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25 OAKHILL AVENUE, LONDON, NW3 7RD

Structural Engineer's Calculations for Planning

October 2022

Revision P1- Planning

Revision	Issued For	Date	Author
P1	PLANNING	20.10.2021	AZ

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1. **INTRODUCTION:**

   This project covers the design of the refurbishment and the new basement extension at No 25 Oakhill Avenue. The current calculation includes the design of a reinforced concrete basement slab, underpinning of party walls, design of lining walls and new concrete slab above basement to accommodate new Ground Floor layouts as well as the structural elements for the upper floors.

2. **RELEVANT DOCUMENTS:**
   - Site geological investigation carried out by GEA Geotechnical Engineers.
   - MBP's Construction Method Statement
   - MBP's Specification for the works
   - MBP's Structural Drawings for the works

3. **STRUCTURAL DRAWINGS:**
   - MBP-8536-100- PROPOSED BASEMENT GENERAL ARRANGEMENT
   - MBP-8536-101- PROPOSED GROUND FLOOR GENERAL ARRANGEMENT
   - MBP-8536-102- PROPOSED FIRST FLOOR GENERAL ARRANGEMENT
   - MBP-8536-103- PROPOSED SECOND FLOOR GENERAL ARRANGEMENT
   - MBP-8536-104- PROPOSED THIRD FLOOR GENERAL ARRANGEMENT
   - MBP-8536-103- PROPOSED ROOF GENERAL ARRANGEMENT
   - MBP-8536-200- PROPOSED SECTION 1-1 GENERAL ARRANGEMENT
   - MBP-8236-210- PROPOSED SECTION A-A GENERAL ARRANGEMENT
   - MBP-8236-211- PROPOSED SECTION B-B GENERAL ARRANGEMENT

4. **SECOND FLOOR STRCUTURE**

   The analysis and design of steel and timber elements has been carried out using TEDDS. The steel beams and timber joists has been designed to carry out the domestic loads. Results can be found in Section 4.

5. **FIRST FLOOR STRUCTURE**

   The analysis and design of the Flat Roof steel elements has been carried out using TEDDS. The steel beams have been designed to support existing walls above and new flat roof structure with rooflights. Results can be found in Section 5.

6. **GROUND FLOOR SLAB STRUCTURE**

   The ground floor will be reinforced concrete spanning between liner walls and internal loadbering elements, the maximum span for the slab to be 6.0m the verification has been carried out using TEDDS. Results can be found in Section 6.

7. **BASEMENT RETAINING WALLS**

   The analysis and design of the RC liner walls, has been carried out using TEDDS. The liner walls have been designed to support ground floor load. Results can be found in Section 8.

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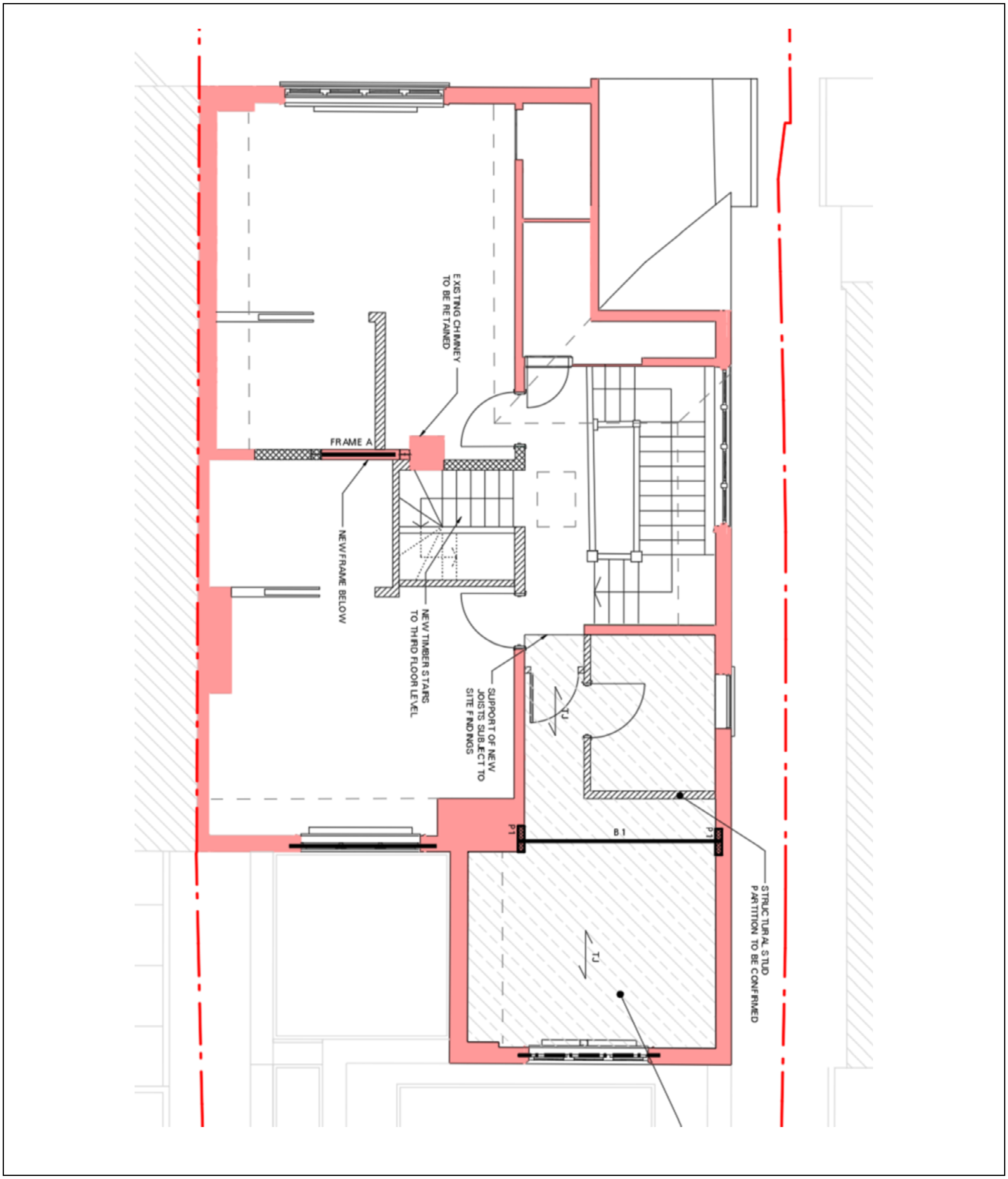
Job Title	Job Number	Sheet Number	Revision
25 OAKHILL AVENUE, LONDON	8536	4.1	P1
Calculation/sketch Title	**Date**	**Author**	**Checked**
SECTION 4 SECOND FLOOR STRUCTURE	OCT 2022	AZ	TH

GEOMETRY



SECOND FLOOR PLAN

DEAD LOAD

Floor finishes	0.50	
Boarding	0.15	
Timber joists	0.25	
Ceiling and services	0.25	
TOTAL	**1.15**	**kN/m²**

Solid brick wall	**4.30**	**kN/m²**

IMPOSED LOAD

Imposed Load (Including Partitions)	**2.50**	**kN/m²**

TYPICAL TIMBER JOISTS

Max. span 3.3m

	Dead load (excluding self weight)	Imposed load
Loading	0.9 kN/m^2	2.5 kN/m^2

TIMBER JOIST ANALYSIS & DESIGN (EN1995-1-1:2004)

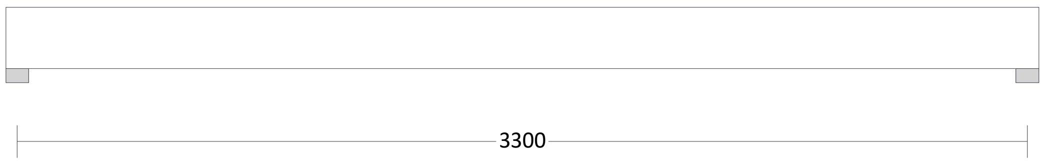
In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex

Tedds calculation version 1.0.05

Joist details

Description 47 x 200 C18 timber joists

Joist spacing s_{Joist} = **350** mm



Forces input on Joist

Vertical permanent load on joist F_{G_Joist}= **0.90** kN/m^2

Vertical imposed load on joist F_{Q_Joist}= **2.50** kN/m^2

Joist loading details

Distributed loads

Vertical permanent load on joist $p_G = F_{G\_Joist} \times s_{Joist}$= **0.32** kN/m

Vertical imposed load on joist $p_Q = F_{Q\_Joist} \times s_{Joist}$= **0.88** kN/m

ANALYSIS

Tedds calculation version 1.0.36

Loading

Self weight included (Permanent x 1)

Load combination factors

Load combination	Permanent	Imposed	Snow	Wind
1.35G + 1.50Q (Strength)	1.35	1.50	0.00	0.00
1.00G + 1.00Q (Service)	1.00	1.00	0.00	0.00

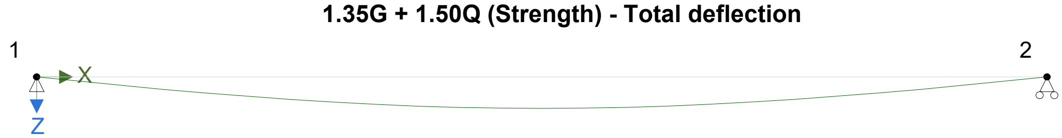
Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.32 kN/m at 0 m to 3.3 m
Member	Imposed	UDL	GlobalZ	0.88 kN/m at 0 m to 3.3 m

Results

Total deflection

1.35G + 1.50Q (Strength) - Total deflection



1.00G + 1.00Q (Service) - Total deflection

Node deflections

Load combination: 1.35G + 1.50Q (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.54079	
2	0	0	-0.54079	

Load combination: 1.00G + 1.00Q (Service)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0.37101	
2	0	0	-0.37101	

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.50Q (Strength)	0	5.9
1.00G + 1.00Q (Service)	0	4

Element end forces

Load combination: 1.35G + 1.50Q (Strength)

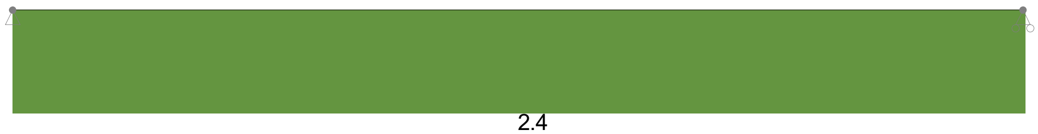
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3.3	1	0	-2.9	0
		2	0	-2.9	0

Load combination: 1.00G + 1.00Q (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3.3	1	0	-2	0
		2	0	-2	0

Forces

Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)

Member results

Envelope - Strength combinations

Member	Position (m)	Shear force (kN)		Moment (kNm)	
Member	0	2.9 (max abs)		0 (min)	
	1.65	0		2.4 (max)	
	3.3	-2.9		0 (min)	

Tedds calculation version 2.2.11

Member - Span 1

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 γ_M = **1.300**

Member details

Load duration - cl.2.3.1.2 Medium-term

Service class - cl.2.3.1.3 2

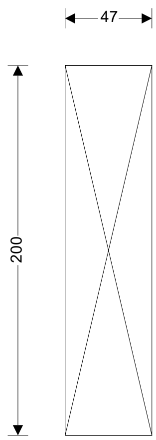
Timber section details

Number of timber sections in member N = **1**

Breadth of sections b = **47** mm

Depth of sections h = **200** mm

Timber strength class - EN 338:2016 Table 1 **C18**



47x200 timber section

Cross-sectional area, A, 9400 mm^2

Section modulus, W_y, 313333.3 mm^3

Section modulus, W_z, 73633 mm^3

Second moment of area, I_y, 31333333 mm^4

Second moment of area, I_z, 1730383 mm^4

Radius of gyration, i_y, 57.7 mm

Radius of gyration, i_z, 13.6 mm

Timber strength class C18

Characteristic bending strength, $f_{m.k}$, 18 N/mm^2

Characteristic shear strength, $f_{v.k}$, 3.4 N/mm^2

Characteristic compression strength parallel to grain, $f_{c.0.k}$, 18 N/mm^2

Characteristic compression strength perpendicular to grain, $f_{c.90.k}$, 2.2 N/mm^2

Characteristic tension strength parallel to grain, $f_{t.0.k}$, 10 N/mm^2

Mean modulus of elasticity, $E_{0.mean}$, 9000 N/mm^2

Fifth percentile modulus of elasticity, $E_{0.05}$, 6000 N/mm^2

Shear modulus of elasticity, G_{mean}, 560 N/mm^2

Characteristic density, ρ_k, 320 kg/m^3

Mean density, ρ_{mean}, 380 kg/m^3

Span details

Bearing length L_b = **100** mm

Member results summary	Unit	Capacity	Maximum	Utilisation	Result
Bearing stress	N/mm^2	1.5	0.6	0.419	PASS
Bending stress	N/mm^2	12.2	7.7	0.634	PASS
Shear stress	N/mm^2	2.3	0.7	0.304	PASS
Deflection	mm	13.2	12.7	0.962	PASS

Consider Combination 1 - 1.35G + 1.50Q (Strength)

Modification factors

Duration of load and moisture content - Table 3.1 k_{mod} = **0.8**

Deformation factor - Table 3.2 k_{def} = **0.8**

Bending stress re-distribution factor - cl.6.1.6(2) k_m = **0.7**

Crack factor for shear resistance - cl.6.1.7(2) k_{cr} = **0.67**

System strength factor - cl.6.6 k_{sys} = **1.1**

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis $F_{c,y,90,d}$ = **2.933** kN

Effective contact length $L_{b,ef} = L_b$ = **100** mm

Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef})$ = **0.624** N/mm^2

Design perpendicular compressive strength $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c.90.k} / \gamma_M$ = **1.489** N/mm^2

$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d})$ = **0.419**

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d}$ = **2.933** kN

Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h)$ = **0.699** N/mm^2

Design shear strength $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M$ = **2.302** N/mm^2

$\tau_{y,d}$ / $f_{v,y,d}$ = **0.304**

PASS - Design shear strength exceeds design shear stress

Check design 1650 mm along span

Check bending moment - Section 6.1.6

Design bending moment $M_{y,d}$ = **2.42** kNm

Design bending stress $\sigma_{m,y,d} = M_{y,d} / W_y$ = **7.723** N/mm^2

Design bending strength $f_{m,y,d} = k_{mod} \times k_{sys} \times f_{m.k} / \gamma_M$ = **12.185** N/mm^2

$\sigma_{m,y,d}$ / $f_{m,y,d}$ = **0.634**

PASS - Design bending strength exceeds design bending stress

Consider Combination 2 - 1.00G + 1.00Q (Service)

Check design 1650 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection δ_y = **7.1** mm

Quasi-permanent variable load factor ψ_2 = **0.3**

Final deflection with creep $\delta_{y,Final} = \delta_y \times (1 + k_{def})$ = **12.7** mm

Allowable deflection $\delta_{y,Allowable} = L_{m1\_s1} / 250$ = **13.2** mm

$\delta_{y,Final}$ / $\delta_{y,Allowable}$ = **0.962**

PASS - Allowable deflection exceeds final deflection

TYPICAL STEEL BEAM B1

Max. span 3.3m

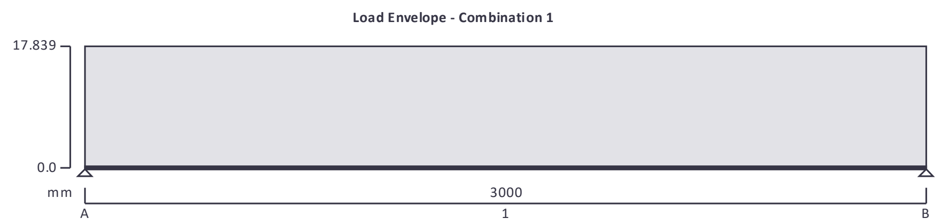
Load attracted from 6.5m/2 = 3.25m

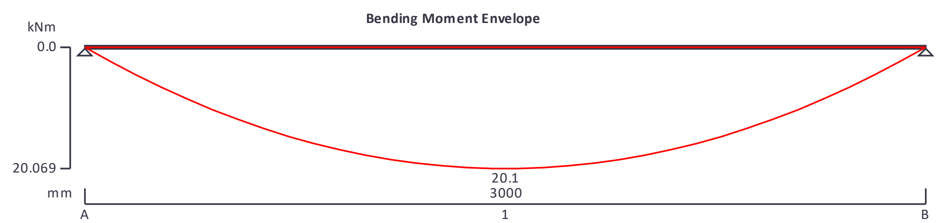
	Dead load	Imposed load
Loading	1.15 x 3.25 = 3.74 kN/m	2.50 x 3.25 = 8.12 kN/m

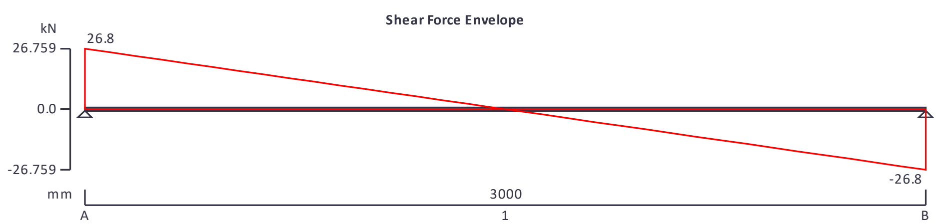
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14







Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam × 1
	Permanent full UDL 3.74 kN/m
	Variable full UDL 8.12 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
		Permanent × 1.35

		Variable $\times$ 1.50
	Support B	Permanent $\times$ 1.35
		Variable $\times$ 1.50

Analysis results

Maximum moment	M_{max} = **20.1** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **26.8** kN	V_{min} = **-26.8** kN
Deflection	δ_{max} = **0.9** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **26.8** kN	R_{A_min} = **26.8** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **6.3** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **12.2** kN	
Maximum reaction at support B	R_{B_max} = **26.8** kN	R_{B_min} = **26.8** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **6.3** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **12.2** kN	

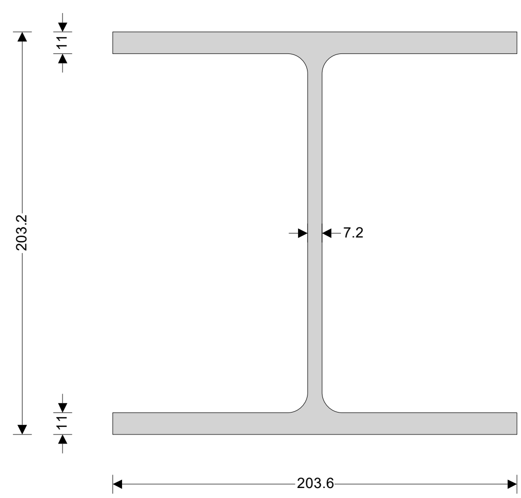
Section details

Section type **UC 203x203x46 (BS4-1)**

Steel grade **S275**

EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element	t = max(t_f, t_w) = **11.0** mm
Nominal yield strength	f_y = **275** N/mm^2
Nominal ultimate tensile strength	f_u = **410** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Resistance of tensile members to fracture	γ_{M2} = **1.10**

Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis K_y = **1.000**

Effective length factor in minor axis K_z = **1.000**

Effective length factor for torsion $K_{LT.A}$ = **1.000**

$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.92**

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section $c = d$ = **160.8** mm

$c / t_w = 24.2 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2$ = **88** mm

$c / t_f = 8.7 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f$ = **181.2** mm

Shear area factor η = **1.000**

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **26.8** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1698** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **269.5** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **20.1** kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **136.8** kNm

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 360$ = **8.3** mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **0.893** mm

PASS - Maximum deflection does not exceed deflection limit

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FRAME A

Max. span 1.4m

Load attracted from 11m/2 = 5.05m

Hight of the wall – 2.6m

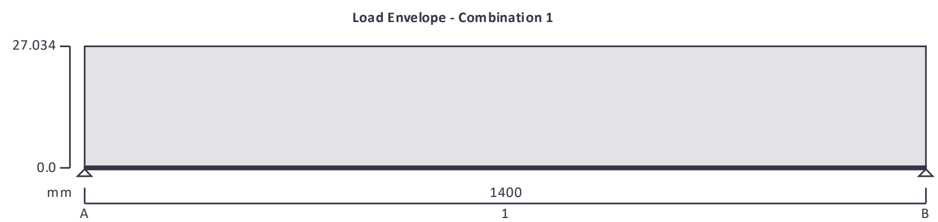
Loading on top memeber

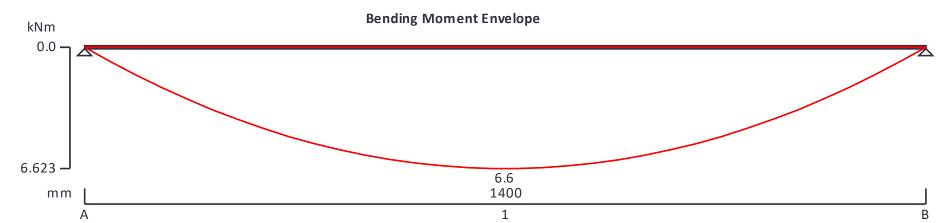
	Dead load	Imposed load
Floor Loading	1.15 x 5.05 = 3.74 kN/m	2.50 x 0.5 = 8.12 kN/m
Wall	4.30 x 2.60 = 6.90 kN/m	
Total	10.64 kN/m	8.12 kN/m

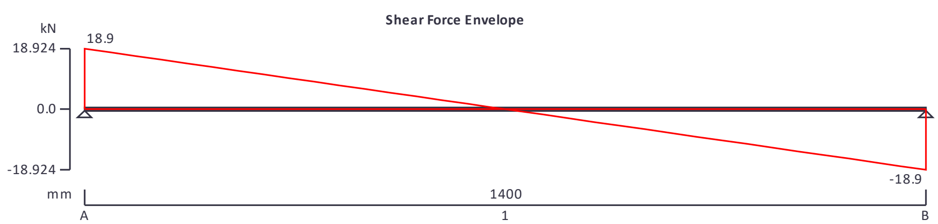
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14







Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam × 1
	Permanent full UDL 10.64 kN/m
	Variable full UDL 8.12 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
		Permanent × 1.35

		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

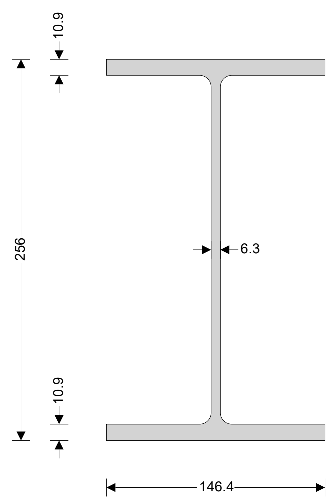
Maximum moment	M_{max} = **6.6** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **18.9** kN	V_{min} = **-18.9** kN
Deflection	δ_{max} = **0** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **18.9** kN	R_{A_min} = **18.9** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **7.7** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **5.7** kN	
Maximum reaction at support B	R_{B_max} = **18.9** kN	R_{B_min} = **18.9** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **7.7** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **5.7** kN	

Section details

Section type	**UB 254x146x37 (BS4-1)**
Steel grade	**S275**

EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element	$t = \max(t_f, t_w)$ = **10.9** mm
Nominal yield strength	f_y = **275** N/mm^2
Nominal ultimate tensile strength	f_u = **410** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Resistance of tensile members to fracture	γ_{M2} = **1.10**

Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis	K_y = **1.000**

Effective length factor in minor axis K_z = **1.000**

Effective length factor for torsion $K_{LT.A}$ = **1.000**

$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.92**

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section $c = d$ = **219** mm

$c / t_w = 37.6 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2$ = **62.5** mm

$c / t_f = 6.2 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f$ = **234.2** mm

Shear area factor η = **1.000**

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **18.9** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1759** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **279.3** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **6.6** kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **132.9** kNm

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

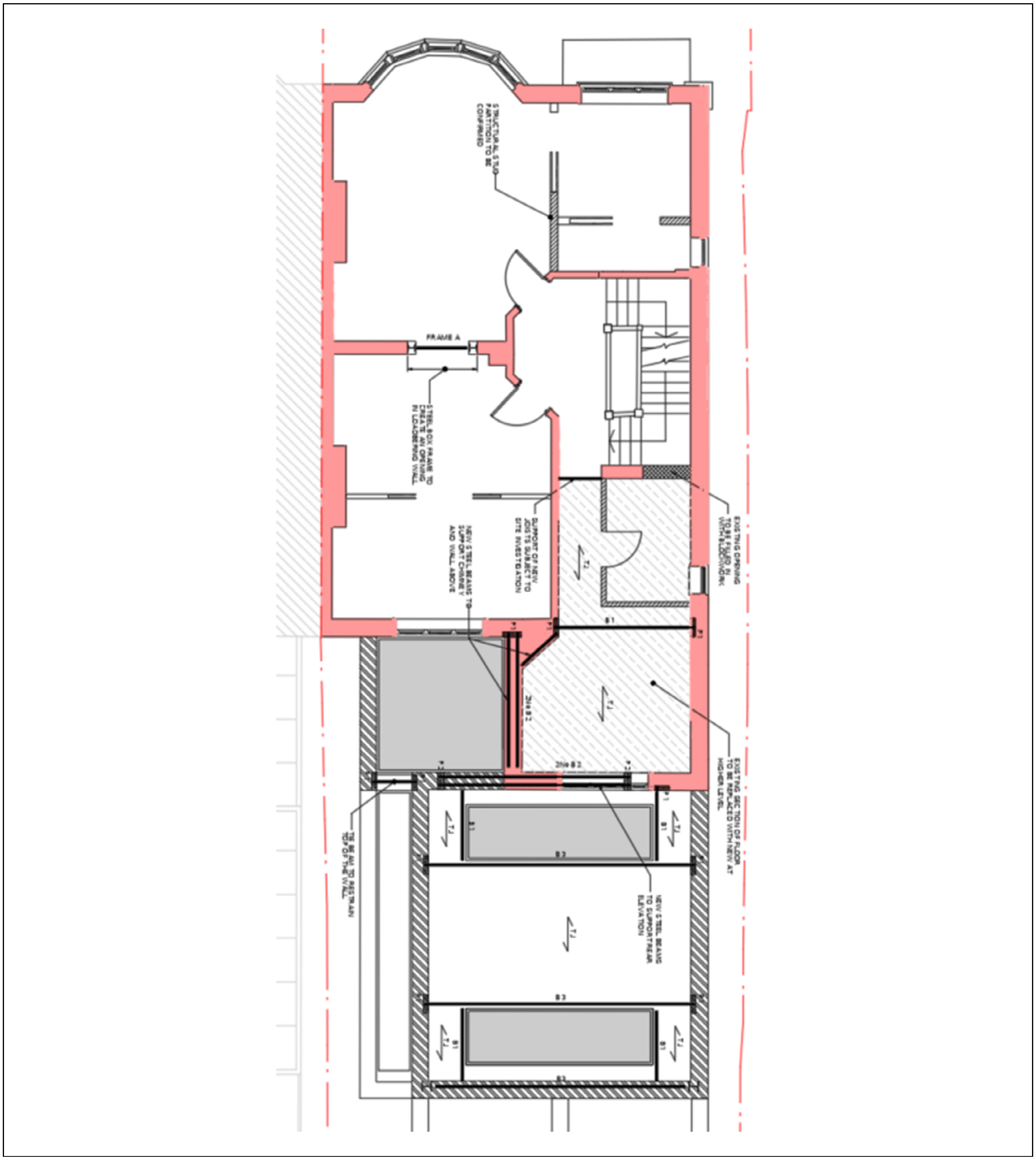
Limiting deflection $\delta_{lim} = L_{s1} / 360$ = **3.9** mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **0.035** mm

PASS - Maximum deflection does not exceed deflection limit

MBP	Michael Barclay Partnership consulting engineers	Job Title	Job Number	Sheet Number	Revision
		25 OAKHILL AVENUE, LONDON	8536	5.1	P1
	1 Lancaster Place WC2E 7ED	Calculation/sketch Title	Date	Author	Checked
	T 020 7240 1191 **F** 020 7240 2241	SECTION 5 FIRST FLOOR STRUCTURE	OCT 2022	AZ	TH
	E london@mbp-uk.com				

GEOMETRY



FIRST FLOOR PLAN

DEAD LOAD

FLAT ROOF			
Flat roof finishes		0.50	
Waterproofing		0.20	
Insulation		0.10	
Timber joists		0.25	
Ceiling and services		0.25	
	TOTAL	**1.30**	**kN/m²**
FIRST FLOOR			
Floor finishes		0.25	
Boarding		0.15	
Timber joists		0.25	
Ceiling and services		0.25	
	TOTAL	**0.90**	**kN/m²**
Solid brick wall		**4.30**	**kN/m²**

IMPOSED LOAD

Imposed Load	**2.50**	**kN/m²**

TYPICAL TIMBER JOISTS

Max. span 3.3m

	Dead load (excluding self weight)	Imposed load
Loading	0.9 kN/m^2	5.5 kN/m^2

Please refer to Section 4 for Timber Joists calculation.

TYPICAL STEEL BEAM B1

Max. span 2.9m

Load attracted from 6.5m/2 = 3.25m

	Dead load	Imposed load
Loading	1.15 x 3.25 = 3.74 kN/m	2.50 x 3.25 = 8.12 kN/m

Please refer to Section 4 for similar beam B1 callculation.

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2No STEEL BEAM B2

Max. span 4.0m

Load attracted from timber floor 3.2m/2 = 1.6m

Load attracted from Flat Roof 1.9m/2 =0.8m

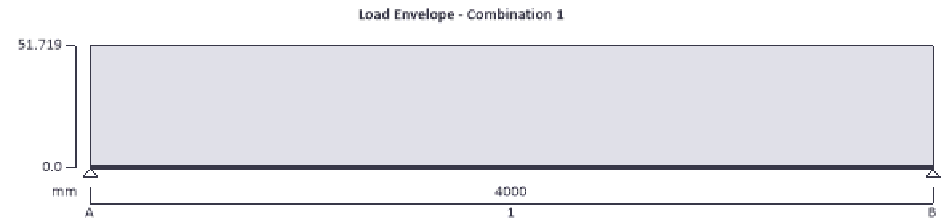
	Dead load	Imposed load
First Floor	0.9 x 1.6 = 1.44 kN/m	2.50 x 1.6 = 4.00 kN/m
Flat Roof	1.3 x 0.9 = 1.20 kN/m	2.50 x 0.9 = 2.25 kN/m
	TOTAL 2.64 kN/m	**6.25 kN/m**

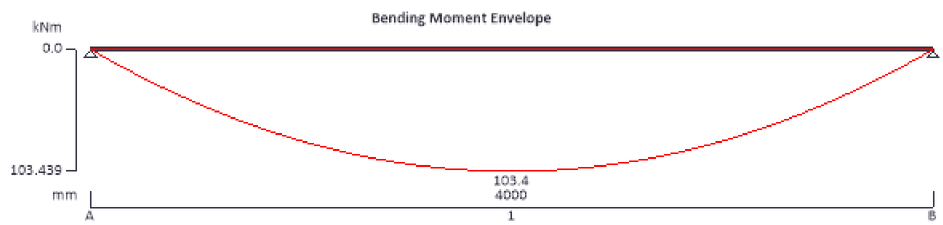
Solid brick wall 4.30 x 6.5m = **28.00 kN/m**

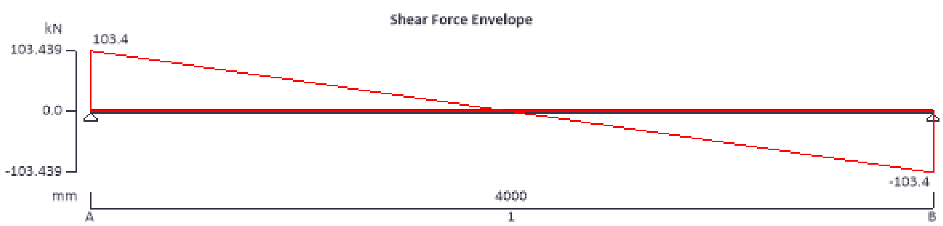
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14







Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam × 1
	Permanent full UDL 30.64 kN/m
	Variable full UDL 6.25 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
		Permanent × 1.35

		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

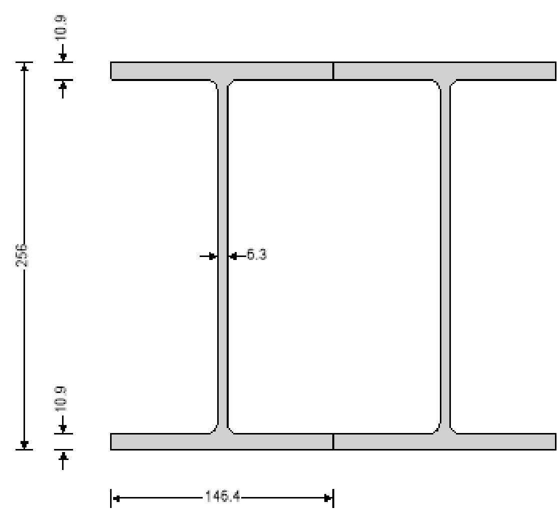
Maximum moment	M_{max} = **103.4** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **103.4** kN	V_{min} = **-103.4** kN
Deflection	δ_{max} = **0.9** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **103.4** kN	R_{A_min} = **103.4** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **62.7** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **12.5** kN	
Maximum reaction at support B	R_{B_max} = **103.4** kN	R_{B_min} = **103.4** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **62.7** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **12.5** kN	

Section details

Section type	**2 x UB 254x146x37 (BS4-1)**
Steel grade	**S275**

EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element	$t = \max(t_f, t_w)$ = **10.9** mm
Nominal yield strength	f_y = **275** N/mm^2
Nominal ultimate tensile strength	f_u = **410** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Resistance of tensile members to fracture	γ_{M2} = **1.10**

Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis	K_y = **1.000**

Effective length factor in minor axis K_z = **1.000**

Effective length factor for torsion $K_{LT.A}$ = **1.000**

$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.92**

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section $c = d$ = **219** mm

$c / t_w = 37.6 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2$ = **62.5** mm

$c / t_f = 6.2 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f$ = **234.2** mm

Shear area factor η = **1.000**

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **103.4** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1759** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = N \times A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **558.7** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **103.4** kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = N \times W_{pl.y} \times f_y / \gamma_{M0}$ = **265.8** kNm

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 360$ = **11.1** mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **0.896** mm

PASS - Maximum deflection does not exceed deflection limit

TYPICAL STEEL BEAM B3

Max. span 5.6m

Load attracted from 4.6m/2 = 2.3m

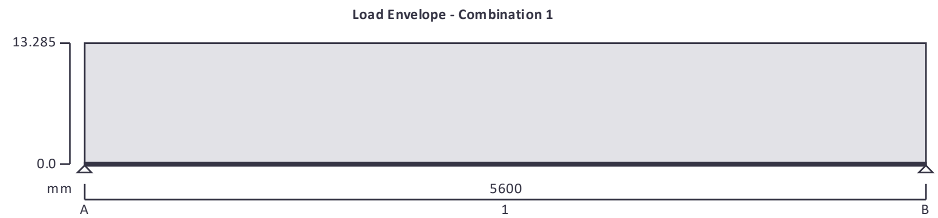
	Dead load	Imposed load
Loading	1.30 x 2.30 = 3.00 kN/m	2.50 x 2.30 = 5.75 kN/m

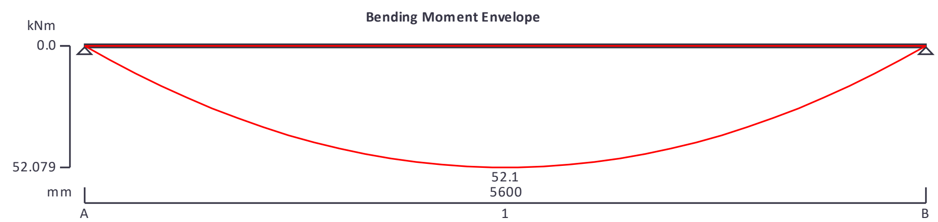
Tekla Tedds	Project			Job Ref.	
	25 OAKHILL AVENUE			8536	
	Section			Sheet no./rev.	
	FIRST FLOOR STRUCTURE			5. 9	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	AZ	21/10/2022			TH	21/10/2022

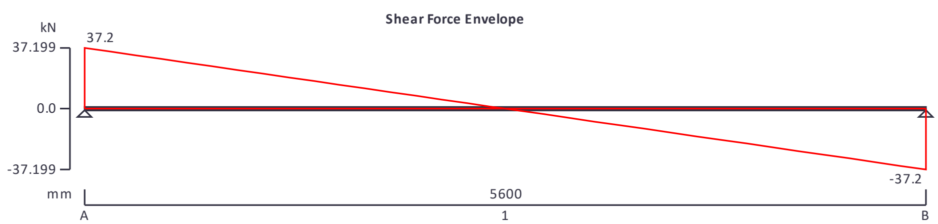
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14







Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam × 1
	Permanent full UDL 3 kN/m
	Variable full UDL 5.75 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
		Permanent × 1.35

		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

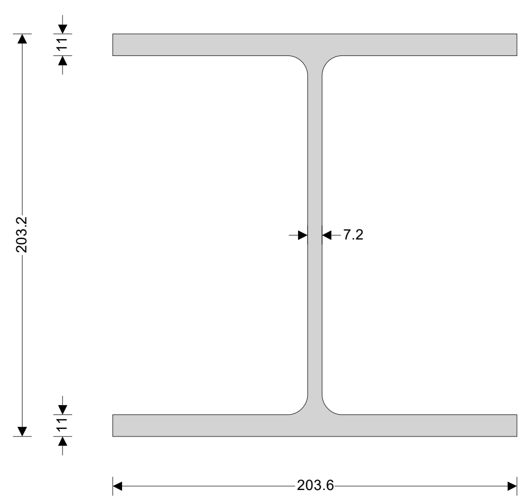
Maximum moment	M_{max} = **52.1** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **37.2** kN	V_{min} = **-37.2** kN
Deflection	δ_{max} = **7.7** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **37.2** kN	R_{A_min} = **37.2** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **9.7** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **16.1** kN	
Maximum reaction at support B	R_{B_max} = **37.2** kN	R_{B_min} = **37.2** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **9.7** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **16.1** kN	

Section details

Section type	**UC 203x203x46 (BS4-1)**
Steel grade	**S275**

EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element	$t = \max(t_f, t_w)$ = **11.0** mm
Nominal yield strength	f_y = **275** N/mm^2
Nominal ultimate tensile strength	f_u = **410** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Resistance of tensile members to fracture	γ_{M2} = **1.10**

Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis	K_y = **1.000**

Effective length factor in minor axis K_z = **1.000**

Effective length factor for torsion $K_{LT.A}$ = **1.000**

$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.92**

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section $c = d$ = **160.8** mm

$c / t_w = 24.2 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2$ = **88** mm

$c / t_f = 8.7 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f$ = **181.2** mm

Shear area factor η = **1.000**

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **37.2** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1698** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **269.5** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **52.1** kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **136.8** kNm

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 360$ = **15.6** mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **7.676** mm

PASS - Maximum deflection does not exceed deflection limit

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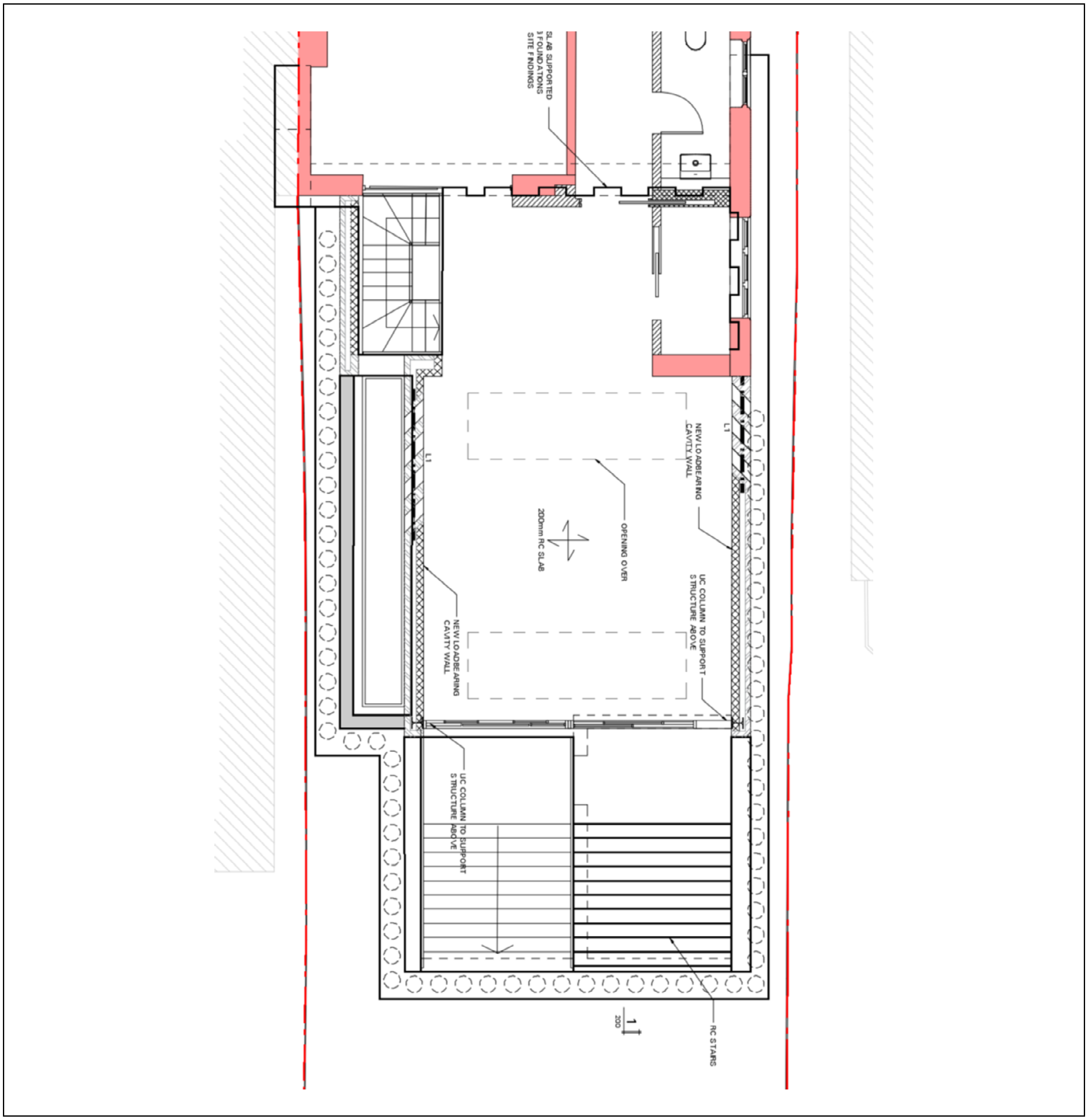
Job Title	Job Number	Sheet Number	Revision
25 OAKHILL AVENUE, LONDON	8536	6.1	P1
Calculation/sketch Title	**Date**	**Author**	**Checked**
SECTION 6 GROUND FLOOR SLAB STRUCTURE	OCT 2022	AZ	TH

GEOMETRY



GROUND FLOOR PLAN

DEAD LOAD

Floor finishes	0.50	
100mm Screed	2.20	
250mm RC Slab	6.00	
Ceiling and services	0.50	
TOTAL	**9.20**	**kN/m^2**

New cavity wall	**4.88**	**kN/m^2**

IMPOSED LOAD

Imposed Load (Including Partitions)	**2.50**	**kN/m^2**

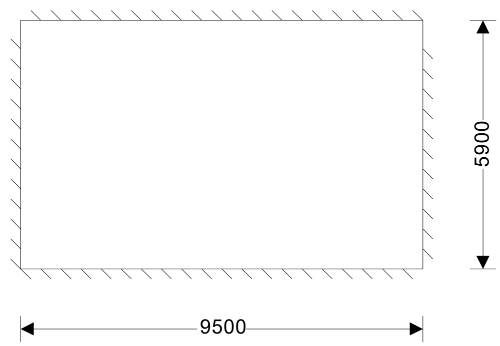
RC SLAB DESIGN

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

Tedds calculation version 1.0.21

Design summary

Description	Unit	Provided	Required	Utilisation	Result
Short span					
Reinf. at midspan	mm^2/m	1005	311	0.309	PASS
Bar spacing at midspan	mm	200	300	0.667	PASS
Shear at discont. supp	kN/m	103.9	23.8	0.229	PASS
Deflection ratio		27.83	39.31	0.708	PASS
Long span					
Reinf. at midspan	mm^2/m	1005	261	0.260	PASS
Bar spacing at midspan	mm	200	300	0.667	PASS
Shear at discont. supp	kN/m	99.0	23.8	0.240	PASS
Cover					
Min cover bottom	mm	30	26	0.867	PASS



Slab definition

Slab reference name **250mm RC Slab**

Type of slab **Two way spanning with restrained edges**

Overall slab depth h = **250** mm

Shorter effective span of panel l_x = **5900** mm

Longer effective span of panel l_y = **9500** mm

Support conditions **Four edges discontinuous**

Bottom outer layer of reinforcement **Short span direction**

Loading

Characteristic permanent action $G_k = \mathbf{3.2}$ kN/m^2

Characteristic variable action $Q_k = \mathbf{2.5}$ kN/m^2

Partial factor for permanent action $\gamma_G = \mathbf{1.35}$

Partial factor for variable action $\gamma_Q = \mathbf{1.50}$

Quasi-permanent value of variable action $\psi_2 = \mathbf{0.30}$

Design ultimate load $q = \gamma_G \times G_k + \gamma_Q \times Q_k = \mathbf{8.1}$ kN/m^2

Quasi-permanent load $q_{SLS} = 1.0 \times G_k + \psi_2 \times Q_k = \mathbf{4.0}$ kN/m^2

Concrete properties

Concrete strength class	C25/30
Characteristic cylinder strength	f_{ck} = **25** 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} = **14.2** N/mm^2
Mean axial tensile strength (Table 3.1)	$f_{ctm} = 0.30\ \text{N/mm}^2 \times (f_{ck} / 1\ \text{N/mm}^2)^{2/3}$ = **2.6** 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 outer bottom reinforcement	c_{nom_b} = **30** mm
Fire resistance period to bottom of slab	R_{btm} = **60** min
Axia distance to bottom reinft (Table 5.8)	a_{fi_b} = **15** mm
Min. btm cover requirement with regard to bond	c_{min,b_b} = **16** mm
Reinforcement fabrication	**Not subject to QA system**
Cover allowance for deviation	Δc_{dev} = **10** mm
Min. required nominal cover to bottom reinft	$c_{nom\_b\_min}$ = **26.0** mm

PASS - There is sufficient cover to the bottom reinforcement

Reinforcement design at midspan in short span direction (cl.6.1)

Bending moment coefficient	β_{sx_p} = **0.0968**
Design bending moment	$M_{x\_p} = \beta_{sx\_p} \times q \times l_x^2$ = **27.2** kNm/m
Reinforcement provided	16 mm dia. bars at 200 mm centres
Area provided	A_{sx_p} = 1005 mm^2/m
Effective depth to tension reinforcement	$d_{x\_p} = h - c_{nom\_b} - \phi_{x\_p} / 2$ = **212.0** mm
K factor	$K = M_{x\_p} / (b \times d_{x\_p}^2 \times f_{ck})$ = **0.024**
Redistribution ratio	δ = 1.0
K' factor	$K' = 0.598 \times \delta - 0.18 \times \delta^2 - 0.21$ = **0.208**

K < K' - Compression reinforcement is not required

Lever arm	$z = \min(0.95 \times d_{x\_p}, d_{x\_p}/2 \times (1 + \sqrt{(1 - 3.53 \times K)}))$ = **201.4** mm
Area of reinforcement required for bending	$A_{sx\_p\_m} = M_{x\_p} / (f_{yd} \times z)$ = **311** mm^2/m
Minimum area of reinforcement required	$A_{sx\_p\_min} = \max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{x\_p}, 0.0013 \times b \times d_{x\_p})$ = **283** mm^2/m
Area of reinforcement required	$A_{sx\_p\_req} = \max(A_{sx\_p\_m}, A_{sx\_p\_min})$ = **311** mm^2/m

PASS - Area of reinforcement provided exceeds area required

Check reinforcement spacing

Reinforcement service stress	$\sigma_{sx\_p} = (f_{yk} / \gamma_S) \times \min((A_{sx\_p\_m}/A_{sx\_p}), 1.0) \times q_{SLS} / q$ = **65.8** N/mm^2
Maximum allowable spacing (Table 7.3N)	$s_{max\_x\_p}$ = **300** mm
Actual bar spacing	s_{x_p} = **200** mm

PASS - The reinforcement spacing is acceptable

Reinforcement design at midspan in long span direction (cl.6.1)

Bending moment coefficient β_{sy_p} = **0.0560**

Design bending moment $M_{y\_p} = \beta_{sy\_p} \times q \times l_x^2$ = **15.7** kNm/m

Reinforcement provided 16 mm dia. bars at 200 mm centres

Area provided A_{sy_p} = 1005 mm²/m

Effective depth to tension reinforcement $d_{y\_p} = h - c_{nom\_b} - \phi_{x\_p} - \phi_{y\_p} / 2$ = **196.0** mm

K factor $K = M_{y\_p} / (b \times d_{y\_p}^2 \times f_{ck})$ = **0.016**

Redistribution ratio δ = 1.0

K' factor $K' = 0.598 \times \delta - 0.18 \times \delta^2 - 0.21$ = **0.208**

K < K' - Compression reinforcement is not required

Lever arm $z = \min(0.95 \times d_{y\_p}, d_{y\_p}/2 \times (1 + \sqrt{(1 - 3.53 \times K)}))$ = **186.2** mm

Area of reinforcement required for bending $A_{sy\_p\_m} = M_{y\_p} / (f_{yd} \times z)$ = **194** mm²/m

Minimum area of reinforcement required $A_{sy\_p\_min} = \max(0.26 \times (f_{ctm}/f_{yk}) \times b \times d_{y\_p}, 0.0013 \times b \times d_{y\_p})$ = **261** mm²/m

Area of reinforcement required $A_{sy\_p\_req} = \max(A_{sy\_p\_m}, A_{sy\_p\_min})$ = **261** mm²/m

PASS - Area of reinforcement provided exceeds area required

Check reinforcement spacing

Reinforcement service stress $\sigma_{sy\_p} = (f_{yk} / \gamma_S) \times \min((A_{sy\_p\_m}/A_{sy\_p}), 1.0) \times q_{SLS} / q$ = **41.1** N/mm²

Maximum allowable spacing (Table 7.3N) $s_{max\_y\_p}$ = **300** mm

Actual bar spacing s_{y_p} = **200** mm

PASS - The reinforcement spacing is acceptable

Shear capacity check at short span discontinuous support

Shear force $V_{x\_d} = q \times l_x / 2$ = **23.8** kN/m

Reinforcement provided **8 mm dia. bars at 200 mm centres**

Area provided A_{sx_d} = **251** mm²/m

Effective depth $d_{x\_d} = h - c_{nom\_b} - \phi_{x\_d} / 2$ = **216.0** mm

Effective depth factor $k = \min(2.0, 1 + (200 \text{ mm} / d_{x\_d})^{0.5})$ = **1.962**

Reinforcement ratio $\rho_l = \min(0.02, A_{sx\_d} / (b \times d_{x\_d}))$ = **0.0012**

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

V_{Rd,c_min} = **103.9** kN/m

Shear resistance constant (cl. 6.2.2) $C_{Rd,c} = 0.18 \text{ N/mm}^2 / \gamma_C$ = **0.12** N/mm²

Shear resistance

$V_{Rd,c\_x\_d} = \max(V_{Rd,c\_min}, C_{Rd,c} \times k \times (100 \times \rho_l \times (f_{ck}/1 \text{ N/mm}^2))^{0.333} \times b \times d_{x\_d})$ = **103.9** kN/m

PASS - Shear capacity is adequate (0.229)

Shear capacity check at long span discontinuous support

Shear force $V_{y\_d} = q \times l_x / 2$ = **23.8** kN/m

Reinforcement provided **8 mm dia. bars at 200 mm centres**

Area provided A_{sy_d} = **251** mm²/m

Effective depth $d_{y\_d} = h - c_{nom\_b} - \phi_{x\_p} - \phi_{y\_d} / 2$ = **200.0** mm

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

Reinforcement ratio $\rho_l = \min(0.02, A_{sy\_d} / (b \times d_{y\_d}))$ = **0.0013**

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

V_{Rd,c_min} = **99.0** kN/m

Shear resistance constant (cl. 6.2.2) $C_{Rd,c} = 0.18 \text{ N/mm}^2 / \gamma_C$ = **0.12** N/mm²

Shear resistance

V_{Rd,c_y_d} = max(V_{Rd,c_min}, $C_{Rd,c}$ × k × (100 × ρ_l × (f_{ck}/1 N/mm^2))$^{0.333}$ × b × d_{y_d}) = **99.0** kN/m

PASS - Shear capacity is adequate (0.240)

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

Reference reinforcement ratio ρ_0 = (f_{ck} / 1 N/mm^2)$^{0.5}$ / 1000 = **0.0050**

Required tension reinforcement ratio ρ = max(0.0035, $A_{sx\_p\_req}$ / (b × d_{x_p})) = **0.0035**

Required compression reinforcement ratio ρ' = $A_{scx\_p\_req}$ / (b × d_{x_p}) = **0.0000**

Stuctural system factor (Table 7.4N) K_δ = **1.0**

Basic limit span-to-depth ratio (Exp. 7.16)

$ratio_{lim\_x\_bas}$ = K_δ × [11 +1.5×(f_{ck}/1 N/mm^2)$^{0.5}$×ρ_0/ρ + 3.2×(f_{ck}/1 N/mm^2)$^{0.5}$×(ρ_0/ρ -1)$^{1.5}$] = **26.20**

Mod span-to-depth ratio limit

$ratio_{lim\_x}$ = min(40 × K_δ, min(1.5, (500 N/mm^2/ f_{yk}) × (A_{sx_p} / $A_{sx\_p\_m}$)) × $ratio_{lim\_x\_bas}$) = **39.31**

Actual span-to-eff. depth ratio $ratio_{act\_x}$ = l_x / d_{x_p} = **27.83**

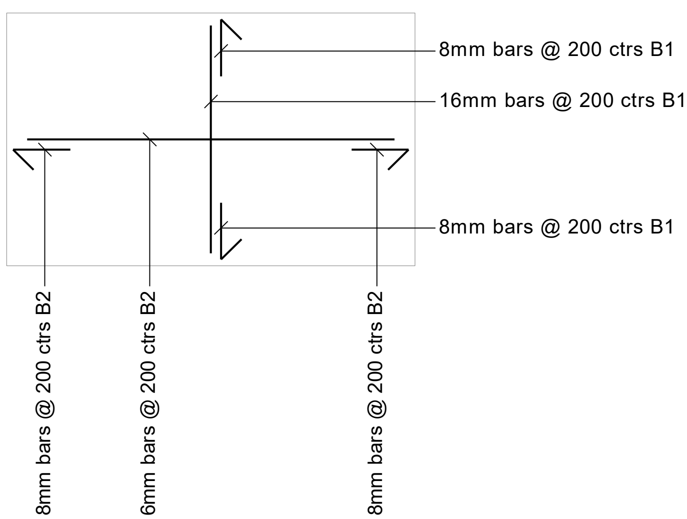
PASS - Actual span-to-effective depth ratio is acceptable

Reinforcement summary

Midspan in short span direction	**16 mm dia. bars at 200 mm centres B1**
Midspan in long span direction	**16 mm dia. bars at 200 mm centres B2**
Discontinuous support in short span direction	**8 mm dia. bars at 200 mm centres B1**
Discontinuous support in long span direction	**8 mm dia. bars at 200 mm centres B2**

Reinforcement sketch

The following sketch is indicative only. Note that additional reinforcement may be required in accordance with clauses 9.2.1.2, 9.2.1.4 and 9.2.1.5 of EN 1992-1-1:2004 to meet detailing rules.



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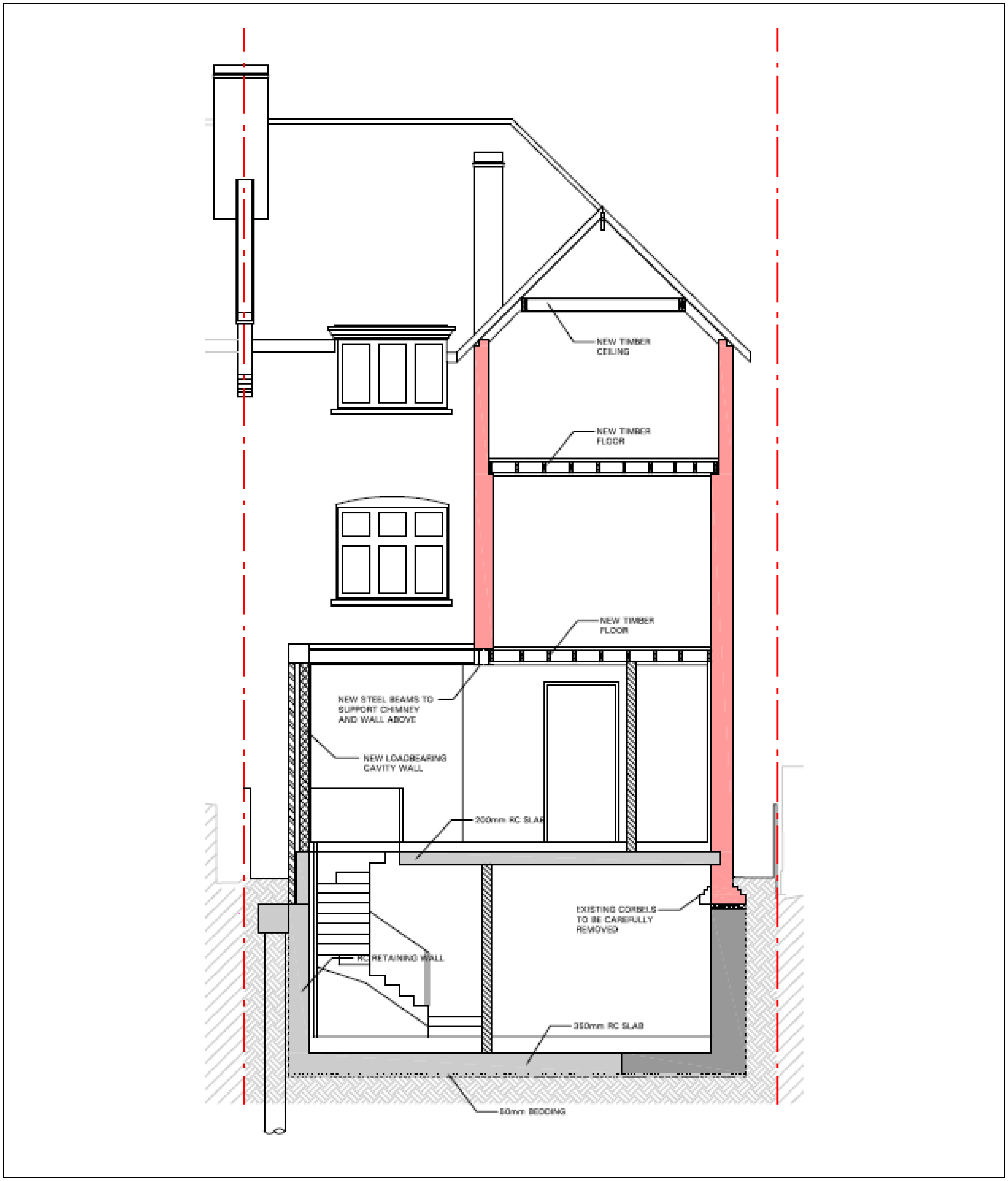
Job Title	Job Number	Sheet Number	Revision
25 OAKHILL AVENUE, LONDON	8536	7.1	P1
Calculation/sketch Title	**Date**	**Author**	**Checked**
SECTION 7 RETAINING WALL CALCULATION	OCT 2022	AZ	TH

GEOMETRY



SECTION THROUGT RETAINING WALL

575mm THICK RETAINING WALL

DEAD LOAD

VERTICAL LOAD:	
Roof	1.05 kN/m^2 x 1.1m = 2.15 kN/m
Second Floor	0.90 kN/m^2 x 1.9m = 1.71 kN/m
Flat Roof	1.30 kN/m^2 x 1.9m = 2.50 kN/m
Ground Floor	6.80 kN/m^2 x 3.4m = 23.12 kN/m
TOTAL:	**29.50 kN/m**
Existing Wall	6.31 kN/m^2 x 10m = **63.10 kN/m**
GROUND FORCE:	(trapeziondal force) height 4.4m, $\gamma =$ 18.5 kN/m^3

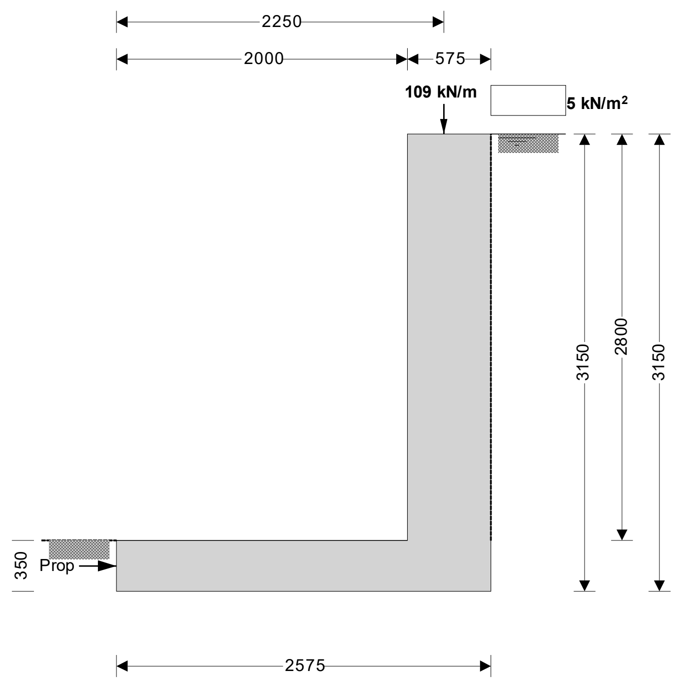
IMPOSED LOAD

VERTICAL LOAD:	
Roof	0.6 kN/m^2 x 1.1m = 0.66 kN/m
Second Floor	2.5 kN/m^2 x 1.9m = 4.75 kN/m
Flat Roof	1.5 kN/m^2 x 1.9m = 2.85 kN/m
Ground Floor	2.5 kN/m^2 x 3.4m = 8.50 kN/m
TOTAL:	**16.76 kN/m**
SURCHARGE:	5 kN/m^2
WATER:	Full high water level 3.0m above slab

575 mm thick Lining wall to be propped at the bottom by RC slab.

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.08



Wall details

Retaining wall type	**Cantilever propped at base**
Height of retaining wall stem	h_{stem} = **2800** mm
Thickness of wall stem	t_{wall} = **575** mm
Length of toe	l_{toe} = **2000** mm
Length of heel	l_{heel} = **0** mm
Overall length of base	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **2575** mm
Thickness of base	t_{base} = **350** mm
Depth of downstand	d_{ds} = **0** mm
Position of downstand	l_{ds} = **500** mm
Thickness of downstand	t_{ds} = **350** mm
Height of retaining wall	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **3150** mm
Depth of cover in front of wall	d_{cover} = **0** mm
Depth of unplanned excavation	d_{exc} = **0** mm
Height of ground water behind wall	h_{water} = **3150** mm
Height of saturated fill above base	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0\text{ mm})$ = **2800** mm
Density of wall construction	γ_{wall} = **24.0** kN/m^3
Density of base construction	γ_{base} = **24.0** kN/m^3
Angle of rear face of wall	α = **90.0** deg
Angle of soil surface behind wall	β = **0.0** deg
Effective height at virtual back of wall	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **3150** mm

Retained material details

Mobilisation factor — M = **1.5**

Moist density of retained material — γ_m = **18.5** kN/m^3

Saturated density of retained material — γ_s = **21.5** kN/m^3

Design shear strength — ϕ' = **34.0** deg

Angle of wall friction — δ = **0.7** deg

Base material details

Stiff clay

Moist density — γ_{mb} = **18.5** kN/m^3

Design shear strength — ϕ'_b = **34.0** deg

Design base friction — δ_b = **0.7** deg

Allowable bearing pressure — $P_{bearing}$ = **300** kN/m^2

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{}(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))]^2) = \mathbf{0.281}$$

Passive pressure coefficient for base material

$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{}(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))]^2) = \mathbf{3.620}$$

At-rest pressure

At-rest pressure for retained material — $K_0 = 1 – \sin(\phi')$ = **0.441**

Loading details

Surcharge load on plan — Surcharge = **5.0** kN/m^2

Applied vertical dead load on wall — W_{dead} = **92.6** kN/m

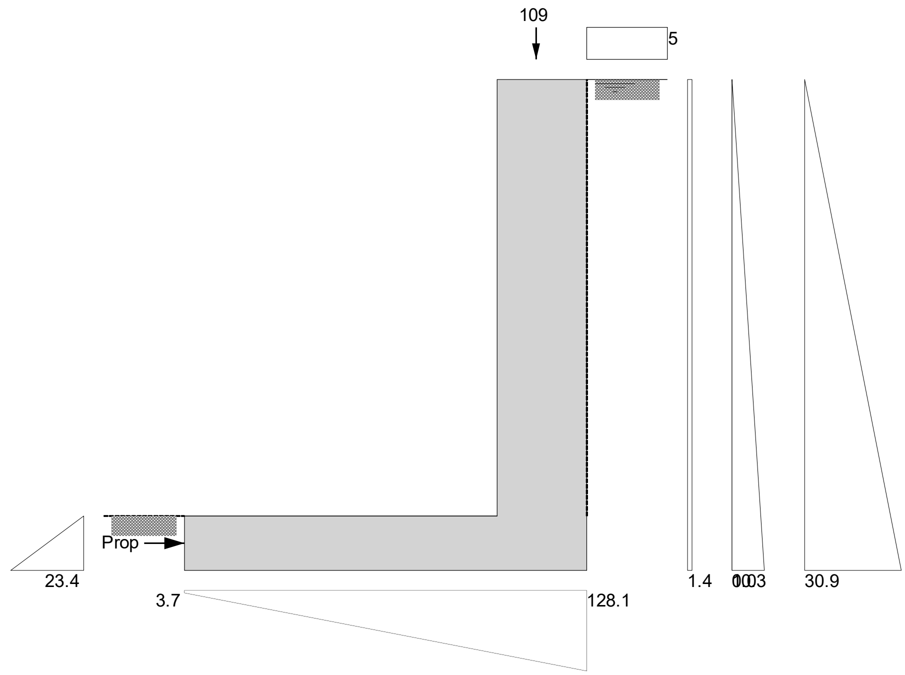
Applied vertical live load on wall — W_{live} = **16.8** kN/m

Position of applied vertical load on wall — l_{load} = **2250** mm

Applied horizontal dead load on wall — F_{dead} = **0.0** kN/m

Applied horizontal live load on wall — F_{live} = **0.0** kN/m

Height of applied horizontal load on wall — h_{load} = **0** mm



Loads shown in kN/m, pressures shown in kN/m^2

Vertical forces on wall

Wall stem	$w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **38.6** kN/m
Wall base	$w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **21.6** kN/m
Applied vertical load	$W_v = W_{dead} + W_{live}$ = **109.4** kN/m
Total vertical load	$W_{total} = w_{wall} + w_{base} + W_v$ = **169.6** kN/m

Horizontal forces on wall

Surcharge	$F_{sur} = K_a \times \cos(90 - \alpha + \delta) \times \text{Surcharge} \times h_{eff}$ = **4.4** kN/m
Moist backfill below water table	$F_{m\_b} = K_a \times \cos(90 - \alpha + \delta) \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **0** kN/m
Saturated backfill	$F_s = 0.5 \times K_a \times \cos(90 - \alpha + \delta) \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **16.3** kN/m
Water	$F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water}$ = **48.7** kN/m
Total horizontal load	$F_{total} = F_{sur} + F_{m\_b} + F_s + F_{water}$ = **69.4** kN/m

Calculate propping force

Passive resistance of soil in front of wall	$F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **4.1** kN/m
Propping force	$F_{prop} = \max(F_{total} - F_p - (W_{total} - W_{live}) \times \tan(\delta_b), 0 \text{ kN/m})$
	F_{prop} = **63.4** kN/m

Overturning moments

Surcharge	$M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **7** kNm/m
Moist backfill below water table	$M_{m\_b} = F_{m\_b} \times (h_{water} - 2 \times d_{ds}) / 2$ = **0** kNm/m
Saturated backfill	$M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3$ = **17.1** kNm/m
Water	$M_{water} = F_{water} \times (h_{water} - 3 \times d_{ds}) / 3$ = **51.1** kNm/m
Total overturning moment	$M_{ot} = M_{sur} + M_{m\_b} + M_s + M_{water}$ = **75.2** kNm/m

Restoring moments

Wall stem	$M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **88.4** kNm/m

Wall base $M_{base} = w_{base} \times l_{base} / 2 =$ **27.8** kNm/m

Design vertical dead load $M_{dead} = W_{dead} \times l_{load} =$ **208.4** kNm/m

Total restoring moment $M_{rest} = M_{wall} + M_{base} + M_{dead} =$ **324.6** kNm/m

Check bearing pressure

Design vertical live load $M_{live} = W_{live} \times l_{load} =$ **37.7** kNm/m

Total moment for bearing $M_{total} = M_{rest} - M_{ot} + M_{live} =$ **287.1** kNm/m

Total vertical reaction $R = W_{total} =$ **169.6** kN/m

Distance to reaction $x_{bar} = M_{total} / R =$ **1693** mm

Eccentricity of reaction $e = abs((l_{base} / 2) - x_{bar}) =$ **405** mm

Reaction acts within middle third of base

Bearing pressure at toe $p_{toe} = (R / l_{base}) - (6 \times R \times e / l_{base}^2) =$ **3.7** kN/m^2

Bearing pressure at heel $p_{heel} = (R / l_{base}) + (6 \times R \times e / l_{base}^2) =$ **128.1** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.08

Ultimate limit state load factors

Dead load factor γ_{f_d} = **1.4**

Live load factor γ_{f_l} = **1.6**

Earth and water pressure factor γ_{f_e} = **1.4**

Factored vertical forces on wall

Wall stem $w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall}$ = **54.1** kN/m

Wall base $w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base}$ = **30.3** kN/m

Applied vertical load $W_{v\_f} = \gamma_{f\_d} \times W_{dead} + \gamma_{f\_l} \times W_{live}$ = **156.5** kN/m

Total vertical load $W_{total\_f} = w_{wall\_f} + w_{base\_f} + W_{v\_f}$ = **240.8** kN/m

Factored horizontal at-rest forces on wall

Surcharge $F_{sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times h_{eff}$ = **11.1** kN/m

Moist backfill below water table $F_{m\_b\_f} = \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **0** kN/m

Saturated backfill $F_{s\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **35.8** kN/m

Water $F_{water\_f} = \gamma_{f\_e} \times 0.5 \times h_{water}^2 \times \gamma_{water}$ = **68.1** kN/m

Total horizontal load $F_{total\_f} = F_{sur\_f} + F_{m\_b\_f} + F_{s\_f} + F_{water\_f}$ = **115** kN/m

Calculate propping force

Passive resistance of soil in front of wall $F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **5.7** kN/m

Propping force $F_{prop\_f} = \max(F_{total\_f} - F_{p\_f} - (W_{total\_f} - \gamma_{f\_l} \times W_{live}) \times \tan(\delta_b), 0 \text{ kN/m})$

F_{prop_f} = **106.7** kN/m

Factored overturning moments

Surcharge $M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **17.5** kNm/m

Moist backfill below water table $M_{m\_b\_f} = F_{m\_b\_f} \times (h_{water} - 2 \times d_{ds}) / 2$ = **0** kNm/m

Saturated backfill $M_{s\_f} = F_{s\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **37.6** kNm/m

Water $M_{water\_f} = F_{water\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **71.5** kNm/m

Total overturning moment $M_{ot\_f} = M_{sur\_f} + M_{m\_b\_f} + M_{s\_f} + M_{water\_f}$ = **126.6** kNm/m

Restoring moments

Wall stem $M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2)$ = **123.7** kNm/m

Wall base $M_{base\_f} = w_{base\_f} \times l_{base} / 2$ = **39** kNm/m

Design vertical load $M_{v\_f} = W_{v\_f} \times l_{load}$ = **352** kNm/m

Total restoring moment $M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{v\_f}$ = **514.8** kNm/m

Factored bearing pressure

Total moment for bearing $M_{total\_f} = M_{rest\_f} - M_{ot\_f}$ = **388.1** kNm/m

Total vertical reaction $R_f = W_{total\_f}$ = **240.8** kN/m

Distance to reaction $x_{bar\_f} = M_{total\_f} / R_f$ = **1612** mm

Eccentricity of reaction $e_f = \text{abs}((l_{base} / 2) - x_{bar\_f})$ = **324** mm

Reaction acts within middle third of base

Bearing pressure at toe $p_{toe\_f} = (R_f / l_{base}) - (6 \times R_f \times e_f / l_{base}^2)$ = **22.9** kN/m²

Bearing pressure at heel $p_{heel\_f} = (R_f / l_{base}) + (6 \times R_f \times e_f / l_{base}^2)$ = **164.2** kN/m²

Rate of change of base reaction rate $= (p_{toe\_f} - p_{heel\_f}) / l_{base}$ = **-54.87** kN/m²/m

Bearing pressure at stem / toe	$p_{stem\_toe\_f} = \max(p_{heel\_f} + (rate \times (l_{heel} + t_{wall})), 0\ kN/m^2) = \mathbf{132.6}\ kN/m^2$
Bearing pressure at mid stem	$p_{stem\_mid\_f} = \max(p_{heel\_f} + (rate \times (l_{heel} + t_{wall} / 2)), 0\ kN/m^2) = \mathbf{148.4}\ kN/m^2$
Bearing pressure at stem / heel	$p_{stem\_heel\_f} = \max(p_{heel\_f} + (rate \times l_{heel}), 0\ kN/m^2) = \mathbf{164.2}\ kN/m^2$

<u>Design of reinforced concrete retaining wall toe (BS 8002:1994)</u>

Material properties

Characteristic strength of concrete	$f_{cu} = \mathbf{35}\ N/mm^2$
Characteristic strength of reinforcement	$f_y = \mathbf{500}\ N/mm^2$

Base details

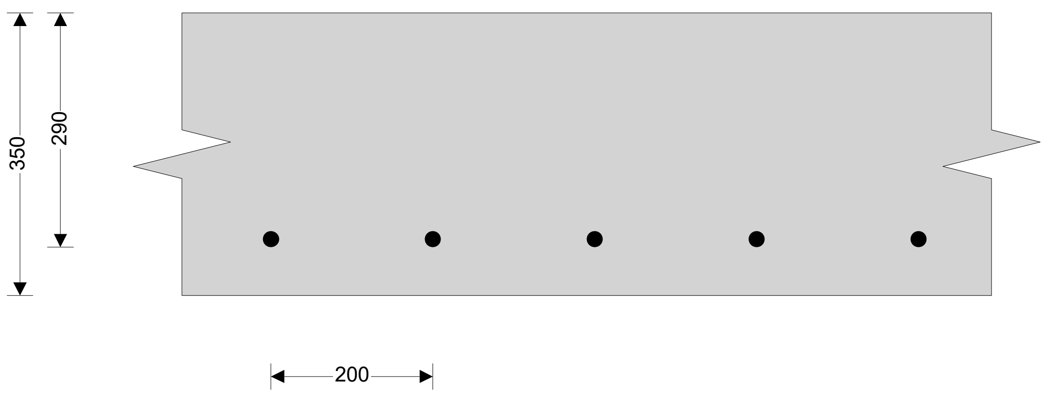
Minimum area of reinforcement	$k = \mathbf{0.13}\ \%$
Cover to reinforcement in toe	$c_{toe} = \mathbf{50}\ mm$

Calculate shear for toe design

Shear from bearing pressure	$V_{toe\_bear} = (p_{toe\_f} + p_{stem\_toe\_f}) \times l_{toe} / 2 = \mathbf{155.5}\ kN/m$
Shear from weight of base	$V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base} = \mathbf{23.5}\ kN/m$
Total shear for toe design	$V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base} = \mathbf{132}\ kN/m$

Calculate moment for toe design

Moment from bearing pressure	$M_{toe\_bear} = (2 \times p_{toe\_f} + p_{stem\_mid\_f}) \times (l_{toe} + t_{wall} / 2)^2 / 6 = \mathbf{169.3}\ kNm/m$
Moment from weight of base	$M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2) = \mathbf{30.8}\ kNm/m$
Total moment for toe design	$M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base} = \mathbf{138.6}\ kNm/m$



Check toe in bending

Width of toe	$b = \mathbf{1000}\ mm/m$
Depth of reinforcement	$d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2) = \mathbf{290.0}\ mm$
Constant	$K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu}) = \mathbf{0.047}$
	Compression reinforcement is not required
Lever arm	$z_{toe} = \min(0.5 + \sqrt{(0.25 - (\min(K_{toe}, 0.225) / 0.9))}, 0.95) \times d_{toe}$
	$z_{toe} = \mathbf{274}\ mm$
Area of tension reinforcement required	$As_{toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe}) = \mathbf{1163}\ mm^2/m$
Minimum area of tension reinforcement	$As_{toe\_min} = k \times b \times t_{base} = \mathbf{455}\ mm^2/m$
Area of tension reinforcement required	$As_{toe\_req} = \mathrm{Max}(As_{toe\_des}, As_{toe\_min}) = \mathbf{1163}\ mm^2/m$
Reinforcement provided	**20 mm dia.bars @ 200 mm centres**
Area of reinforcement provided	$As_{toe\_prov} = \mathbf{1571}\ mm^2/m$
	PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress $v_{toe} = V_{toe} / (b \times d_{toe}) = \mathbf{0.455}$ N/mm²

Allowable shear stress $v_{adm} = \min(0.8 \times \sqrt{}(f_{cu} / 1\ \text{N/mm}^2), 5) \times 1\ \text{N/mm}^2 = \mathbf{4.733}$ N/mm²

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress $v_{c\_toe} = \mathbf{0.625}$ N/mm²

$v_{toe} < v_{c\_toe}$ - No shear reinforcement required

Design of reinforced concrete retaining wall stem (BS 8002:1994)

Material properties

Characteristic strength of concrete $f_{cu} = \mathbf{35}$ N/mm²

Characteristic strength of reinforcement $f_y = \mathbf{500}$ N/mm²

Wall details

Minimum area of reinforcement $k = \mathbf{0.13}$ %

Cover to reinforcement in stem $c_{stem} = \mathbf{50}$ mm

Cover to reinforcement in wall $c_{wall} = \mathbf{50}$ mm

Factored horizontal at-rest forces on stem

Surcharge $F_{s\_sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times (h_{eff} - t_{base} - d_{ds}) = \mathbf{9.9}$ kN/m

Moist backfill below water table $F_{s\_m\_b\_f} = \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat}) \times h_{sat} = \mathbf{0}$ kN/m

Saturated backfill $F_{s\_s\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{sat}^2 = \mathbf{28.3}$ kN/m

Water $F_{s\_water\_f} = 0.5 \times \gamma_{f\_e} \times \gamma_{water} \times h_{sat}^2 = \mathbf{53.8}$ kN/m

Calculate shear for stem design

Shear at base of stem $V_{stem} = F_{s\_sur\_f} + F_{s\_m\_b\_f} + F_{s\_s\_f} + F_{s\_water\_f} - F_{prop\_f} = \mathbf{-14.7}$ kN/m

Calculate moment for stem design

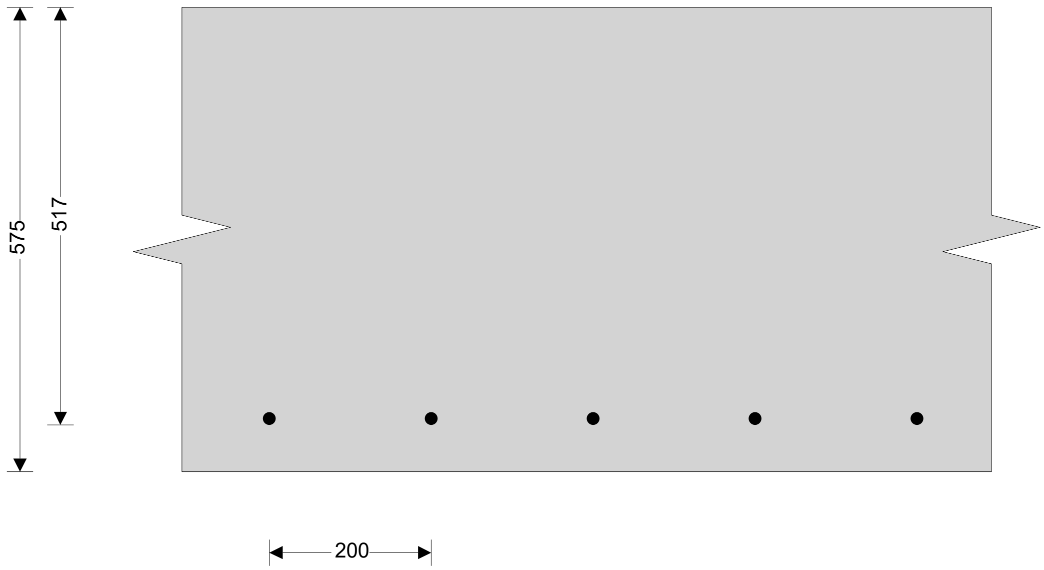
Surcharge $M_{s\_sur} = F_{s\_sur\_f} \times (h_{stem} + t_{base}) / 2 = \mathbf{15.6}$ kNm/m

Moist backfill below water table $M_{s\_m\_b} = F_{s\_m\_b\_f} \times h_{sat} / 2 = \mathbf{0}$ kNm/m

Saturated backfill $M_{s\_s} = F_{s\_s\_f} \times h_{sat} / 3 = \mathbf{26.4}$ kNm/m

Water $M_{s\_water} = F_{s\_water\_f} \times h_{sat} / 3 = \mathbf{50.2}$ kNm/m

Total moment for stem design $M_{stem} = M_{s\_sur} + M_{s\_m\_b} + M_{s\_s} + M_{s\_water} = \mathbf{92.2}$ kNm/m



Check wall stem in bending

Width of wall stem	b = **1000** mm/m
Depth of reinforcement	$d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2)$ = **517.0** mm
Constant	$K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu})$ = **0.010**

Compression reinforcement is not required

Lever arm	$z_{stem} = \min(0.5 + \sqrt{(0.25 - (\min(K_{stem}, 0.225) / 0.9))}, 0.95) \times d_{stem}$
	z_{stem} = **491** mm
Area of tension reinforcement required	$As_{\_stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem})$ = **432** mm^2/m
Minimum area of tension reinforcement	$As_{\_stem\_min} = k \times b \times t_{wall}$ = **748** mm^2/m
Area of tension reinforcement required	$As_{\_stem\_req} = \mathrm{Max}(As_{\_stem\_des}, As_{\_stem\_min})$ = **748** mm^2/m
Reinforcement provided	**16 mm dia.bars @ 200 mm centres**
Area of reinforcement provided	$As_{\_stem\_prov}$ = **1005** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress	$v_{stem} = V_{stem} / (b \times d_{stem})$ = **-0.028** N/mm^2
Allowable shear stress	$v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1\ \mathrm{N/mm^2})}, 5) \times 1\ \mathrm{N/mm^2}$ = **4.733** N/mm^2

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress	v_{c_stem} = **0.410** N/mm^2

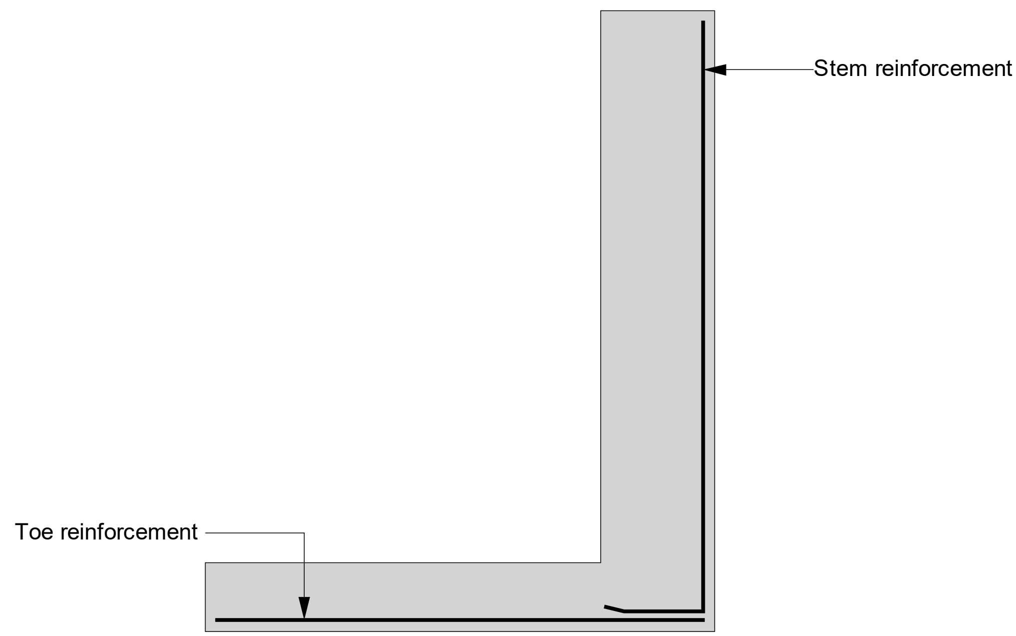
$v_{stem} < v_{c\_stem}$ - No shear reinforcement required

Check retaining wall deflection

Basic span/effective depth ratio	$ratio_{bas}$ = **7**
Design service stress	$f_s = 2 \times f_y \times As_{\_stem\_req} / (3 \times As_{\_stem\_prov})$ = **247.9** N/mm^2
Modification factor	$factor_{tens} = \min(0.55 + (477\ \mathrm{N/mm^2} - f_s)/(120 \times (0.9\ \mathrm{N/mm^2} + (M_{stem}/(b \times d_{stem}^2)))), 2)$ = **2.00**
Maximum span/effective depth ratio	$ratio_{max} = ratio_{bas} \times factor_{tens}$ = **14.00**
Actual span/effective depth ratio	$ratio_{act} = h_{stem} / d_{stem}$ = **5.42**

PASS - Span to depth ratio is acceptable

Indicative retaining wall reinforcement diagram



Toe bars - 20 mm dia.@ 200 mm centres - (1571 mm^2/m)

Stem bars - 16 mm dia.@ 200 mm centres - (1005 mm^2/m)

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250mm THICK RETAINING WALL

DEAD LOAD

VERTICAL LOAD:	
Flat Roof	1.30 kN/m^2 x 1.6m = 2.10 kN/m
Ground Floor	6.80 kN/m^2 x 3.4m = 23.12 kN/m
TOTAL:	**25.22 kN/m**
New Cavity Wall	4.04 kN/m^2 x 4.3m = **8.34 kN/m**
GROUND FORCE:	(trapeziondal force) height 4.4m, γ = 18.5 kN/m^3

IMPOSED LOAD

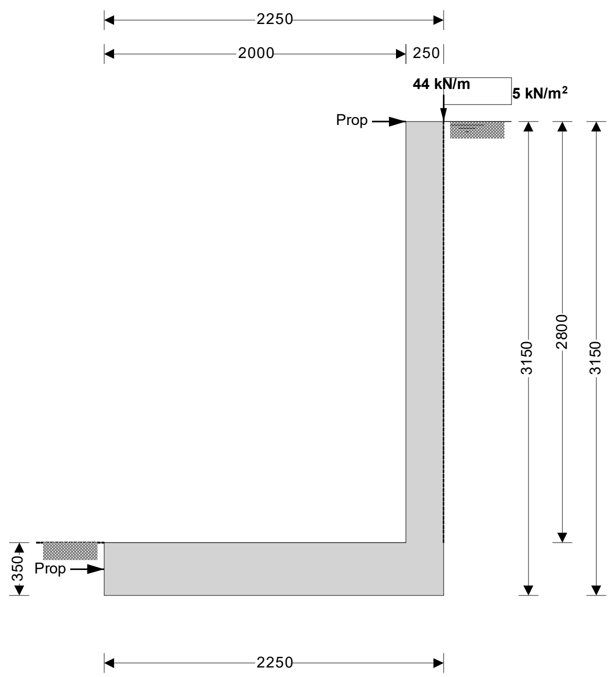
VERTICAL LOAD:	
Flat Roof	1.5 kN/m^2 x 1.6m = 2.40 kN/m
Ground Floor	2.5 kN/m^2 x 3.4m = 8.50 kN/m
TOTAL:	**10.90 kN/m**
SURCHARGE:	5 kN/m^2
WATER:	Full high water level 3.0m above slab

250mm linning wall to be propped at the top and at the bottom by RC slab.

Tekla Tedds Mbp	Project 25 OAK HILL ROAD, LONDON				Job Ref. 8415	
	Section RETAINING WALL				Sheet no./rev. 7. 14	
	Calc. by AZ	Date 21/10/2022	Chk'd by	Date	App'd by TH	Date 21/10/2022

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.08



Wall details

Retaining wall type	**Cantilever propped at both**
Height of retaining wall stem	h_{stem} = **2800** mm
Thickness of wall stem	t_{wall} = **250** mm
Length of toe	l_{toe} = **2000** mm
Length of heel	l_{heel} = **0** mm
Overall length of base	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **2250** mm
Thickness of base	t_{base} = **350** mm
Depth of downstand	d_{ds} = **0** mm
Position of downstand	l_{ds} = **500** mm
Thickness of downstand	t_{ds} = **350** mm
Height of retaining wall	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **3150** mm
Depth of cover in front of wall	d_{cover} = **0** mm
Depth of unplanned excavation	d_{exc} = **0** mm
Height of ground water behind wall	h_{water} = **3150** mm
Height of saturated fill above base	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0\ \text{mm})$ = **2800** mm
Density of wall construction	γ_{wall} = **24.0** kN/m^3
Density of base construction	γ_{base} = **24.0** kN/m^3
Angle of rear face of wall	α = **90.0** deg
Angle of soil surface behind wall	β = **0.0** deg
Effective height at virtual back of wall	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **3150** mm

Retained material details

Mobilisation factor	M = **1.5**

Moist density of retained material γ_m = **18.5** kN/m^3

Saturated density of retained material γ_s = **21.5** kN/m^3

Design shear strength ϕ' = **34.0** deg

Angle of wall friction δ = **0.7** deg

Base material details

Stiff clay

Moist density γ_{mb} = **18.5** kN/m^3

Design shear strength ϕ'_b = **34.0** deg

Design base friction δ_b = **0.7** deg

Allowable bearing pressure $P_{bearing}$ = **300** kN/m^2

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))}]^2) = \mathbf{0.281}$$

Passive pressure coefficient for base material

$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))}]^2) = \mathbf{3.620}$$

At-rest pressure

At-rest pressure for retained material $K_0 = 1 - \sin(\phi')$ = **0.441**

Loading details

Surcharge load on plan Surcharge = **5.0** kN/m^2

Applied vertical dead load on wall W_{dead} = **33.6** kN/m

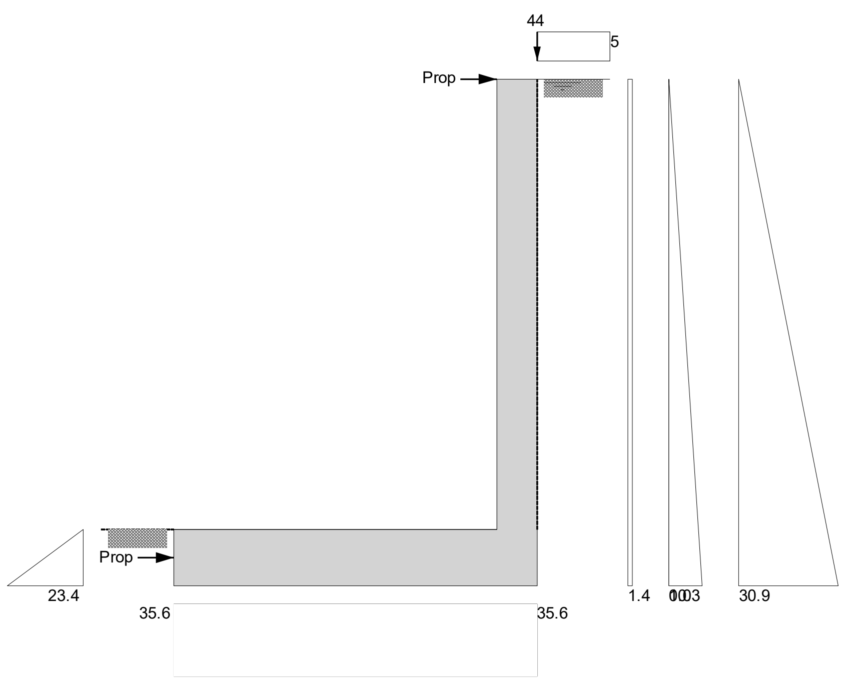
Applied vertical live load on wall W_{live} = **10.9** kN/m

Position of applied vertical load on wall l_{load} = **2250** mm

Applied horizontal dead load on wall F_{dead} = **0.0** kN/m

Applied horizontal live load on wall F_{live} = **0.0** kN/m

Height of applied horizontal load on wall h_{load} = **0** mm



Loads shown in kN/m, pressures shown in kN/m^2

Vertical forces on wall

Wall stem $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **16.8** kN/m

Wall base $w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **18.9** kN/m

Applied vertical load $W_v = W_{dead} + W_{live}$ = **44.5** kN/m

Total vertical load $W_{total} = w_{wall} + w_{base} + W_v$ = **80.2** kN/m

Horizontal forces on wall

Surcharge $F_{sur} = K_a \times \cos(90 - \alpha + \delta) \times \text{Surcharge} \times h_{eff}$ = **4.4** kN/m

Moist backfill below water table $F_{m\_b} = K_a \times \cos(90 - \alpha + \delta) \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **0** kN/m

Saturated backfill $F_s = 0.5 \times K_a \times \cos(90 - \alpha + \delta) \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **16.3** kN/m

Water $F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water}$ = **48.7** kN/m

Total horizontal load $F_{total} = F_{sur} + F_{m\_b} + F_s + F_{water}$ = **69.4** kN/m

Calculate total propping force

Passive resistance of soil in front of wall $F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **4.1** kN/m

Propping force $F_{prop} = \max(F_{total} - F_p - (W_{total} - W_{live}) \times \tan(\delta_b), 0 \text{ kN/m})$

F_{prop} = **64.4** kN/m

Overturning moments

Surcharge $M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **7** kNm/m

Moist backfill below water table $M_{m\_b} = F_{m\_b} \times (h_{water} - 2 \times d_{ds}) / 2$ = **0** kNm/m

Saturated backfill $M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3$ = **17.1** kNm/m

Water $M_{water} = F_{water} \times (h_{water} - 3 \times d_{ds}) / 3$ = **51.1** kNm/m

Total overturning moment $M_{ot} = M_{sur} + M_{m\_b} + M_s + M_{water}$ = **75.2** kNm/m

Restoring moments

Wall stem $M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **35.7** kNm/m

Wall base $M_{base} = w_{base} \times l_{base} / 2$ = **21.3** kNm/m

Design vertical dead load $M_{dead} = W_{dead} \times l_{load}$ = **75.5** kNm/m

Total restoring moment $M_{rest} = M_{wall} + M_{base} + M_{dead}$ = **132.5** kNm/m

Check bearing pressure

Total vertical reaction $R = W_{total}$ = **80.2** kN/m

Distance to reaction $x_{bar} = l_{base} / 2$ = **1125** mm

Eccentricity of reaction $e = \text{abs}((l_{base} / 2) - x_{bar})$ = **0** mm

Reaction acts within middle third of base

Bearing pressure at toe $p_{toe} = (R / l_{base}) - (6 \times R \times e / l_{base}^2)$ = **35.6** kN/m^2

Bearing pressure at heel $p_{heel} = (R / l_{base}) + (6 \times R \times e / l_{base}^2)$ = **35.6** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

Calculate propping forces to top and base of wall

Propping force to top of wall

$F_{prop\_top} = (M_{ot} - M_{rest} + R \times l_{base} / 2 - F_{prop} \times t_{base} / 2) / (h_{stem} + t_{base} / 2)$ = **7.263** kN/m

Propping force to base of wall $F_{prop\_base} = F_{prop} - F_{prop\_top}$ = **57.174** kN/m

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.08

Ultimate limit state load factors

Dead load factor γ_{f_d} = **1.4**

Live load factor γ_{f_l} = **1.6**

Earth and water pressure factor γ_{f_e} = **1.4**

Factored vertical forces on wall

Wall stem $w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall}$ = **23.5** kN/m

Wall base $w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base}$ = **26.5** kN/m

Applied vertical load $W_{v\_f} = \gamma_{f\_d} \times W_{dead} + \gamma_{f\_l} \times W_{live}$ = **64.4** kN/m

Total vertical load $W_{total\_f} = w_{wall\_f} + w_{base\_f} + W_{v\_f}$ = **114.4** kN/m

Factored horizontal at-rest forces on wall

Surcharge $F_{sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times h_{eff}$ = **11.1** kN/m

Moist backfill below water table $F_{m\_b\_f} = \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **0** kN/m

Saturated backfill $F_{s\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **35.8** kN/m

Water $F_{water\_f} = \gamma_{f\_e} \times 0.5 \times h_{water}^2 \times \gamma_{water}$ = **68.1** kN/m

Total horizontal load $F_{total\_f} = F_{sur\_f} + F_{m\_b\_f} + F_{s\_f} + F_{water\_f}$ = **115** kN/m

Calculate total propping force

Passive resistance of soil in front of wall $F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **5.7** kN/m

Propping force $F_{prop\_f} = \max(F_{total\_f} - F_{p\_f} - (W_{total\_f} - \gamma_{f\_l} \times W_{live}) \times \tan(\delta_b), 0\text{ kN/m})$

F_{prop_f} = **108.1** kN/m

Factored overturning moments

Surcharge $M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **17.5** kNm/m

Moist backfill below water table $M_{m\_b\_f} = F_{m\_b\_f} \times (h_{water} - 2 \times d_{ds}) / 2$ = **0** kNm/m

Saturated backfill $M_{s\_f} = F_{s\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **37.6** kNm/m

Water $M_{water\_f} = F_{water\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **71.5** kNm/m

Total overturning moment $M_{ot\_f} = M_{sur\_f} + M_{m\_b\_f} + M_{s\_f} + M_{water\_f}$ = **126.6** kNm/m

Restoring moments

Wall stem $M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2)$ = **50** kNm/m

Wall base $M_{base\_f} = w_{base\_f} \times l_{base} / 2$ = **29.8** kNm/m

Design vertical load $M_{v\_f} = W_{v\_f} \times l_{load}$ = **145** kNm/m

Total restoring moment $M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{v\_f}$ = **224.7** kNm/m

Factored bearing pressure

Total vertical reaction $R_f = W_{total\_f}$ = **114.4** kN/m

Distance to reaction $x_{bar\_f} = l_{base} / 2$ = **1125** mm

Eccentricity of reaction $e_f = \text{abs}((l_{base} / 2) - x_{bar\_f})$ = **0** mm

Reaction acts within middle third of base

Bearing pressure at toe $p_{toe\_f} = (R_f / l_{base}) - (6 \times R_f \times e_f / l_{base}^2)$ = **50.8** kN/m²

Bearing pressure at heel $p_{heel\_f} = (R_f / l_{base}) + (6 \times R_f \times e_f / l_{base}^2)$ = **50.8** kN/m²

Rate of change of base reaction $rate = (p_{toe\_f} - p_{heel\_f}) / l_{base}$ = **0.00** kN/m²/m

Bearing pressure at stem / toe $p_{stem\_toe\_f} = \max(p_{toe\_f} - (rate \times l_{toe}), 0\text{ kN/m}^2)$ = **50.8** kN/m²

Bearing pressure at mid stem $p_{stem\_mid\_f} = \max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall} / 2)), 0\ kN/m^2) = \mathbf{50.8}\ kN/m^2$

Bearing pressure at stem / heel $p_{stem\_heel\_f} = \max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall})), 0\ kN/m^2) = \mathbf{50.8}\ kN/m^2$

Calculate propping forces to top and base of wall

Propping force to top of wall

$F_{prop\_top\_f} = (M_{ot\_f} - M_{rest\_f} + R_f \times l_{base} / 2 - F_{prop\_f} \times t_{base} / 2) / (h_{stem} + t_{base} / 2) = \mathbf{3.934}\ kN/m$

Propping force to base of wall $F_{prop\_base\_f} = F_{prop\_f} - F_{prop\_top\_f} = \mathbf{104.177}\ kN/m$

Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

Characteristic strength of concrete $f_{cu} = \mathbf{35}\ N/mm^2$

Characteristic strength of reinforcement $f_y = \mathbf{500}\ N/mm^2$

Base details

Minimum area of reinforcement $k = \mathbf{0.13}\ \%$

Cover to reinforcement in toe $c_{toe} = \mathbf{50}\ mm$

Calculate shear for toe design

Shear from bearing pressure $V_{toe\_bear} = (p_{toe\_f} + p_{stem\_toe\_f}) \times l_{toe} / 2 = \mathbf{101.7}\ kN/m$

Shear from weight of base $V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base} = \mathbf{23.5}\ kN/m$

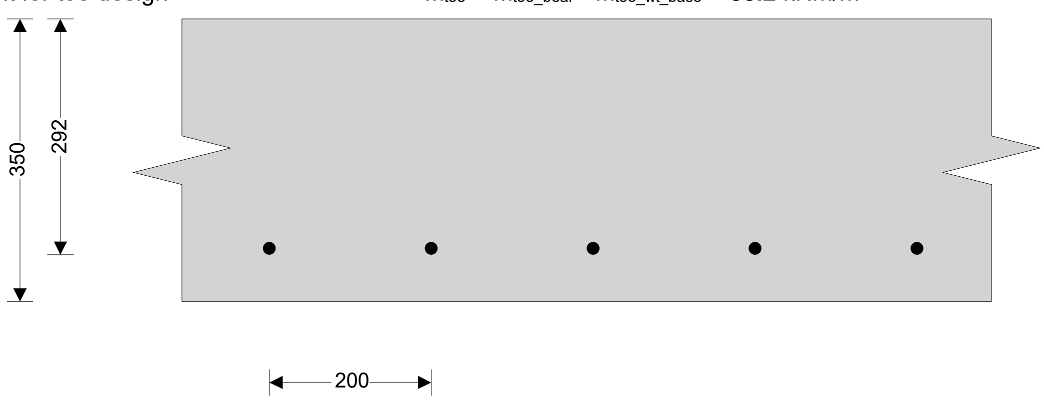
Total shear for toe design $V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base} = \mathbf{78.2}\ kN/m$

Calculate moment for toe design

Moment from bearing pressure $M_{toe\_bear} = (2 \times p_{toe\_f} + p_{stem\_mid\_f}) \times (l_{toe} + t_{wall} / 2)^2 / 6 = \mathbf{114.8}\ kNm/m$

Moment from weight of base $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2) = \mathbf{26.6}\ kNm/m$

Total moment for toe design $M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base} = \mathbf{88.2}\ kNm/m$



Check toe in bending

Width of toe $b = \mathbf{1000}\ mm/m$

Depth of reinforcement $d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2) = \mathbf{292.0}\ mm$

Constant $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu}) = \mathbf{0.030}$

Compression reinforcement is not required

Lever arm $z_{toe} = \min(0.5 + \sqrt{(0.25 - (\min(K_{toe}, 0.225) / 0.9))}, 0.95) \times d_{toe}$

$z_{toe} = \mathbf{277}\ mm$

Area of tension reinforcement required $As_{toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe}) = \mathbf{731}\ mm^2/m$

Minimum area of tension reinforcement $As_{toe\_min} = k \times b \times t_{base} = \mathbf{455}\ mm^2/m$

Area of tension reinforcement required $As_{toe\_req} = \mathrm{Max}(As_{toe\_des}, As_{toe\_min}) = \mathbf{731}\ mm^2/m$

Reinforcement provided **16 mm dia.bars @ 200 mm centres**

Area of reinforcement provided As_{toe_prov} = **1005** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress $v_{toe} = V_{toe} / (b \times d_{toe})$ = **0.268** N/mm^2

Allowable shear stress $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1 N/mm^2)}, 5) \times 1 N/mm^2$ = **4.733** N/mm^2

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress v_{c_toe} = **0.536** N/mm^2

$v_{toe} < v_{c\_toe}$ - No shear reinforcement required

Design of reinforced concrete retaining wall stem (BS 8002:1994)

Material properties

Characteristic strength of concrete f_{cu} = **35** N/mm^2

Characteristic strength of reinforcement f_y = **500** N/mm^2

Wall details

Minimum area of reinforcement k = **0.13** %

Cover to reinforcement in stem c_{stem} = **50** mm

Cover to reinforcement in wall c_{wall} = **50** mm

Factored horizontal at-rest forces on stem

Surcharge $F_{s\_sur\_f} = \gamma_{f\_l} \times K_0 \times Surcharge \times (h_{eff} - t_{base} - d_{ds})$ = **9.9** kN/m

Moist backfill below water table $F_{s\_m\_b\_f} = \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat}) \times h_{sat}$ = **0** kN/m

Saturated backfill $F_{s\_s\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{sat}^2$ = **28.3** kN/m

Water $F_{s\_water\_f} = 0.5 \times \gamma_{f\_e} \times \gamma_{water} \times h_{sat}^2$ = **53.8** kN/m

Calculate shear for stem design

Surcharge $V_{s\_sur\_f} = 5 \times F_{s\_sur\_f} / 8$ = **6.2** kN/m

Moist backfill below water table $V_{s\_m\_b\_f} = F_{s\_m\_b\_f} \times (8 - (n^2 \times (4 - n))) / 8$ = **0** kN/m

Saturated backfill $V_{s\_s\_f} = F_{s\_s\_f} \times (1 - (a_l^2 \times ((5 \times L) - a_l) / (20 \times L^3)))$ = **22.6** kN/m

Water $V_{s\_water\_f} = F_{s\_water\_f} \times (1 - (a_l^2 \times ((5 \times L) - a_l) / (20 \times L^3)))$ = **43.1** kN/m

Total shear for stem design $V_{stem} = V_{s\_sur\_f} + V_{s\_m\_b\_f} + V_{s\_s\_f} + V_{s\_water\_f}$ = **71.9** kN/m

Calculate moment for stem design

Surcharge $M_{s\_sur} = F_{s\_sur\_f} \times L / 8$ = **3.7** kNm/m

Moist backfill below water table $M_{s\_m\_b} = F_{s\_m\_b\_f} \times a_l \times (2 - n)^2 / 8$ = **0** kNm/m

Saturated backfill $M_{s\_s} = F_{s\_s\_f} \times a_l \times ((3 \times a_l^2)-(15 \times a_l \times L)+(20 \times L^2))/(60 \times L^2)$ = **11.2** kNm/m

Water $M_{s\_water} = F_{s\_water\_f} \times a_l \times ((3 \times a_l^2)-(15 \times a_l \times L)+(20 \times L^2))/(60 \times L^2)$ = **21.4** kNm/m

Total moment for stem design $M_{stem} = M_{s\_sur} + M_{s\_m\_b} + M_{s\_s} + M_{s\_water}$ = **36.2** kNm/m

Calculate moment for wall design

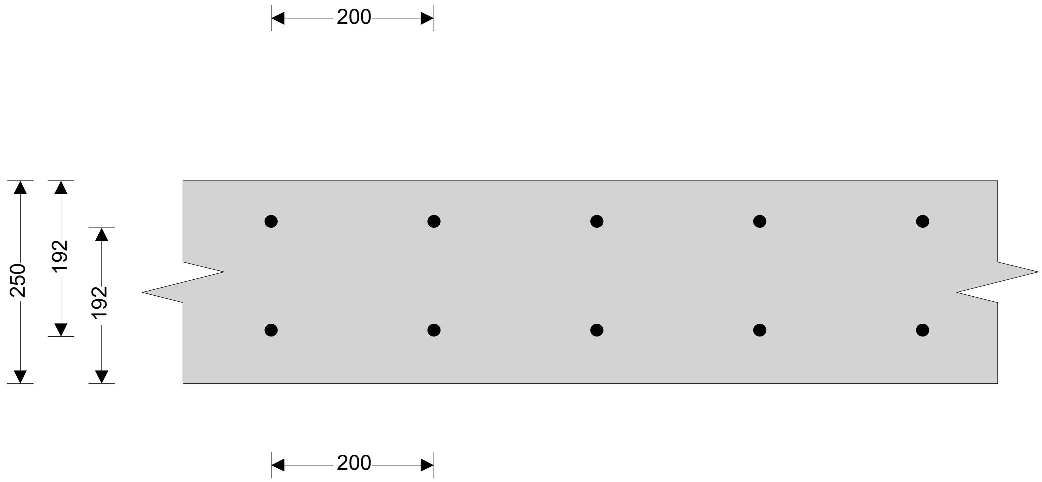
Surcharge $M_{w\_sur} = 9 \times F_{s\_sur\_f} \times L / 128$ = **2.1** kNm/m

Moist backfill below water table $M_{w\_m\_b} = F_{s\_m\_b\_f} \times a_l \times [((8-n^2 \times (4-n))^2 /16)-4+n \times (4-n)]/8$ = **0** kNm/m

Saturated backfill $M_{w\_s} = F_{s\_s\_f} \times [a_l^2 \times x \times ((5 \times L)-a_l)/(20 \times L^3)-(x-b_l)^3 /(3 \times a_l^2)]$ = **5** kNm/m

Water $M_{w\_water} = F_{s\_water\_f} \times [a_l^2 \times x \times ((5 \times L)-a_l)/(20 \times L^3)-(x-b_l)^3 /(3 \times a_l^2)]$ = **9.6** kNm/m

Total moment for wall design $M_{wall} = M_{w\_sur} + M_{w\_m\_b} + M_{w\_s} + M_{w\_water}$ = **16.6** kNm/m



Check wall stem in bending

Width of wall stem — b = **1000** mm/m

Depth of reinforcement — $d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2) =$ **192.0** mm

Constant — $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu}) =$ **0.028**

Compression reinforcement is not required

Lever arm — $z_{stem} = \min(0.5 + \sqrt{(0.25 - (\min(K_{stem}, 0.225) / 0.9))},0.95) \times d_{stem}$

$z_{stem} =$ **182** mm

Area of tension reinforcement required — $A_{s\_stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem}) =$ **457** mm^2/m

Minimum area of tension reinforcement — $A_{s\_stem\_min} = k \times b \times t_{wall} =$ **325** mm^2/m

Area of tension reinforcement required — $A_{s\_stem\_req} = \mathrm{Max}(A_{s\_stem\_des}, A_{s\_stem\_min}) =$ **457** mm^2/m

Reinforcement provided — **16 mm dia.bars @ 200 mm centres**

Area of reinforcement provided — $A_{s\_stem\_prov} =$ **1005** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress — $v_{stem} = V_{stem} / (b \times d_{stem}) =$ **0.374** N/mm^2

Allowable shear stress — $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1\ N/mm^2)}, 5) \times 1\ N/mm^2 =$ **4.733** N/mm^2

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress — $v_{c\_stem} =$ **0.685** N/mm^2

$v_{stem} < v_{c\_stem}$ - No shear reinforcement required

Check mid height of wall in bending

Depth of reinforcement — $d_{wall} = t_{wall} - c_{wall} - (\phi_{wall} / 2) =$ **192.0** mm

Constant — $K_{wall} = M_{wall} / (b \times d_{wall}^2 \times f_{cu}) =$ **0.013**

Compression reinforcement is not required

Lever arm — $z_{wall} = \mathrm{Min}(0.5 + \sqrt{(0.25 - (\min(K_{wall}, 0.225) / 0.9))},0.95) \times d_{wall}$

$z_{wall} =$ **182** mm

Area of tension reinforcement required — $A_{s\_wall\_des} = M_{wall} / (0.87 \times f_y \times z_{wall}) =$ **210** mm^2/m

Minimum area of tension reinforcement — $A_{s\_wall\_min} = k \times b \times t_{wall} =$ **325** mm^2/m

Area of tension reinforcement required — $A_{s\_wall\_req} = \mathrm{Max}(A_{s\_wall\_des}, A_{s\_wall\_min}) =$ **325** mm^2/m

Reinforcement provided **16 mm dia.bars @ 200 mm centres**

Area of reinforcement provided $A_{s\_wall\_prov}$ = **1005** mm^2/m

PASS - Reinforcement provided to the retaining wall at mid height is adequate

Check retaining wall deflection

Basic span/effective depth ratio $ratio_{bas}$ = **20**

Design service stress $f_s = 2 \times f_y \times A_{s\_stem\_req} / (3 \times A_{s\_stem\_prov})$ = **151.5** N/mm^2

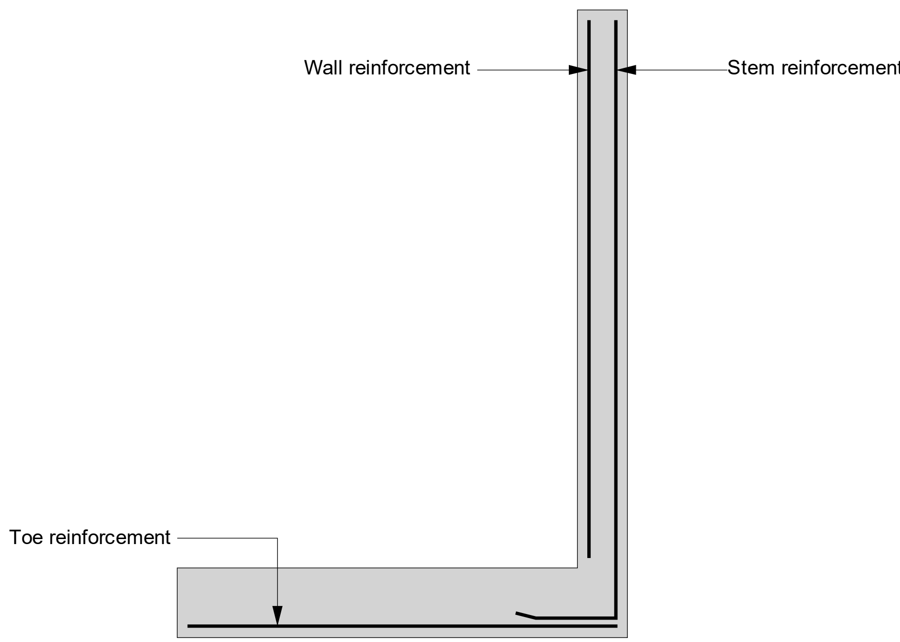
Modification factor $factor_{tens} = \min(0.55 + (477\ N/mm^2 - f_s)/(120 \times (0.9\ N/mm^2 + (M_{stem}/(b \times d_{stem}^2)))),2)$ = **1.99**

Maximum span/effective depth ratio $ratio_{max} = ratio_{bas} \times factor_{tens}$ = **39.81**

Actual span/effective depth ratio $ratio_{act} = h_{stem} / d_{stem}$ = **14.58**

PASS - Span to depth ratio is acceptable

Indicative retaining wall reinforcement diagram



Toe bars - 16 mm dia.@ 200 mm centres - (1005 mm^2/m)
Wall bars - 16 mm dia.@ 200 mm centres - (1005 mm^2/m)
Stem bars - 16 mm dia.@ 200 mm centres - (1005 mm^2/m)

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