuming unrestrained compression flange
Refer to attached table for detailed desing check according to BS 5950 2000
Design check ok

use 2no. UC 203x203x71 grade S355

iv- supports

assume support width = 30 cm

f_{all}= 0.55 N/mm2

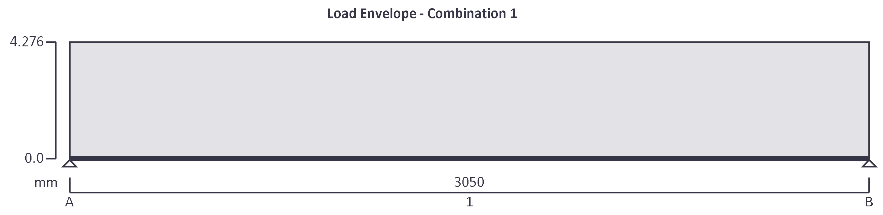
L_{req}= 64.42 cm

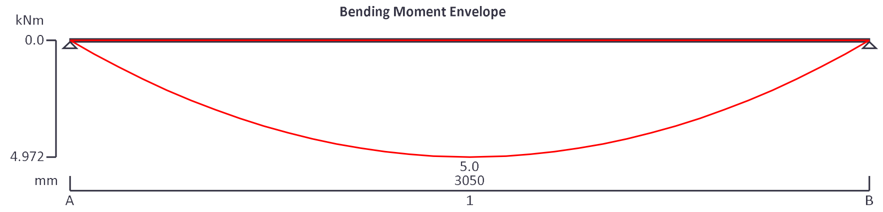
use minimum bearing of 65cm either side

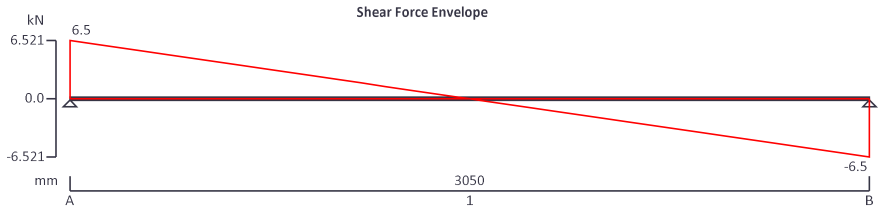
TRIMMER (T1):

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.01







Applied loading

Beam loads

Dead full UDL 4.160 kN/m

Dead self weight of beam × 1

Load combinations

Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Maximum moment;	M_{max} = **4.972** kNm;	M_{min} = **0.000** kNm
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **4.972** kNm	
Maximum shear;	F_{max} = **6.521** kN;	F_{min} = **-6.521** kN

Design shear; $F = max(abs(F_{max}),abs(F_{min}))$ = **6.521** kN

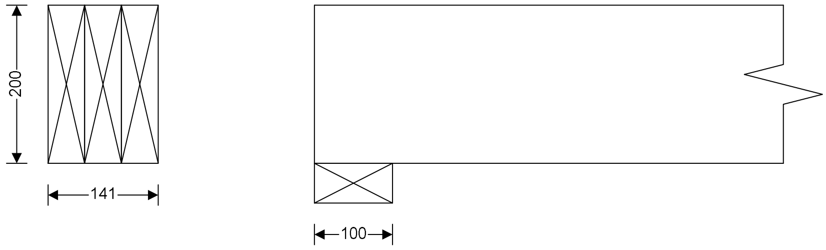
Total load on beam; W_{tot} = **13.042** kN

Reactions at support A; R_{A_max} = **6.521** kN; R_{A_min} = **6.521** kN

Unfactored dead load reaction at support A; R_{A_Dead} = **6.521** kN

Reactions at support B; R_{B_max} = **6.521** kN; R_{B_min} = **6.521** kN

Unfactored dead load reaction at support B; R_{B_Dead} = **6.521** kN



Timber section details

Breadth of sections; b = **47** mm

Depth of sections; h = **200** mm

Number of sections in member; N = **3**

Overall breadth of member; $b_b = N \times b$ = **141** mm

Timber strength class; **C24**

Member details

Service class of timber; **1**

Load duration; **Long term**

Length of span; L_{s1} = **3050** mm

Length of bearing; L_b = **100** mm

Section properties

Cross sectional area of member; $A = N \times b \times h$ = **28200** mm^2

Section modulus; $Z_x = N \times b \times h^2 / 6$ = **940000** mm^3

$Z_y = h \times (N \times b)^2 / 6$ = **662700** mm^3

Second moment of area; $I_x = N \times b \times h^3 / 12$ = **94000000** mm^4

$I_y = h \times (N \times b)^3 / 12$ = **46720350** mm^4

Radius of gyration; $i_x = \sqrt{(I_x / A)}$ = **57.7** mm

$i_y = \sqrt{(I_y / A)}$ = **40.7** mm

Modification factors

Duration of loading - Table 17; K_3 = **1.00**

Bearing stress - Table 18; K_4 = **1.00**

Total depth of member - cl.2.10.6; $K_7 = (300\ mm / h)^{0.11}$ = **1.05**

Load sharing - cl.2.10.11; K_8 = **1.10**

Minimum modulus of elasticity - Table 20; K_9 = **1.21**

Lateral support - cl.2.10.8

Ends held in position

Permissible depth-to-breadth ratio - Table 19; **3.00**

Actual depth-to-breadth ratio; $h / (N \times b)$ = **1.42**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane); $\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8 =$ **2.640** N/mm^2

Applied bearing stress; $\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b) =$ **0.462** N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm} =$ **0.175**

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 =$ **8.626** N/mm^2

Applied bending stress; $\sigma_{m\_a} = M / Z_x =$ **5.290** N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm} =$ **0.613**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress; $\tau_{adm} = \tau \times K_3 \times K_8 =$ **0.781** N/mm^2

Applied shear stress; $\tau_a = 3 \times F / (2 \times A) =$ **0.347** N/mm^2

$\tau_a / \tau_{adm} =$ **0.444**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection; $E = E_{min} \times K_9 =$ **8712** N/mm^2

Permissible deflection; $\delta_{adm} = \min(0.551 \text{ in}, 0.003 \times L_{s1}) =$ **9.150** mm

Bending deflection; $\delta_{b\_s1} =$ **5.884** mm

Shear deflection; $\delta_{v\_s1} =$ **0.389** mm

Total deflection; $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} =$ **6.272** mm

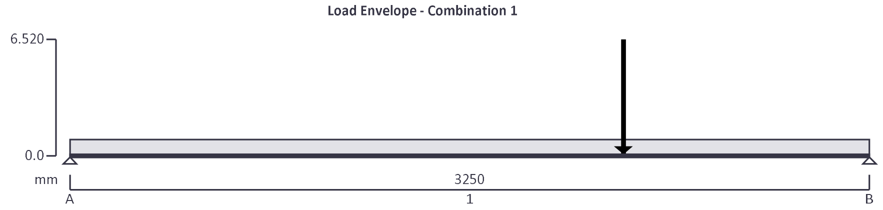
$\delta_a / \delta_{adm} =$ **0.685**

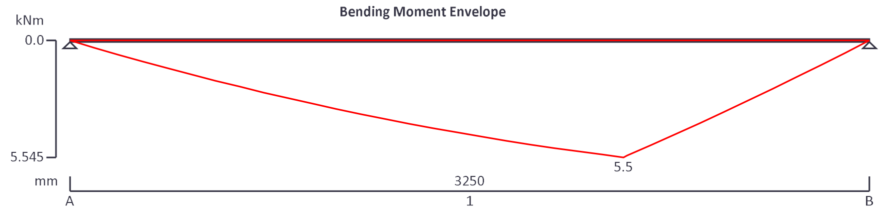
PASS - Total deflection is less than permissible deflection

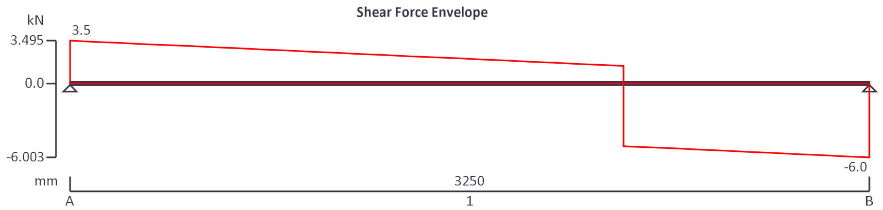
TRIMMER (T2):

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.01







Applied loading

Beam loads

Dead full UDL 0.800 kN/m

Dead self weight of beam × 1

Dead point load 6.520 kN at 2250 mm

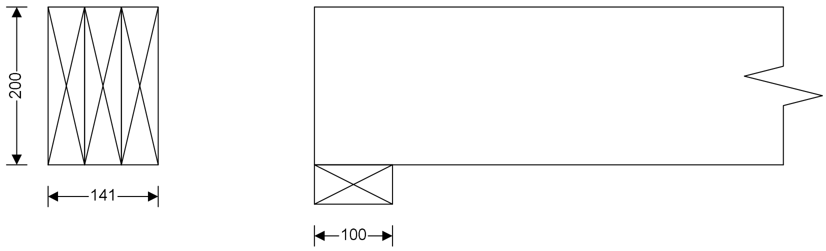
Load combinations

Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Maximum moment;	M_{max} = **5.545** kNm;	M_{min} = **0.000** kNm
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **5.545** kNm	

Maximum shear;	F_{max} = **3.495** kN;	F_{min} = **-6.003** kN
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **6.003** kN	
Total load on beam;	W_{tot} = **9.497** kN	
Reactions at support A;	R_{A_max} = **3.495** kN;	R_{A_min} = **3.495** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **3.495** kN	
Reactions at support B;	R_{B_max} = **6.003** kN;	R_{B_min} = **6.003** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **6.003** kN	



Timber section details

Breadth of sections;	b = **47** mm
Depth of sections;	h = **200** mm
Number of sections in member;	N = **3**
Overall breadth of member;	$b_b = N \times b$ = **141** mm
Timber strength class;	**C24**

Member details

Service class of timber;	**1**
Load duration;	**Long term**
Length of span;	L_{s1} = **3250** mm
Length of bearing;	L_b = **100** mm

Section properties

Cross sectional area of member;	$A = N \times b \times h$ = **28200** mm^2
Section modulus;	$Z_x = N \times b \times h^2 / 6$ = **940000** mm^3
	$Z_y = h \times (N \times b)^2 / 6$ = **662700** mm^3
Second moment of area;	$I_x = N \times b \times h^3 / 12$ = **94000000** mm^4
	$I_y = h \times (N \times b)^3 / 12$ = **46720350** mm^4
Radius of gyration;	$i_x = \sqrt{(I_x / A)}$ = **57.7** mm
	$i_y = \sqrt{(I_y / A)}$ = **40.7** mm

Modification factors

Duration of loading - Table 17;	K_3 = **1.00**
Bearing stress - Table 18;	K_4 = **1.00**
Total depth of member - cl.2.10.6;	$K_7 = (300\text{ mm} / h)^{0.11}$ = **1.05**
Load sharing - cl.2.10.11;	K_8 = **1.10**
Minimum modulus of elasticity - Table 20;	K_9 = **1.21**

Lateral support - cl.2.10.8

Ends held in position	
Permissible depth-to-breadth ratio - Table 19;	**3.00**
Actual depth-to-breadth ratio;	$h / (N \times b)$ = **1.42**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane); $\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8$ = **2.640** N/mm^2

Applied bearing stress; $\sigma_{c\_a} = R_{B\_max} / (N \times b \times L_b)$ = **0.426** N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.161**

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **8.626** N/mm^2

Applied bending stress; $\sigma_{m\_a} = M / Z_x$ = **5.898** N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm}$ = **0.684**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress; $\tau_{adm} = \tau \times K_3 \times K_8$ = **0.781** N/mm^2

Applied shear stress; $\tau_a = 3 \times F / (2 \times A)$ = **0.319** N/mm^2

τ_a / τ_{adm} = **0.409**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection; $E = E_{min} \times K_9$ = **8712** N/mm^2

Permissible deflection; δ_{adm} = min(0.551 in, 0.003 × L_{s1}) = **9.750** mm

Bending deflection; δ_{b_s1} = **6.258** mm

Shear deflection; δ_{v_s1} = **0.433** mm

Total deflection; $\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **6.691** mm

δ_a / δ_{adm} = **0.686**

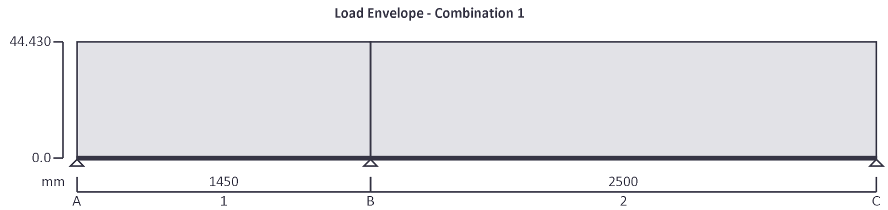
PASS - Total deflection is less than permissible deflection

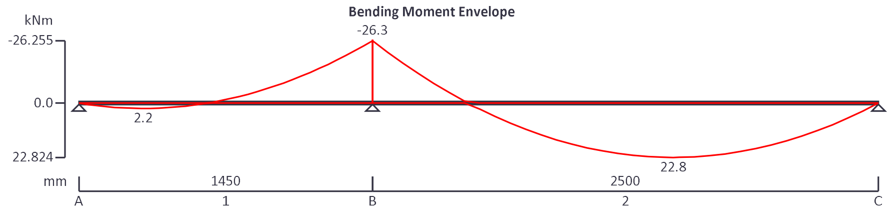
BEAM (B1):

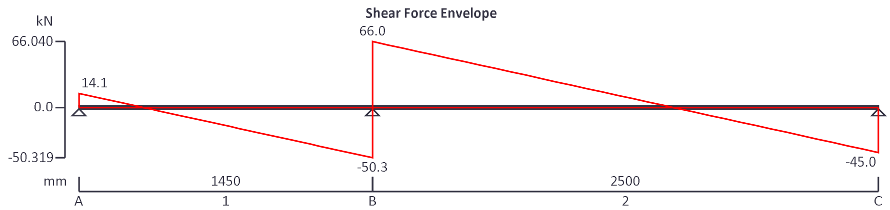
STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07







Support conditions

Support A — Vertically restrained
Rotationally free

Support B — Vertically restrained
Rotationally free

Support C — Vertically restrained
Rotationally free

Applied loading

Beam loads — Dead full UDL 29.62 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.50
		Imposed × 1.50
		Dead × 1.50
		Imposed × 1.50

	Support B	Dead × 1.50
		Imposed × 1.50
		Dead × 1.50
		Imposed × 1.50
	Support C	Dead × 1.50
		Imposed × 1.50

Analysis results

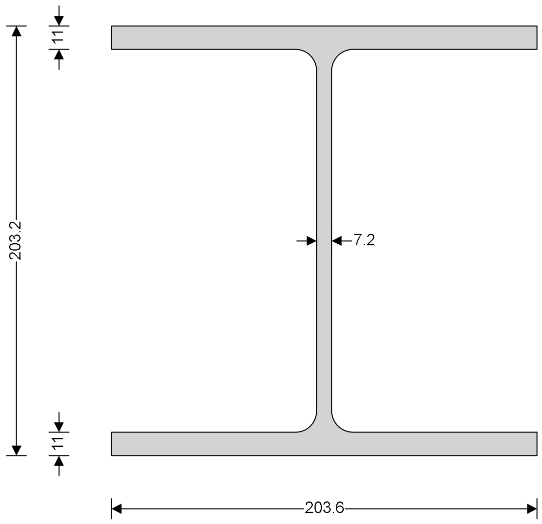
Maximum moment;	M_{max} = **22.8** kNm;	M_{min} = **-26.3** kNm
Maximum moment span 1;	M_{s1_max} = **2.2** kNm;	M_{s1_min} = **-26.3** kNm
Maximum moment span 2;	M_{s2_max} = **22.8** kNm;	M_{s2_min} = **-26.3** kNm
Maximum shear;	V_{max} = **66** kN;	V_{min} = **-50.3** kN
Maximum shear span 1;	V_{s1_max} = **14.1** kN;	V_{s1_min} = **-50.3** kN
Maximum shear span 2;	V_{s2_max} = **66** kN;	V_{s2_min} = **-45** kN
Deflection;	δ_{max} = **0.9** mm;	δ_{min} = **0.1** mm
Deflection span 1;	δ_{s1_max} = **0** mm;	δ_{s1_min} = **0.1** mm
Deflection span 2;	δ_{s2_max} = **0.9** mm;	δ_{s2_min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **14.1** kN;	R_{A_min} = **14.1** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **9.4** kN	
Maximum reaction at support B;	R_{B_max} = **116.4** kN;	R_{B_min} = **116.4** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **77.6** kN	
Maximum reaction at support C;	R_{C_max} = **45** kN;	R_{C_min} = **45** kN
Unfactored dead load reaction at support C;	R_{C_Dead} = **30** kN	

Section details

Section type;	**UKC 203x203x46 (Tata Steel Advance)**
Steel grade;	**S275**

From table 9: Design strength p_y

Thickness of element;	max(T, t) = **11.0** mm
Design strength;	p_y = **275** N/mm^2
Modulus of elasticity;	E = **205000** N/mm^2



Lateral restraint

Span 1 has lateral restraint at supports only

Span 2 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis; K_x = **1.00**

Effective length factor in minor axis; K_y = **1.00**

Effective length factor for lateral-torsional buckling; $K_{LT.A}$ = **1.20**; + 2 × D

$K_{LT.B}$ = **1.20**; + 2 × D

$K_{LT.C}$ = **1.00**;

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **1.00**

Internal compression parts - Table 11

Depth of section; d = **160.8** mm

$d / t = 22.3 \times \varepsilon <= 80 \times \varepsilon$; Class 1 plastic

Outstand flanges - Table 11

Width of section; b = B / 2 = **101.8** mm

$b / T = 9.3 \times \varepsilon <= 10 \times \varepsilon$; Class 2 compact

Section is class 2 compact

Shear capacity - Section 4.2.3

Design shear force; $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **66** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area; $A_v = t \times D$ = **1463** mm²

Design shear resistance; $P_v = 0.6 \times p_y \times A_v$ = **241.4** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity at span 2 - Section 4.2.5

Design bending moment; $M = \max(\text{abs}(M_{s2\_max}), \text{abs}(M_{s2\_min}))$ = **26.3** kNm

Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y \times S_{xx}, 1.5 \times p_y \times Z_{xx})$ = **136.8** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = ((1.2 + 1.0) \times L_{s2} + 2 \times D) / 2$ = **2953** mm

Slenderness ratio; $\lambda = L_E / r_{yy}$ = **57.519**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; u = **0.847**

Torsional index; x = **17.713**

Slenderness factor; $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25}$ = **0.900**

Ratio - cl.4.3.6.9; β_W = **1.000**

Equivalent slenderness - cl.4.3.6.7; $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]}$ = **43.799**

Limiting slenderness - Annex B.2.2; $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5}$ = **34.310**

$\lambda_{LT} > \lambda_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant; α_{LT} = **7.0**

Perry factor; $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0)$ = **0.066**

Euler stress; $p_E = \pi^2 \times E / \lambda_{LT}^2$ = **1054.7** N/mm²

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = \mathbf{699.9}$ N/mm^2

Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = \mathbf{252.9}$ N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; $M_2 = \mathbf{6.3}$ kNm

Moment at centre-line of segment; $M_3 = \mathbf{21.6}$ kNm

Moment at three quarter point of segment; $M_4 = \mathbf{19.5}$ kNm

Maximum moment in segment; $M_{abs} = \mathbf{26.3}$ kNm

Maximum moment governing buckling resistance; $M_{LT} = M_{abs} = \mathbf{26.3}$ kNm

Equivalent uniform moment factor for lateral-torsional buckling;

$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = \mathbf{0.758}$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{xx} = \mathbf{125.8}$ kNm

$M_b / m_{LT} = \mathbf{165.9}$ kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection; $\delta_{lim} = L_{s2} / 250 = \mathbf{10}$ mm

Maximum deflection span 2; $\delta = \max(abs(\delta_{max}), abs(\delta_{min})) = \mathbf{0.891}$ mm

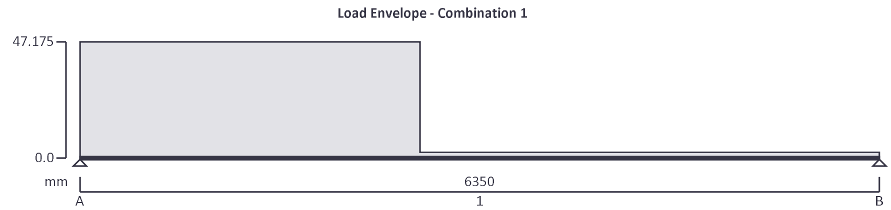
PASS - Maximum deflection does not exceed deflection limit

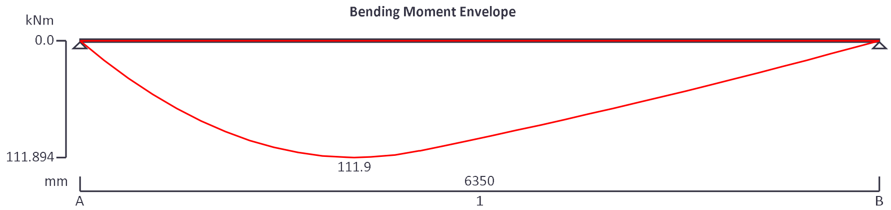
BEAM (B2):

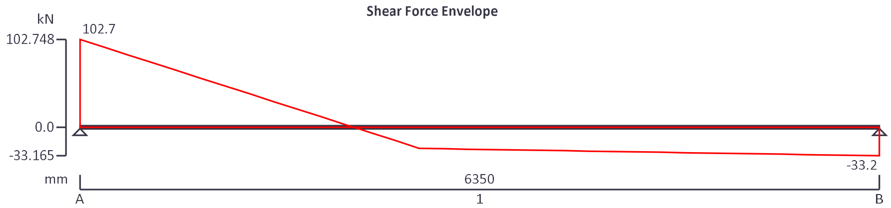
STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07







Support conditions

Support A	Vertically restrained	
	Rotationally free	
Support B	Vertically restrained	
	Rotationally free	

Applied loading

Beam loads	Dead full UDL 1.56 kN/m	
	Dead partial UDL 29.89 kN/m from 0 mm to 2700 mm	

Load combinations

Load combination 1	Support A	Dead × 1.50
		Imposed × 1.50
		Dead × 1.50
		Imposed × 1.50
	Support B	Dead × 1.50

Imposed $\times$ 1.50

Analysis results

Maximum moment;	M_{max} = **111.9** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **102.7** kN;	V_{min} = **-33.2** kN
Deflection;	δ_{max} = **17.7** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **102.7** kN;	R_{A_min} = **102.7** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **68.5** kN	
Maximum reaction at support B;	R_{B_max} = **33.2** kN;	R_{B_min} = **33.2** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **22.1** kN	

Section details

Section type; **UKC 203x203x71 (Tata Steel Advance)**

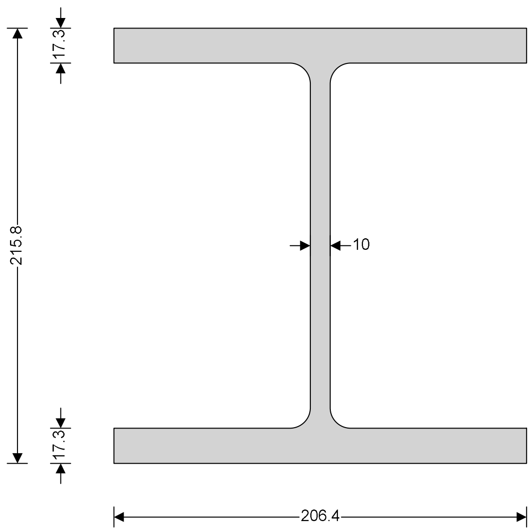
Steel grade; **S355**

From table 9: Design strength p_y

Thickness of element; max(T, t) = **17.3** mm

Design strength; p_y = **345** N/mm^2

Modulus of elasticity; E = **205000** N/mm^2



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis; K_x = **1.00**

Effective length factor in minor axis; K_y = **1.00**

Effective length factor for lateral-torsional buckling; $K_{LT.A}$ = **1.20**; + 2 $\times$ D

$K_{LT.B}$ = **1.20**; + 2 $\times$ D

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **0.89**

Internal compression parts - Table 11

Depth of section; d = **160.8** mm

$d / t = 18.0 \times \varepsilon <= 80 \times \varepsilon$; Class 1 plastic

Outstand flanges - Table 11

Width of section; $b = B / 2 =$ **103.2** mm

$b / T = 6.7 \times \varepsilon <= 9 \times \varepsilon$; Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force; $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) =$ **102.7** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area; $A_v = t \times D =$ **2158** mm^2

Design shear resistance; $P_v = 0.6 \times p_y \times A_v =$ **446.7** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) =$ **111.9** kNm

Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) =$ **275.6** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = 1.2 \times L_{s1} + 2 \times D =$ **8052** mm

Slenderness ratio; $\lambda = L_E / r_{yy} =$ **152.001**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; $u =$ **0.853**

Torsional index; $x =$ **11.926**

Slenderness factor; $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} =$ **0.575**

Ratio - cl.4.3.6.9; $\beta_W =$ **1.000**

Equivalent slenderness - cl.4.3.6.7; $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]} =$ **74.567**

Limiting slenderness - Annex B.2.2; $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} =$ **30.632**

$\lambda_{LT} > \lambda_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant; $\alpha_{LT} =$ **7.0**

Perry factor; $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) =$ **0.308**

Euler stress; $p_E = \pi^2 \times E / \lambda_{LT}^2 =$ **363.9** N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 =$ **410.4** N/mm^2

Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) =$ **203.3** N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; $M_2 =$ **103.7** kNm

Moment at centre-line of segment; $M_3 =$ **93.5** kNm

Moment at three quarter point of segment; $M_4 =$ **49.7** kNm

Maximum moment in segment; $M_{abs} =$ **111.9** kNm

Maximum moment governing buckling resistance; $M_{LT} = M_{abs} =$ **111.9** kNm

Equivalent uniform moment factor for lateral-torsional buckling;

$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) =$ **0.823**

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{xx} =$ **162.4** kNm

$M_b / m_{LT} =$ **197.2** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection; $\delta_{lim} = L_{s1} / 250 =$ **25.4** mm

Maximum deflection span 1; $\delta = max(abs(\delta_{max}), abs(\delta_{min})) =$ **17.747** mm

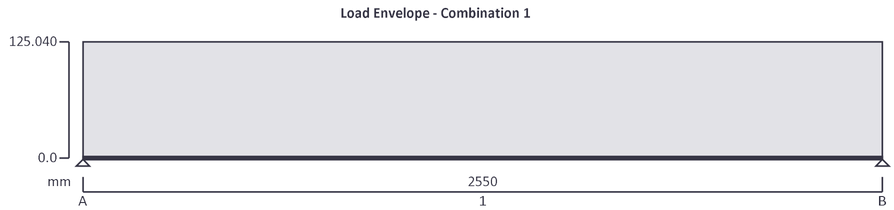
PASS - Maximum deflection does not exceed deflection limit

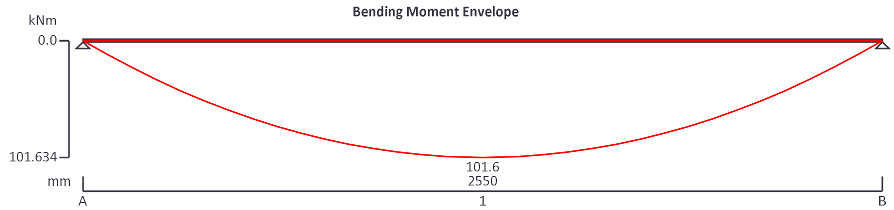
BEAM (B3):

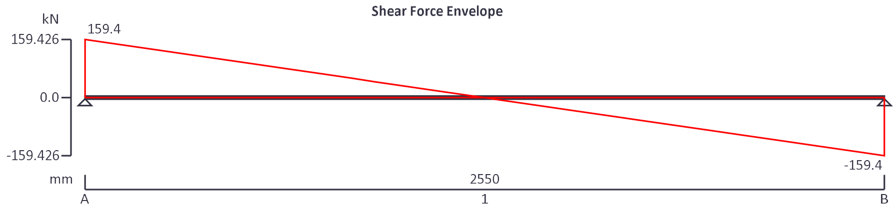
STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Dead full UDL 83.36 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.50
		Imposed × 1.50
		Dead × 1.50
		Imposed × 1.50
	Support B	Dead × 1.50
		Imposed × 1.50

Analysis results

Maximum moment;	M_{max} = **101.6** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **159.4** kN;	V_{min} = **-159.4** kN
Deflection;	δ_{max} = **2.9** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **159.4** kN;	R_{A_min} = **159.4** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **106.3** kN	
Maximum reaction at support B;	R_{B_max} = **159.4** kN;	R_{B_min} = **159.4** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **106.3** kN	

Section details

Section type; **UKC 203x203x71 (Tata Steel Advance)**

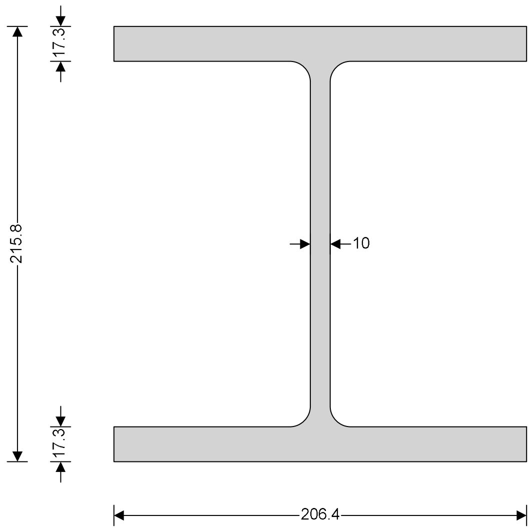
Steel grade; **S355**

From table 9: Design strength p_y

Thickness of element; max(T, t) = **17.3** mm

Design strength; p_y = **345** N/mm^2

Modulus of elasticity; E = **205000** N/mm^2



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis; K_x = **1.00**

Effective length factor in minor axis; K_y = **1.00**

Effective length factor for lateral-torsional buckling; $K_{LT.A}$ = **1.20**; + 2 × D

$K_{LT.B}$ = **1.20**; + 2 × D

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **0.89**

Internal compression parts - Table 11

Depth of section; d = **160.8** mm

d / t = 18.0 × ε <= 80 × ε; Class 1 plastic

Outstand flanges - Table 11

Width of section; b = B / 2 = **103.2** mm

$b / T = 6.7 \times \varepsilon <= 9 \times \varepsilon$; Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force; $F_v = \max(\mathrm{abs}(V_{max}), \mathrm{abs}(V_{min})) =$ **159.4** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area; $A_v = t \times D =$ **2158** mm^2

Design shear resistance; $P_v = 0.6 \times p_y \times A_v =$ **446.7** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = \max(\mathrm{abs}(M_{s1\_max}), \mathrm{abs}(M_{s1\_min})) =$ **101.6** kNm

Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) =$ **275.6** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = 1.2 \times L_{s1} + 2 \times D =$ **3492** mm

Slenderness ratio; $\lambda = L_E / r_{yy} =$ **65.916**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; $u =$ **0.853**

Torsional index; $x =$ **11.926**

Slenderness factor; $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} =$ **0.793**

Ratio - cl.4.3.6.9; $\beta_W =$ **1.000**

Equivalent slenderness - cl.4.3.6.7; $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]} =$ **44.570**

Limiting slenderness - Annex B.2.2; $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} =$ **30.632**

$\lambda_{LT} > \lambda_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant; $\alpha_{LT} =$ **7.0**

Perry factor; $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) =$ **0.098**

Euler stress; $p_E = \pi^2 \times E / \lambda_{LT}^2 =$ **1018.5** N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 =$ **731.5** N/mm^2

Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) =$ **302.9** N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; $M_2 =$ **76.2** kNm

Moment at centre-line of segment; $M_3 =$ **101.6** kNm

Moment at three quarter point of segment; $M_4 =$ **76.2** kNm

Maximum moment in segment; $M_{abs} =$ **101.6** kNm

Maximum moment governing buckling resistance; $M_{LT} = M_{abs} =$ **101.6** kNm

Equivalent uniform moment factor for lateral-torsional buckling;

$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) =$ **0.925**

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{xx} =$ **242** kNm

$M_b / m_{LT} =$ **261.6** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection; $\delta_{lim} = L_{s1} / 250 =$ **10.2** mm

Maximum deflection span 1; $\delta = max(abs(\delta_{max}), abs(\delta_{min})) =$ **2.939** mm

PASS - Maximum deflection does not exceed deflection limit

RETAINING WALL ANALYSIS LOAD CASE 1 (NO GROUND WATER PRESENT):

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.9.07

Retaining wall details

Stem type;	Cantilever
Stem height;	h_{stem} = **3650** mm
Stem thickness;	t_{stem} = **350** mm
Angle to rear face of stem;	α = **90** deg
Stem density;	γ_{stem} = **25** kN/m^3
Toe length;	l_{toe} = **1400** mm
Heel length;	l_{heel} = **1300** mm
Base thickness;	t_{base} = **350** mm
Base density;	γ_{base} = **25** kN/m^3
Height of retained soil;	h_{ret} = **3400** mm
Angle of soil surface;	β = **0** deg
Depth of cover;	d_{cover} = **250** mm
Depth of excavation;	d_{exc} = **250** mm

Retained soil properties

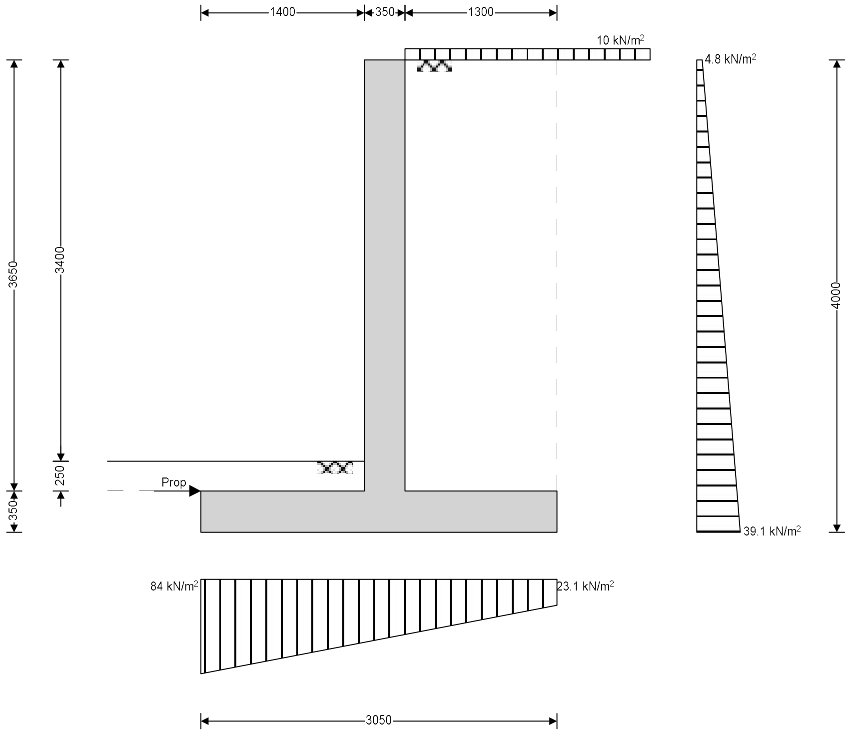
Soil type;	Firm clay
Moist density;	γ_{mr} = **18** kN/m^3
Saturated density;	γ_{sr} = **18** kN/m^3
Characteristic effective shear resistance angle;	$\phi'_{r.k}$ = **18** deg
Characteristic wall friction angle;	$\delta_{r.k}$ = **9** deg

Base soil properties

Soil type;	Firm clay
Soil density;	γ_b = **18** kN/m^3
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
Presumed bearing capacity;	$P_{bearing}$ = **100** kN/m^2

Loading details

Variable surcharge load;	$Surcharge_Q$ = **10** kN/m^2



General arrangement

Calculate retaining wall geometry

Base length;	$l_{base} = l_{toe} + t_{stem} + l_{heel}$ = **3050** mm
Moist soil height;	$h_{moist} = h_{soil}$ = **3650** mm
Length of surcharge load;	$l_{sur} = l_{heel}$ = **1300** mm
- Distance to vertical component;	$x_{sur\_v} = l_{base} - l_{heel} / 2$ = **2400** mm
Effective height of wall;	$h_{eff} = h_{base} + d_{cover} + h_{ret}$ = **4000** mm
- Distance to horizontal component;	$x_{sur\_h} = h_{eff} / 2$ = **2000** mm
Area of wall stem;	$A_{stem} = h_{stem} \times t_{stem}$ = **1.278** m^2
- Distance to vertical component;	$x_{stem} = l_{toe} + t_{stem} / 2$ = **1575** mm
Area of wall base;	$A_{base} = l_{base} \times t_{base}$ = **1.068** m^2
- Distance to vertical component;	$x_{base} = l_{base} / 2$ = **1525** mm
Area of moist soil;	$A_{moist} = h_{moist} \times l_{heel}$ = **4.745** m^2
- Distance to vertical component;	$x_{moist\_v} = l_{base} - (h_{moist} \times l_{heel}^2 / 2) / A_{moist}$ = **2400** mm
- Distance to horizontal component;	$x_{moist\_h} = h_{eff} / 3$ = **1333** mm
Area of base soil;	$A_{pass} = d_{cover} \times l_{toe}$ = **0.35** m^2
- Distance to vertical component;	$x_{pass\_v} = l_{base} - (d_{cover} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{pass}$ = **700** mm
- Distance to horizontal component;	$x_{pass\_h} = (d_{cover} + h_{base}) / 3$ = **200** mm

Using Coulomb theory

Active pressure coefficient; $K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]}]^2)$ = **0.483**

Passive pressure coefficient; $K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2)$ = **2.359**

Bearing pressure check

Vertical forces on wall

Wall stem; $F_{stem} = A_{stem} \times \gamma_{stem}$ = **31.9** kN/m

Wall base; $F_{base} = A_{base} \times \gamma_{base}$ = **26.7** kN/m

Surcharge load; $F_{sur\_v} = Surcharge_Q \times l_{heel}$ = **13** kN/m

Moist retained soil; $F_{moist\_v} = A_{moist} \times \gamma_{mr}$ = **85.4** kN/m

Base soil; $F_{pass\_v} = A_{pass} \times \gamma_b$ = **6.3** kN/m

Total; $F_{total\_v} = F_{stem} + F_{base} + F_{sur\_v} + F_{moist\_v} + F_{pass\_v}$ = **163.3** kN/m

Horizontal forces on wall

Surcharge load; $F_{sur\_h} = K_A \times \cos(\delta_{r.k}) \times Surcharge_Q \times h_{eff}$ = **19.1** kN/m

Moist retained soil; $F_{moist\_h} = K_A \times \cos(\delta_{r.k}) \times \gamma_{mr} \times h_{eff}^2 / 2$ = **68.7** kN/m

Base soil; $F_{pass\_h} = -K_P \times \cos(\delta_{b.k}) \times \gamma_b \times (d_{cover} + h_{base})^2 / 2$ = **-7.5** kN/m

Total; $F_{total\_h} = F_{sur\_h} + F_{moist\_h} + F_{pass\_h}$ = **80.2** kN/m

Moments on wall

Wall stem; $M_{stem} = F_{stem} \times x_{stem}$ = **50.3** kNm/m

Wall base; $M_{base} = F_{base} \times x_{base}$ = **40.7** kNm/m

Surcharge load; $M_{sur} = F_{sur\_v} \times x_{sur\_v} - F_{sur\_h} \times x_{sur\_h}$ = **-7** kNm/m

Moist retained soil; $M_{moist} = F_{moist\_v} \times x_{moist\_v} - F_{moist\_h} \times x_{moist\_h}$ = **113.4** kNm/m

Base soil; $M_{pass} = F_{pass\_v} \times x_{pass\_v}$ = **4.4** kNm/m

Total; $M_{total} = M_{stem} + M_{base} + M_{sur} + M_{moist} + M_{pass}$ = **201.8** kNm/m

Check bearing pressure

Propping force; $F_{prop\_base} = F_{total\_h}$ = **80.2** kN/m

Distance to reaction; $\bar{x} = M_{total} / F_{total\_v}$ = **1236** mm

Eccentricity of reaction; $e = \bar{x} - l_{base} / 2$ = **-289** mm

Loaded length of base; $l_{load} = l_{base}$ = **3050** mm

Bearing pressure at toe; $q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base})$ = **84** kN/m^2

Bearing pressure at heel; $q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base})$ = **23.1** kN/m^2

Factor of safety; $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel})$ = **1.19**

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 2.9.07

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

Concrete strength class; C35/45

Characteristic compressive cylinder strength; f_{ck} = **35** N/mm^2

Characteristic compressive cube strength; $f_{ck,cube}$ = **45** N/mm^2

Mean value of compressive cylinder strength; $f_{cm} = f_{ck} + 8$ N/mm^2 = **43** N/mm^2

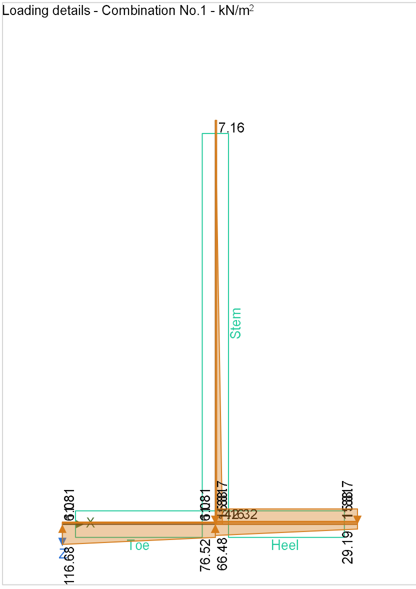
Mean value of axial tensile strength;	$f_{ctm} = 0.3\ N/mm^2 \times (f_{ck} / 1\ N/mm^2)^{2/3} = \mathbf{3.2}\ N/mm^2$
5% fractile of axial tensile strength;	$f_{ctk,0.05} = 0.7 \times f_{ctm} = \mathbf{2.2}\ N/mm^2$
Secant modulus of elasticity of concrete;	$E_{cm} = 22\ kN/mm^2 \times (f_{cm} / 10\ N/mm^2)^{0.3} = \mathbf{34077}\ N/mm^2$
Partial factor for concrete - Table 2.1N;	$\gamma_C = \mathbf{1.50}$
Compressive strength coefficient - cl.3.1.6(1);	$\alpha_{cc} = \mathbf{0.85}$
Design compressive concrete strength - exp.3.15;	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = \mathbf{19.8}\ N/mm^2$
Maximum aggregate size;	$h_{agg} = \mathbf{20}$ mm
Ultimate strain - Table 3.1;	$\varepsilon_{cu2} = \mathbf{0.0035}$
Shortening strain - Table 3.1;	$\varepsilon_{cu3} = \mathbf{0.0035}$
Effective compression zone height factor;	$\lambda = \mathbf{0.80}$
Effective strength factor;	$\eta = \mathbf{1.00}$
Bending coefficient k_1;	$K_1 = \mathbf{0.40}$
Bending coefficient k_2;	$K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2}) = \mathbf{1.00}$
Bending coefficient k_3;	$K_3 = \mathbf{0.40}$
Bending coefficient k_4;	$K_4 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2}) = \mathbf{1.00}$

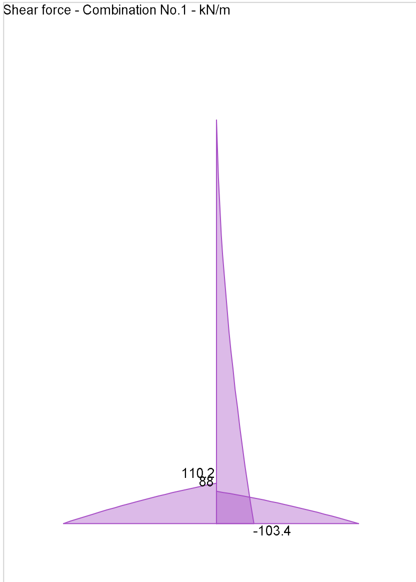
Reinforcement details

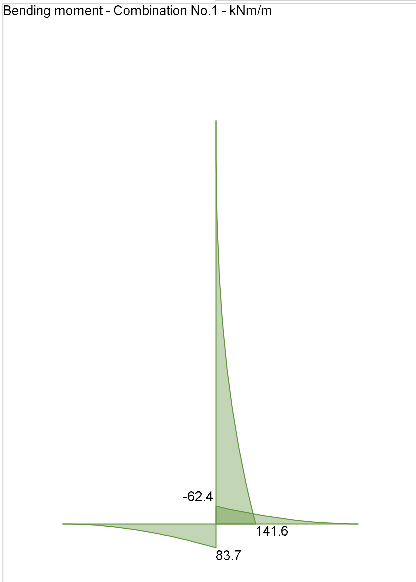
Characteristic yield strength of reinforcement;	$f_{yk} = \mathbf{500}\ N/mm^2$
Modulus of elasticity of reinforcement;	$E_s = \mathbf{200000}\ N/mm^2$
Partial factor for reinforcing steel - Table 2.1N;	$\gamma_S = \mathbf{1.15}$
Design yield strength of reinforcement;	$f_{yd} = f_{yk} / \gamma_S = \mathbf{435}\ N/mm^2$

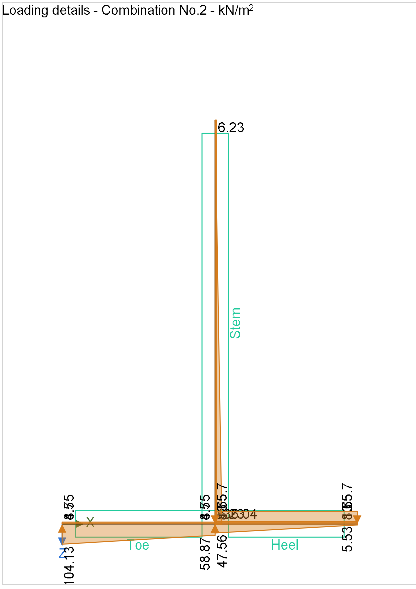
Cover to reinforcement

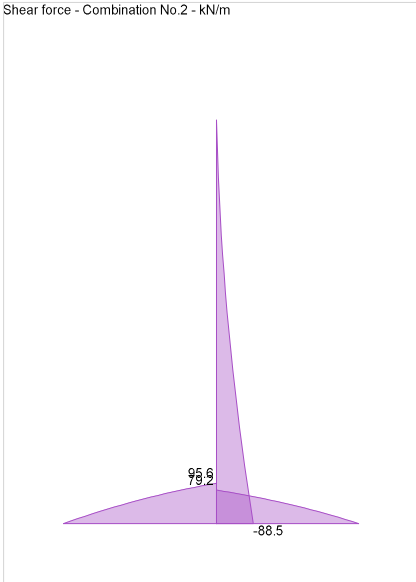
Front face of stem;	$c_{sf} = \mathbf{50}$ mm
Rear face of stem;	$c_{sr} = \mathbf{50}$ mm
Top face of base;	$c_{bt} = \mathbf{50}$ mm
Bottom face of base;	$c_{bb} = \mathbf{50}$ mm

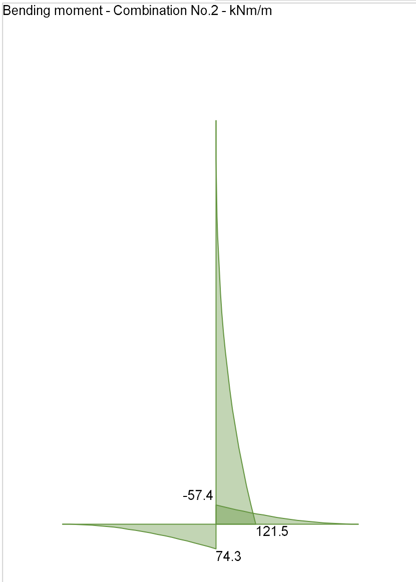

Loading details - Combination No.1 - kN/m²
7.16
Stem
Toe
Heel
116.68
76.52
66.48
29.19


Shear force - Combination No.1 - kN/m
110.2
88
-103.4


Bending moment - Combination No.1 - kNm/m
-62.4
141.6
83.7







Check stem design at base of stem

Depth of section; $h = \mathbf{350}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; $M = \mathbf{141.6}$ kNm/m

Depth to tension reinforcement; $d = h - c_{sr} - \phi_{sr} / 2 = \mathbf{294}$ mm

$K = M / (d^2 \times f_{ck}) = \mathbf{0.047}$

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

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d =$ **279** mm

Depth of neutral axis; $x = 2.5 \times (d - z) =$ **37** mm

Area of tension reinforcement required; $A_{sr.req} = M / (f_{yd} \times z) =$ **1166** mm^2/m

Tension reinforcement provided; 12 dia.bars @ 50 c/c

Area of tension reinforcement provided; $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) =$ **2262** mm^2/m

Minimum area of reinforcement - exp.9.1N; $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d =$ **491** mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{sr.max} = 0.04 \times h =$ **14000** mm^2/m

$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov} =$ **0.516**

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; $\rho_0 = \sqrt{(f_{ck} / 1\ N/mm^2)} / 1000 =$ **0.006**

Required tension reinforcement ratio; $\rho = A_{sr.req} / d =$ **0.004**

Required compression reinforcement ratio; $\rho' = A_{sr.2.req} / d_2 =$ **0.000**

Structural system factor - Table 7.4N; $K_b =$ **0.4**

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

Limiting span to depth ratio - exp.7.16.a; $\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b) =$ **16**

Actual span to depth ratio; $h_{stem} / d =$ **12.4**

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width; $w_{max} =$ **0.3** mm

Variable load factor - EN1990 – Table A1.1; $\psi_2 =$ **0.6**

Serviceability bending moment; $M_{sls} =$ **88.7** kNm/m

Tensile stress in reinforcement; $\sigma_s = M_{sls} / (A_{sr.prov} \times z) =$ **140.4** N/mm^2

Load duration; Long term

Load duration factor; $k_t =$ **0.4**

Effective area of concrete in tension; $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$

$A_{c.eff} =$ **104417** mm^2/m

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm} =$ **3.2** N/mm^2

Reinforcement ratio; $\rho_{p.eff} = A_{sr.prov} / A_{c.eff} =$ **0.022**

Modular ratio; $\alpha_e = E_s / E_{cm} =$ **5.869**

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; $s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff} =$ **264** mm

Maximum crack width - exp.7.8; $w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

$w_k =$ **0.111** mm

$w_k / w_{max} =$ **0.371**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force; $V =$ **103.4** kN/m

$C_{Rd,c} = 0.18 / \gamma_C =$ **0.120**

$k = \min(1 + \sqrt{(200\ mm / d)}, 2) =$ **1.825**

Longitudinal reinforcement ratio; $\rho_l = \min(A_{sr.prov} / d, 0.02) = \mathbf{0.008}$

$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.510}\ N/mm^2$

Design shear resistance - exp.6.2a & 6.2b; $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c} = \mathbf{193}$ kN/m

$V / V_{Rd.c} = \mathbf{0.536}$

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); $A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem}) = \mathbf{565}\ mm^2/m$

Maximum spacing of reinforcement – cl.9.6.3(2); $s_{sx\_max} = \mathbf{400}$ mm

Transverse reinforcement provided; 12 dia.bars @ 100 c/c

Area of transverse reinforcement provided; $A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx}) = \mathbf{1131}\ mm^2/m$

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

Check base design at toe

Depth of section; $h = \mathbf{350}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; $M = \mathbf{83.7}$ kNm/m

Depth to tension reinforcement; $d = h - c_{bb} - \phi_{bb} / 2 = \mathbf{294}$ mm

$K = M / (d^2 \times f_{ck}) = \mathbf{0.028}$

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

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d =$ **279** mm

Depth of neutral axis; $x = 2.5 \times (d – z) = \mathbf{37}$ mm

Area of tension reinforcement required; $A_{bb.req} = M / (f_{yd} \times z) = \mathbf{689}\ mm^2/m$

Tension reinforcement provided; 12 dia.bars @ 100 c/c

Area of tension reinforcement provided; $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) = \mathbf{1131}\ mm^2/m$

Minimum area of reinforcement - exp.9.1N; $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{491}\ mm^2/m$

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{bb.max} = 0.04 \times h = \mathbf{14000}\ mm^2/m$

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov} = \mathbf{0.609}$

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; $w_{max} = \mathbf{0.3}$ mm

Variable load factor - EN1990 – Table A1.1; $\psi_2 = \mathbf{0.6}$

Serviceability bending moment; $M_{sls} = \mathbf{60.2}$ kNm/m

Tensile stress in reinforcement; $\sigma_s = M_{sls} / (A_{bb.prov} \times z) = \mathbf{190.7}\ N/mm^2$

Load duration; Long term

Load duration factor; $k_t = \mathbf{0.4}$

Effective area of concrete in tension; $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$

$A_{c.eff} = \mathbf{104417}\ mm^2/m$

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm} = \mathbf{3.2}\ N/mm^2$

Reinforcement ratio; $\rho_{p.eff} = A_{bb.prov} / A_{c.eff} = \mathbf{0.011}$

Modular ratio; $\alpha_e = E_s / E_{cm} = \mathbf{5.869}$

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; $s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff}$ = **358** 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 = **0.205** mm

w_k / w_{max} = **0.683**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force; V = **110.2** kN/m

$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.825**

Longitudinal reinforcement ratio; $\rho_l = \min(A_{bb.prov} / d, 0.02)$ = **0.004**

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.510** N/mm^2

Design shear resistance - exp.6.2a & 6.2b; $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c}$ = **153.2** kN/m

$V / V_{Rd.c}$ = **0.720**

PASS - Design shear resistance exceeds design shear force

Check base design at heel

Depth of section; h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; M = **62.4** kNm/m

Depth to tension reinforcement; $d = h - c_{bt} - \phi_{bt} / 2$ = **294** mm

$K = M / (d^2 \times f_{ck})$ = **0.021**

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

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = **279** mm

Depth of neutral axis; $x = 2.5 \times (d - z)$ = **37** mm

Area of tension reinforcement required; $A_{bt.req} = M / (f_{yd} \times z)$ = **514** mm^2/m

Tension reinforcement provided; 12 dia.bars @ 100 c/c

Area of tension reinforcement provided; $A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt})$ = **1131** mm^2/m

Minimum area of reinforcement - exp.9.1N; $A_{bt.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **491** mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{bt.max} = 0.04 \times h$ = **14000** mm^2/m

$\max(A_{bt.req}, A_{bt.min}) / A_{bt.prov}$ = **0.455**

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} = **0.3** mm

Variable load factor - EN1990 – Table A1.1; ψ_2 = **0.6**

Serviceability bending moment; M_{sls} = **41.2** kNm/m

Tensile stress in reinforcement;	$\sigma_s = M_{sls} / (A_{bt.prov} \times z)$ = **130.3** N/mm^2
Load duration;	Long term
Load duration factor;	k_t = **0.4**
Effective area of concrete in tension;	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$
	$A_{c.eff}$ = **104417** mm^2/m
Mean value of concrete tensile strength;	$f_{ct.eff} = f_{ctm}$ = **3.2** N/mm^2
Reinforcement ratio;	$\rho_{p.eff} = A_{bt.prov} / A_{c.eff}$ = **0.011**
Modular ratio;	$\alpha_e = E_s / E_{cm}$ = **5.869**
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;	$s_{r.max} = k_3 \times c_{bt} + k_1 \times k_2 \times k_4 \times \phi_{bt} / \rho_{p.eff}$ = **358** 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 = **0.14** mm
	w_k / w_{max} = **0.467**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

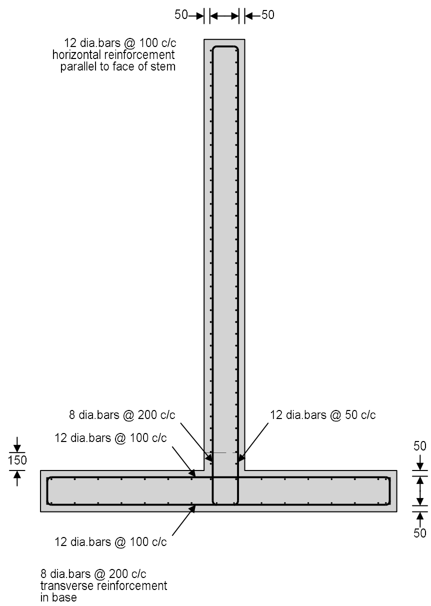
Design shear force;	V = **88** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**
	$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.825**
Longitudinal reinforcement ratio;	$\rho_l = \min(A_{bt.prov} / d, 0.02)$ = **0.004**
	$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.510** N/mm^2
Design shear resistance - exp.6.2a & 6.2b;	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c}$ = **153.2** kN/m
	$V / V_{Rd.c}$ = **0.574**

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);	$A_{bx.req} = 0.2 \times A_{bb.prov}$ = **226** mm^2/m
Maximum spacing of reinforcement – cl.9.3.1.1(3);	s_{bx_max} = **450** mm
Transverse reinforcement provided;	8 dia.bars @ 200 c/c
Area of transverse reinforcement provided;	$A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx})$ = **251** mm^2/m

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



Reinforcement details

RETAINING WALL ANALYSIS LOAD CASE 2 (GROUND WATER PRESENT):

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.9.07

Retaining wall details

Stem type;	Cantilever
Stem height;	h_{stem} = **3650** mm
Stem thickness;	t_{stem} = **350** mm
Angle to rear face of stem;	α = **90** deg
Stem density;	γ_{stem} = **25** kN/m^3
Toe length;	l_{toe} = **1400** mm
Heel length;	l_{heel} = **1300** mm
Base thickness;	t_{base} = **350** mm
Base density;	γ_{base} = **25** kN/m^3
Height of retained soil;	h_{ret} = **3400** mm
Angle of soil surface;	β = **0** deg
Depth of cover;	d_{cover} = **250** mm
Depth of excavation;	d_{exc} = **250** mm
Height of water;	h_{water} = **2400** mm
Water density;	γ_w = **9.8** kN/m^3

Retained soil properties

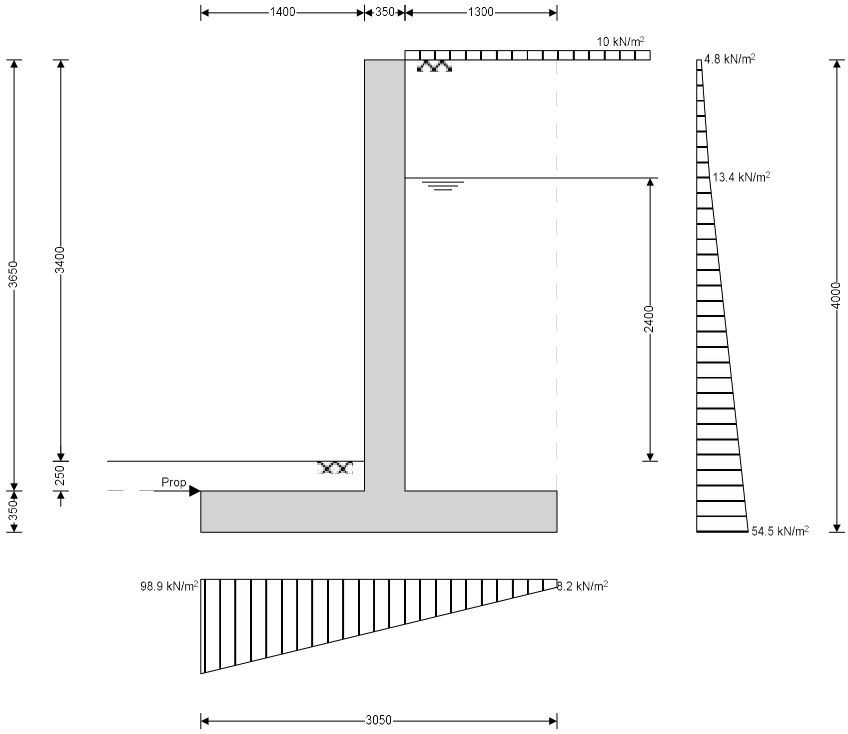
Soil type;	Firm clay
Moist density;	γ_{mr} = **18** kN/m^3
Saturated density;	γ_{sr} = **18** kN/m^3
Characteristic effective shear resistance angle;	$\phi'_{r.k}$ = **18** deg
Characteristic wall friction angle;	$\delta_{r.k}$ = **9** deg

Base soil properties

Soil type;	Firm clay
Soil density;	γ_b = **18** kN/m^3
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
Presumed bearing capacity;	$P_{bearing}$ = **100** kN/m^2

Loading details

Variable surcharge load;	$Surcharge_Q$ = **10** kN/m^2



General arrangement

Calculate retaining wall geometry

Base length;	$l_{base} = l_{toe} + t_{stem} + l_{heel} = \mathbf{3050}$ mm
Saturated soil height;	$h_{sat} = h_{water} + d_{cover} = \mathbf{2650}$ mm
Moist soil height;	$h_{moist} = h_{ret} - h_{water} = \mathbf{1000}$ mm
Length of surcharge load;	$l_{sur} = l_{heel} = \mathbf{1300}$ mm
- Distance to vertical component;	$x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{2400}$ mm
Effective height of wall;	$h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{4000}$ mm
- Distance to horizontal component;	$x_{sur\_h} = h_{eff} / 2 = \mathbf{2000}$ mm
Area of wall stem;	$A_{stem} = h_{stem} \times t_{stem} = \mathbf{1.278}$ m^2
- Distance to vertical component;	$x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{1575}$ mm
Area of wall base;	$A_{base} = l_{base} \times t_{base} = \mathbf{1.068}$ m^2
- Distance to vertical component;	$x_{base} = l_{base} / 2 = \mathbf{1525}$ mm
Area of saturated soil;	$A_{sat} = h_{sat} \times l_{heel} = \mathbf{3.445}$ m^2
- Distance to vertical component;	$x_{sat\_v} = l_{base} - (h_{sat} \times l_{heel}^2 / 2) / A_{sat} = \mathbf{2400}$ mm
- Distance to horizontal component;	$x_{sat\_h} = (h_{sat} + h_{base}) / 3 = \mathbf{1000}$ mm
Area of water;	$A_{water} = h_{sat} \times l_{heel} = \mathbf{3.445}$ m^2
- Distance to vertical component;	$x_{water\_v} = l_{base} - (h_{sat} \times l_{heel}^2 / 2) / A_{sat} = \mathbf{2400}$ mm
- Distance to horizontal component;	$x_{water\_h} = (h_{sat} + h_{base}) / 3 = \mathbf{1000}$ mm
Area of moist soil;	$A_{moist} = h_{moist} \times l_{heel} = \mathbf{1.3}$ m^2
- Distance to vertical component;	$x_{moist\_v} = l_{base} - (h_{moist} \times l_{heel}^2 / 2) / A_{moist} = \mathbf{2400}$ mm

- Distance to horizontal component; $x_{moist\_h} = (h_{moist} \times (t_{base} + h_{sat} + h_{moist} / 3) / 2 + (h_{sat} + t_{base})^2/2) / (h_{sat} + t_{base} + h_{moist} / 2) = \mathbf{1762}$ mm

Area of base soil; $A_{pass} = d_{cover} \times l_{toe} = \mathbf{0.35}$ m^2

- Distance to vertical component; $x_{pass\_v} = l_{base} - (d_{cover} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{pass} = \mathbf{700}$ mm
- Distance to horizontal component; $x_{pass\_h} = (d_{cover} + h_{base}) / 3 = \mathbf{200}$ mm

Using Coulomb theory

Active pressure coefficient; $K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]}]^2) = \mathbf{0.483}$

Passive pressure coefficient; $K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2) = \mathbf{2.359}$

Bearing pressure check

Vertical forces on wall

Wall stem; $F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{31.9}$ kN/m

Wall base; $F_{base} = A_{base} \times \gamma_{base} = \mathbf{26.7}$ kN/m

Surcharge load; $F_{sur\_v} = Surcharge_Q \times l_{heel} = \mathbf{13}$ kN/m

Saturated retained soil; $F_{sat\_v} = A_{sat} \times (\gamma_{sr} - \gamma_w) = \mathbf{28.2}$ kN/m

Water; $F_{water\_v} = A_{water} \times \gamma_w = \mathbf{33.8}$ kN/m

Moist retained soil; $F_{moist\_v} = A_{moist} \times \gamma_{mr} = \mathbf{23.4}$ kN/m

Base soil; $F_{pass\_v} = A_{pass} \times \gamma_b = \mathbf{6.3}$ kN/m

Total; $F_{total\_v} = F_{stem} + F_{base} + F_{sur\_v} + F_{sat\_v} + F_{water\_v} + F_{moist\_v} + F_{pass\_v} = \mathbf{163.3}$ kN/m

Horizontal forces on wall

Surcharge load; $F_{sur\_h} = K_A \times \cos(\delta_{r.k}) \times Surcharge_Q \times h_{eff} = \mathbf{19.1}$ kN/m

Saturated retained soil; $F_{sat\_h} = K_A \times \cos(\delta_{r.k}) \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2 = \mathbf{17.6}$ kN/m

Water; $F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2 = \mathbf{44.1}$ kN/m

Moist retained soil; $F_{moist\_h} = K_A \times \cos(\delta_{r.k}) \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) = \mathbf{30.1}$ kN/m

Base soil; $F_{pass\_h} = -K_P \times \cos(\delta_{b.k}) \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 = \mathbf{-7.5}$ kN/m

Total; $F_{total\_h} = F_{sur\_h} + F_{sat\_h} + F_{water\_h} + F_{moist\_h} + F_{pass\_h} = \mathbf{103.3}$ kN/m

Moments on wall

Wall stem; $M_{stem} = F_{stem} \times x_{stem} = \mathbf{50.3}$ kNm/m

Wall base; $M_{base} = F_{base} \times x_{base} = \mathbf{40.7}$ kNm/m

Surcharge load; $M_{sur} = F_{sur\_v} \times x_{sur\_v} - F_{sur\_h} \times x_{sur\_h} = \mathbf{-7}$ kNm/m

Saturated retained soil; $M_{sat} = F_{sat\_v} \times x_{sat\_v} - F_{sat\_h} \times x_{sat\_h} = \mathbf{50.1}$ kNm/m

Water; $M_{water} = F_{water\_v} \times x_{water\_v} - F_{water\_h} \times x_{water\_h} = \mathbf{37}$ kNm/m

Moist retained soil; $M_{moist} = F_{moist\_v} \times x_{moist\_v} - F_{moist\_h} \times x_{moist\_h} = \mathbf{3.2}$ kNm/m

Base soil; $M_{pass} = F_{pass\_v} \times x_{pass\_v} = \mathbf{4.4}$ kNm/m

Total; $M_{total} = M_{stem} + M_{base} + M_{sur} + M_{sat} + M_{water} + M_{moist} + M_{pass} = \mathbf{178.7}$ kNm/m

Check bearing pressure

Propping force; $F_{prop\_base} = F_{total\_h} = \mathbf{103.3}$ kN/m

Distance to reaction;	$\bar{x} = M_{total} / F_{total\_v}$ = **1094** mm
Eccentricity of reaction;	$e = \bar{x} - l_{base} / 2$ = **-431** mm
Loaded length of base;	$l_{load} = l_{base}$ = **3050** mm
Bearing pressure at toe;	$q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base})$ = **98.9** kN/m^2
Bearing pressure at heel;	$q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base})$ = **8.2** kN/m^2
Factor of safety;	$FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel})$ = **1.011**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

<u>RETAINING WALL DESIGN</u>

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 2.9.07

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

Concrete strength class;	C35/45
Characteristic compressive cylinder strength;	f_{ck} = **35** N/mm^2
Characteristic compressive cube strength;	$f_{ck,cube}$ = **45** N/mm^2
Mean value of compressive cylinder strength;	$f_{cm} = f_{ck} + 8$ N/mm^2 = **43** N/mm^2
Mean value of axial tensile strength;	$f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck} / 1$ N/mm$^2)^{2/3}$ = **3.2** N/mm^2
5% fractile of axial tensile strength;	$f_{ctk,0.05} = 0.7 \times f_{ctm}$ = **2.2** N/mm^2
Secant modulus of elasticity of concrete;	$E_{cm} = 22$ kN/mm$^2 \times (f_{cm} / 10$ N/mm$^2)^{0.3}$ = **34077** N/mm^2
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;	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **19.8** 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**

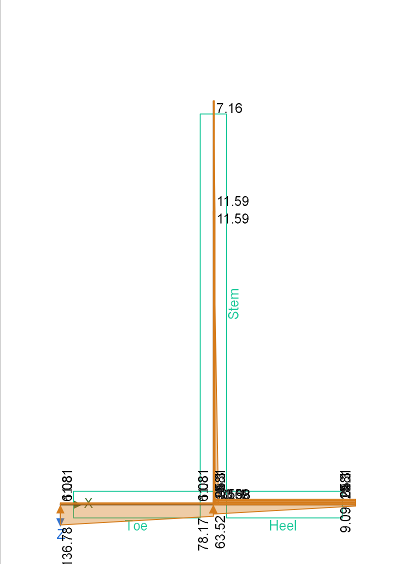
Reinforcement details

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

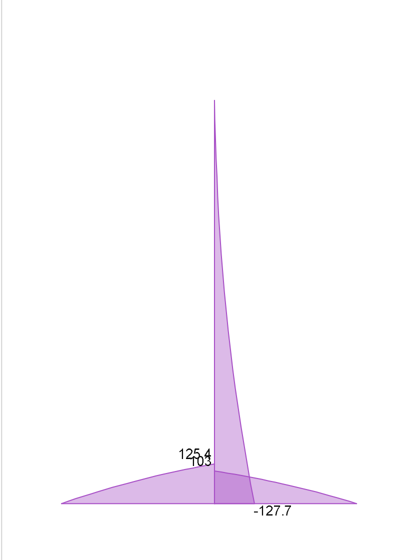
Cover to reinforcement

Front face of stem;	c_{sf} = **50** mm
Rear face of stem;	c_{sr} = **50** mm
Top face of base;	c_{bt} = **50** mm
Bottom face of base;	c_{bb} = **50** mm

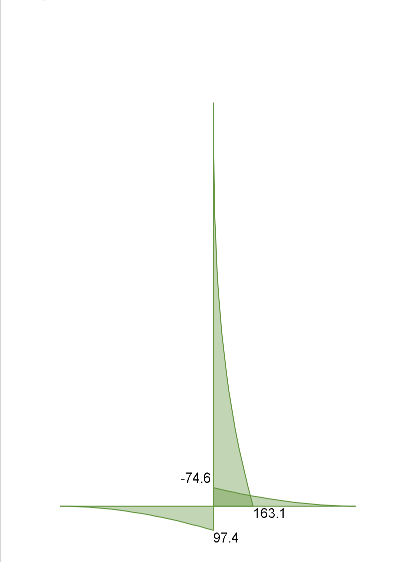
Loading details - Combination No.1 - kN/m²

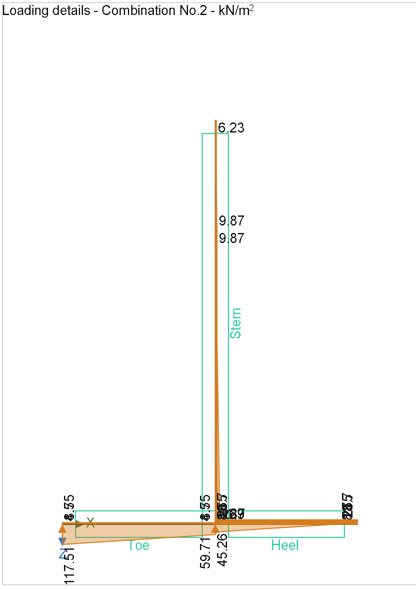


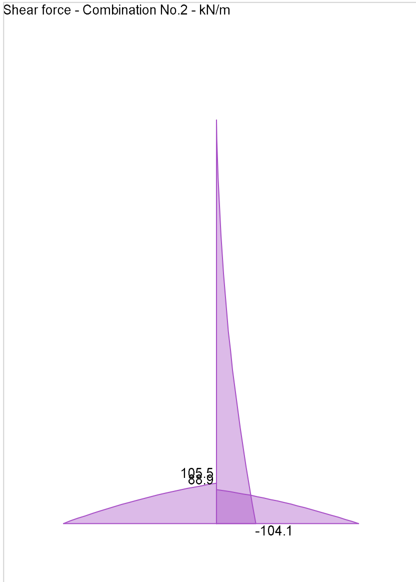
Shear force - Combination No.1 - kN/m

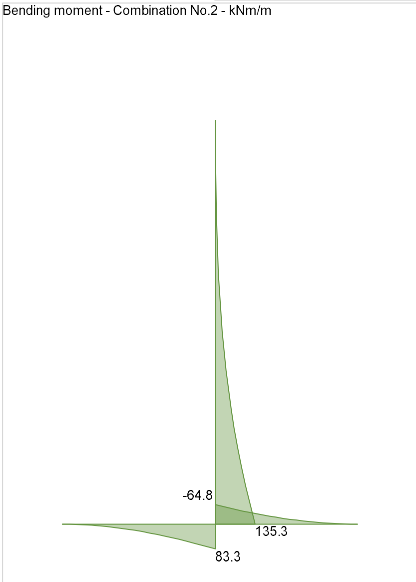


Bending moment - Combination No.1 - kNm/m









Check stem design at base of stem

Depth of section; h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; M = **163.1** kNm/m

Depth to tension reinforcement; $d = h - c_{sr} - \phi_{sr} / 2$ = **294** mm

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

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

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = **279** mm

Depth of neutral axis;	$x = 2.5 \times (d - z) = \mathbf{37}$ mm
Area of tension reinforcement required;	$A_{sr.req} = M / (f_{yd} \times z) = \mathbf{1343}$ mm²/m
Tension reinforcement provided;	12 dia.bars @ 50 c/c
Area of tension reinforcement provided;	$A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = \mathbf{2262}$ mm²/m
Minimum area of reinforcement - exp.9.1N;	$A_{sr.min} = max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{491}$ mm²/m
Maximum area of reinforcement - cl.9.2.1.1(3);	$A_{sr.max} = 0.04 \times h = \mathbf{14000}$ mm²/m
	$max(A_{sr.req}, A_{sr.min}) / A_{sr.prov} = \mathbf{0.594}$

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;	$\rho_0 = \sqrt{(f_{ck} / 1\ N/mm^2)} / 1000 = \mathbf{0.006}$
Required tension reinforcement ratio;	$\rho = A_{sr.req} / d = \mathbf{0.005}$
Required compression reinforcement ratio;	$\rho' = A_{sr.2.req} / d_2 = \mathbf{0.000}$
Structural system factor - Table 7.4N;	$K_b = \mathbf{0.4}$
Reinforcement factor - exp.7.17;	$K_s = min(500\ N/mm^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5) = \mathbf{1.5}$
Limiting span to depth ratio - exp.7.16.a;	$min(K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b) = \mathbf{15.3}$
Actual span to depth ratio;	$h_{stem} / d = \mathbf{12.4}$

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width;	$w_{max} = \mathbf{0.3}$ mm
Variable load factor - EN1990 – Table A1.1;	$\psi_2 = \mathbf{0.6}$
Serviceability bending moment;	$M_{sls} = \mathbf{104.6}$ kNm/m
Tensile stress in reinforcement;	$\sigma_s = M_{sls} / (A_{sr.prov} \times z) = \mathbf{165.6}$ N/mm²
Load duration;	Long term
Load duration factor;	$k_t = \mathbf{0.4}$
Effective area of concrete in tension;	$A_{c.eff} = min(2.5 \times (h - d), (h - x) / 3, h / 2)$
	$A_{c.eff} = \mathbf{104397}$ mm²/m
Mean value of concrete tensile strength;	$f_{ct.eff} = f_{ctm} = \mathbf{3.2}$ N/mm²
Reinforcement ratio;	$\rho_{p.eff} = A_{sr.prov} / A_{c.eff} = \mathbf{0.022}$
Modular ratio;	$\alpha_e = E_s / E_{cm} = \mathbf{5.869}$
Bond property coefficient;	$k_1 = \mathbf{0.8}$
Strain distribution coefficient;	$k_2 = \mathbf{0.5}$
	$k_3 = \mathbf{3.4}$
	$k_4 = \mathbf{0.425}$
Maximum crack spacing - exp.7.11;	$s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff} = \mathbf{264}$ mm
Maximum crack width - exp.7.8;	$w_k = s_{r.max} \times max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	$w_k = \mathbf{0.131}$ mm
	$w_k / w_{max} = \mathbf{0.437}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force;	$V = \mathbf{127.7}$ kN/m
	$C_{Rd,c} = 0.18 / \gamma_C = \mathbf{0.120}$
	$k = min(1 + \sqrt{(200\ mm / d)}, 2) = \mathbf{1.825}$

Longitudinal reinforcement ratio; $\rho_l = \min(A_{sr.prov} / d, 0.02) = \mathbf{0.008}$

$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.510}\ N/mm^2$

Design shear resistance - exp.6.2a & 6.2b; $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c} = \mathbf{193}$ kN/m

$V / V_{Rd.c} = \mathbf{0.662}$

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); $A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem}) = \mathbf{565}\ mm^2/m$

Maximum spacing of reinforcement – cl.9.6.3(2); $s_{sx\_max} = \mathbf{400}$ mm

Transverse reinforcement provided; 12 dia.bars @ 100 c/c

Area of transverse reinforcement provided; $A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx}) = \mathbf{1131}\ mm^2/m$

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

Check base design at toe

Depth of section; $h = \mathbf{350}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; $M = \mathbf{97.4}$ kNm/m

Depth to tension reinforcement; $d = h - c_{bb} - \phi_{bb} / 2 = \mathbf{294}$ mm

$K = M / (d^2 \times f_{ck}) = \mathbf{0.032}$

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

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = \mathbf{279}$ mm

Depth of neutral axis; $x = 2.5 \times (d – z) = \mathbf{37}$ mm

Area of tension reinforcement required; $A_{bb.req} = M / (f_{yd} \times z) = \mathbf{802}\ mm^2/m$

Tension reinforcement provided; 12 dia.bars @ 100 c/c

Area of tension reinforcement provided; $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) = \mathbf{1131}\ mm^2/m$

Minimum area of reinforcement - exp.9.1N; $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{491}\ mm^2/m$

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{bb.max} = 0.04 \times h = \mathbf{14000}\ mm^2/m$

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov} = \mathbf{0.709}$

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; $w_{max} = \mathbf{0.3}$ mm

Variable load factor - EN1990 – Table A1.1; $\psi_2 = \mathbf{0.6}$

Serviceability bending moment; $M_{sls} = \mathbf{70.4}$ kNm/m

Tensile stress in reinforcement; $\sigma_s = M_{sls} / (A_{bb.prov} \times z) = \mathbf{222.7}\ N/mm^2$

Load duration; Long term

Load duration factor; $k_t = \mathbf{0.4}$

Effective area of concrete in tension; $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$

$A_{c.eff} = \mathbf{104417}\ mm^2/m$

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm} = \mathbf{3.2}\ N/mm^2$

Reinforcement ratio; $\rho_{p.eff} = A_{bb.prov} / A_{c.eff} = \mathbf{0.011}$

Modular ratio; $\alpha_e = E_s / E_{cm} = \mathbf{5.869}$

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; $s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff}$ = **358** 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 = **0.239** mm
w_k / w_{max} = **0.798**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force; V = **125.4** kN/m
$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**
$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **1.825**
Longitudinal reinforcement ratio; $\rho_l = \min(A_{bb.prov} / d, 0.02)$ = **0.004**
$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.510** N/mm^2
Design shear resistance - exp.6.2a & 6.2b; $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
$V_{Rd.c}$ = **153.2** kN/m
$V / V_{Rd.c}$ = **0.819**

PASS - Design shear resistance exceeds design shear force

Check base design at heel

Depth of section; h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; M = **74.6** kNm/m
Depth to tension reinforcement; $d = h - c_{bt} - \phi_{bt} / 2$ = **294** mm
$K = M / (d^2 \times f_{ck})$ = **0.025**
$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$
K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = **279** mm
Depth of neutral axis; $x = 2.5 \times (d - z)$ = **37** mm
Area of tension reinforcement required; $A_{bt.req} = M / (f_{yd} \times z)$ = **614** mm^2/m
Tension reinforcement provided; 12 dia.bars @ 100 c/c
Area of tension reinforcement provided; $A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt})$ = **1131** mm^2/m
Minimum area of reinforcement - exp.9.1N; $A_{bt.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **491** mm^2/m
Maximum area of reinforcement - cl.9.2.1.1(3); $A_{bt.max} = 0.04 \times h$ = **14000** mm^2/m
$\max(A_{bt.req}, A_{bt.min}) / A_{bt.prov}$ = **0.543**

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} = **0.3** mm
Variable load factor - EN1990 – Table A1.1; ψ_2 = **0.6**
Serviceability bending moment; M_{sls} = **50.2** kNm/m

Tensile stress in reinforcement;	$\sigma_s = M_{sls} / (A_{bt.prov} \times z) =$ **158.8** N/mm^2
Load duration;	Long term
Load duration factor;	$k_t =$ **0.4**
Effective area of concrete in tension;	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$
	$A_{c.eff} =$ **104417** mm^2/m
Mean value of concrete tensile strength;	$f_{ct.eff} = f_{ctm} =$ **3.2** N/mm^2
Reinforcement ratio;	$\rho_{p.eff} = A_{bt.prov} / A_{c.eff} =$ **0.011**
Modular ratio;	$\alpha_e = E_s / E_{cm} =$ **5.869**
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;	$s_{r.max} = k_3 \times c_{bt} + k_1 \times k_2 \times k_4 \times \phi_{bt} / \rho_{p.eff} =$ **358** mm
Maximum crack width - exp.7.8;	$w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	$w_k =$ **0.171** mm
	$w_k / w_{max} =$ **0.569**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

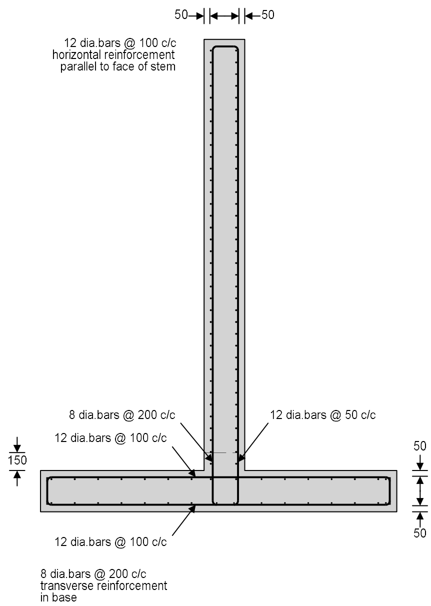
Design shear force;	$V =$ **103** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C =$ **0.120**
	$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) =$ **1.825**
Longitudinal reinforcement ratio;	$\rho_l = \min(A_{bt.prov} / d, 0.02) =$ **0.004**
	$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} =$ **0.510** N/mm^2
Design shear resistance - exp.6.2a & 6.2b;	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c} =$ **153.2** kN/m
	$V / V_{Rd.c} =$ **0.672**

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);	$A_{bx.req} = 0.2 \times A_{bb.prov} =$ **226** mm^2/m
Maximum spacing of reinforcement – cl.9.3.1.1(3);	$s_{bx\_max} =$ **450** mm
Transverse reinforcement provided;	8 dia.bars @ 200 c/c
Area of transverse reinforcement provided;	$A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx}) =$ **251** mm^2/m

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



Reinforcement details